The Foundations of Quiminha Dam

Les Fondations du Barrage de Quiminha

J.FOLQUE Head Foundation Division,
F.G.DE MELO Research Officer, Laboratório Nacional de Engenharia Civil, Lisboa, Portugal

SYNOPSIS The paper contains a description of soil conditions at Quiminha Dam site, particular emphasis being placed on foundations problems. Due to the high permeability of the very deep foundation alluvial soils a diaphragm wall was built, completely intersecting the alluvial valley.

Observations results are given stressing the aspects related with the foundations behaviour.

1 - INTRODUCTION

The present article contains a brief description of the soil survey of Quiminha dam site. The construction works are also described, particular emphasis being placed on foundation problems. In fact, the characteristics of the alluvial foundation soil have in this case strongly influenced the design adopted, with a diaphragm wall completely intersecting the alluvial valley.

Observation results are given, also stressing the aspects related with the foundations. Laboratório Nacional de Engenharia Civil, through the writers, has guided the soil surves studies and has acted as consultor in the design, construction and observation. The design was made by Société d'Études d'Equipment d'Entreprises, with collaboration of Hecasol. Construtora do Tamega was the general contractor, the construction of the diaphragm wall having been entrusted to Sondagens Rödio. The laboratory tests, construction control and observation works had the local support of Laboratório de Engenharia de Angola.

2 - GEOLOGICAL CHARACTERISTICS OF THE SITE

Geologically, Quiminha dam site is a valley excavated by the river Bengo in the maristones of the early Eocene. This excavation started at a plateau which lies near elevation + 90 and gave rise to a slightly asymmetric valley, with a more rounded slope on the left bank, extending in depth down to elevations of about - 23 m.

The bottom of the valley is filled by quaternary sedimentary formations up to elevations of about 20, where the major bed of the river lies, which, at the axis of the dam, has a width of about 170 m. The minor bed, with a width of about 40 m and depths near elevation 19, is out of line with the axial center of the valley and nearer to the left bank.

As regards formations at the base, which have already been said to be limestones, geological studies show that, at the site of the dam, they have not been much affected by orogenic movements. Therefore, fracturing is as a rule not very marked, althrough it is of some importance on the surface layers.

Traces of karstification are also rather small. It is therefore concluded that they will not pose significant problems regarding the construction of an earth dam, which does however not exclude the need for grounting curtains, particularly near the abutments, at the highest elevations.

As to quaternary formations, which in some places reach thicknesses of more than 40 m, the survey carried out shows that the sediments are clay and sand laid out in approximately horizontal layers with very variable thicknesses. On the whole, the more superficial layers are a mixture of quartzous sands and clays of the illite type. They are followed, at the lower elevations, by silt and clay deposits of very variable thickness, and finally, at the deeper zones of the valley, sand and gravel predominate.

Fig. 1 shows the geologic cross-section of the valley at the axis of the dam. From the studies carried out it can be concluded that the order of the sediments is the same throughout the whole foundation zone, both upstream and downstream.

As can be seen from this brief description of the geology of the site, the behaviour of the quaternary formations is particularly important for the design of the dam and these formations where thus carefully studied and characterized, as shown hereafter.

3 - GEOTECHNICAL CHARACTERISTICS OF THE QUATERNARY FORMATIONS

With a view to the design, three distinct layers were considered in a schematic way, corresponding to the three main geologic formations. Each layer was the object of studies adapted to its characteristics and to the importance it was supposed to have in

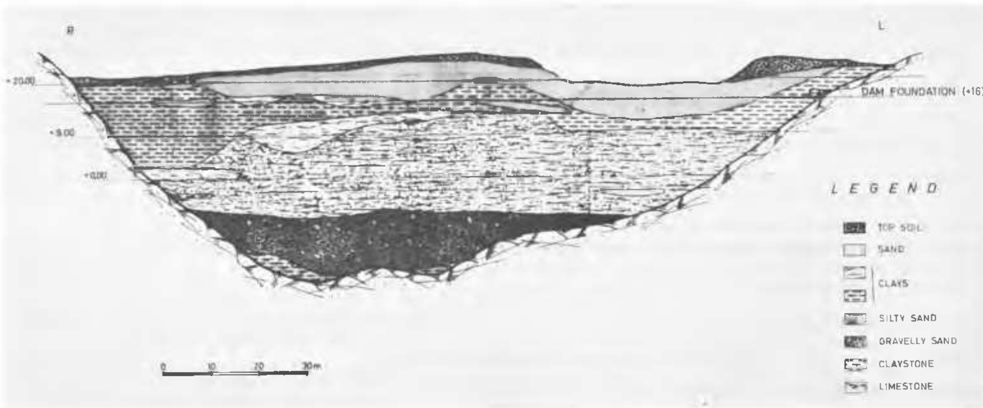

Fig. 1 - Geologic section looking upstream

the general context of the work. The studies carried out and the results thereof are given hereafter.

3.1 - Superficial sand-clay layer

This layer has little importance as regards design and behaviour of the work because, in addition to being rather thin, most of it will be removed during the cleaning out operation for construction of the dam. Its characterization was therefore not very developed and, summing up, it can be said that in the slow triaxial consolidated tests it showed a friction angle of 36^{o} and a cohesion of 0.3 kg/cm^2. Permeability was about 3×10^{-3} cm/sec.

3.2 - Clay layer

This layer has a very variable thickness, ranging from about 20 m near the right bank to almost nothing in some points of the left bank. As this is a layer with clay characteristics, the very marked difference in thickness has strong implications in the deformations and seems to indicate that high differential settlements along the axis of the work are likely to occur.

Although it is not a soil layer with homogeneous characteristics, an overall interpretation of the results of the tests carried out makes it possible to consider statistical values that are representative of its behaviour. It can thus be said that these soils show plasticity index of about 16% with a mean liquid limit of 39%. The natural water content is about 29%.

As to the deformability characteristics it was found out that the average compressibility coefi- cient, C_c, is 0.25 and the consolidation coefficient c_v is 5.10^{-3} cm^2/s.

Regarding shear strength characteristics, the values obtained in the triaxial tests for cohesion and friction angle, as regards effective stresses, were respectively 0.2 kg/cm^2 and 28^{o}. As regards total stresses, their behaviour corresponds to a cohesion of o.7 kg/cm^2 with zero angle of friction.

3.3 - Sand layer

This is a thick sand layer, in which the grain size varies much with the depth. It goes from very fine sand at the upper levels to gravel sizes in the deeper zones of the valley. As is to be expected with this kind of formation sampling was not accurate. Static penetration tests were therefore made to characterize this formation from the mechanical point of view. The results of the tests led to the conclusion that the soils are not very compact, friction angles of about 30 to 32^{o} having been found.

The permeability characteristics were determined by means of pumping tests carried out inside boreholes, piezometers having been installed around these holes. The interpretation of the results led to the conclusion that on the upper layers, where the grain sizes are smaller, permeability ranges from 1×10^{-2} to 4×10^{-3} cm/s, whereas for lower elevations it reaches values of 2×10^{-1} cm/s.

4 - GENERAL CHARACTERISTICS OF THE PROJECT

The quaternary formations where the dam is founded show rather unfavourable characteristics, since they contain both highly permeable sandy zone and clay zones constituting large pockets of materials with considerable plasticity and a high deformability. High percolation through the sand and large settlements through consolidation of the clay layers are thus to be expected. It was in fact this double aspect of the characteristics of the foundation that conditioned the design of the dam.

To counteract the consequences of the high percolations, two types of solutions were considered to begin with: an impervious blanket upstream and a watertight wall cutting the whole quaternary filling of the valley crosswise. A comparative study of these two alternative led designers to opt for the second one. In fact, setting aside for the moment the type of watertight wall that is to be thick and that its permeability coefficient would be about 10^{-6} cm/s, it was concluded that to reach the same discharge the impervious upstream blanket would have the length of about 350 m. The construction of such a large blanket is particularly difficult in a case such as Quiminha because, in addition to being rather steep, the banks are also fairly sinuous, particularly the left bank. Besides, to guarantee the efficiency of this solution, considering the superficial cracking of the limes, it is to be expected that to prevent high percolation in those zones grouting in a large scale would have to be resorted to. Considering all these aspects, the economic study led to the conclusion that the blanket would cost about twice as much as the watertight wall and so the decision taken was in favour of this solution.

The decision about the type of solution once made, the next step would be to decide about the kind of wall best suited for the purpose, bearing in mind both the watertightness and the high settlements of the foundation. These will affect not only the wall but also the dam embankements.

After pondering the different hypoteses a slurry-wall diaphragm type was chosen, 0.80 m thick, made of a mixture of cement and clay. The deformability must be as similar as possible to the quaternary formations into which it would be fitted.

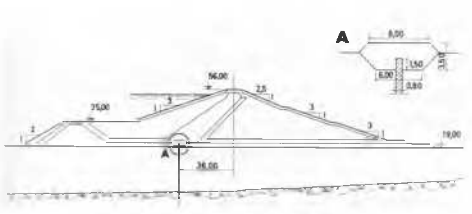

Fig. 2 - Cross-section of the dam

Before being able to go into details about the diaphragm wall, as for instance the definition of the filling material, the dam fill had to be defined and designed, taking into consideration not only the conditioning factors of quality and quantity of the available borrow materials but also the connection between the wall and the embankment. The foundation wall should in fact be continued through the embankment by a watertight element, which, together with the wall would carry out the fundamental mission of retaining the water in the reservoir. Considering the characteristics of the borrow materials, the waterproofing of the fill was entrusted to a clay core which extends from elevation 54, where it is 8 m wide, to elevation 16 where the width is 26 metres. In an attempt to diminish the vertical stresses in the diaphragm wall and consequently the settlements in connexion with it, the wall was placed 36 m further upstream of the axis of the dam thus originating the inclined shape of the core, as can be seen in Fig. 2.

The connection between the diaphragm wall and the core is of course a rather important point in the work since it is essential that it operates in perfect condition ensuring the continuity of the waterproofing devices. This involves that the connection can withstand the inevitable differences in the deformability of the wall and of the quaternary formations into which it will be fitted without the occurence of cracking or punching phenomena of the core.

For this purpose an intermediate piece was designed, constituted by a masscap of the diaphragm wall at the base of the embankment core and surrounded by it. This element, 3.5 m thick, should be made of a fairly plastic material with a high deformability and a clay was therefore chosen with a liquid limit of over 35%, compacted with a moisture content that exceeds by 2% the optimum Proctor. The wall length inside this cap, was established at 1.5 m.

For reasons related with the diversion of the river it was necessary to design a cofferdam at the upstream side. This cofferdam, designed so as to be incorporated in the dam embankment, would be covered by a clay layer with a thickness of about 3 meters. This layer being bonded to the core with no discontinuities, it acts as a small watertight blanket upstream. Although its small size will not enable it to control alone the percolations of the foundation, it will constitute an additional safety in case of accident in the foundation wall.

5 - STUDY OF THE MIXTURE THAT CONSTITUTES THE DIAPHRAGM WALL

One of the most important problems that had to be solved with regard to the diaphragm wall was that of the study of the composition of the filling material. In fact, in addition to low permeability, about 10^{-6} cm/s, the deformability of the wall must be sufficiently high to enable it to withstand localized deformations due to non uniform settlements, without cracking. Besides, still within the limiting factors regarding deformations, the deformability must have to be as close as possible to that of the quaternary formations to prevent punching effects. Under the circunstances, the accurate prediction of the settlements of the dam foundation was indispensable.

The calculation of the settlements was made in 25 points corresponding to the intersection points defined by 10 alignements, of which 5 parallel to the axis of the dam and 5 normal to it. The concentration of these points is much greater near the right bank because it is in this zone that the greatest thicknesses of the clay layers lie.

For each ofthese points was adopted a stratigraphy deduced from the elements obtained by borings carried out all over the place where the dam was to be located.

Several hypoteses were considered in the study, corresponding to several stresses associated with characteristics stages of the life of the work. Besides, for each one of these hypotheses, the values of the parameters representative of the behaviour of the soil were made to vary within the range of scattering that had been obtained in the laboratory tests.

Fig. 3 shows the settlements that were calculated taking as a basis the mean values of the parameters representative of the foundation soils, for each of the 25 points studied and for the three situations described below:

a) Immediatly after construction of the embankment in the hypothesis of it having been constructed at constant time rate, during 18 months.

b) For an infinite time after construction and for the case in which the dam remains unloaded.

c) For the case in which the dam has been loaded immediatly after construction, as said in a), and has remained loaded for an infinite time.

As can be seen from the values indicated, maximum settlements of about 1.70 m are to be expected near the right bank, that is to say, about 4% in relation to the thickness of the quaternary sediments.

Fig. 3 - Predicted settlements of the foundation

Taking as a basis the material available in the area, several mixtures were tested with a view to obtain a material that would withstand deformations of this order. Of all these materials, the one whose results more closely met the requirements of the design contained a 40% clay fraction with a liquid limit of 30%.

A mixture of this soil and cement in percentages of 80, 120 and 160 kg per cubic metre of final material led to results in triaxial tests to which corresponded, after 7 days, friction angles of 11^o to 15^o, and variable cohesions between 0.250 and 0.950 kg/cm^2. These values became, after 28 days, friction angles ranging from 15^o to 26^o and cohesions that varied between 1.250 and 2.450 kg/cm^2.

The modulus of deformability E, defined as the quotient between the main stress, and the corresponding strain, ε, lay between values of 75 to 175 kg/cm^2 for strains of 4%, that is, for the value predicted for the foundation settlements. For higher stresses, say 16%, the values of E would drop to values that ranged from 20 to 50 kg/cm^2.

It was also found in the tests that an increase in the cement content brought about an increase in the initial strength but that, on the other hand, it caused a slight decrease in the ability to withstand large deformation. It was therefore concluded that there is no point in increasing the cement content over 80 kg/m^3.

As regards the borrow material it was also concluded, by comparing its behaviour with that of other materials, that it would be well to increase its percentage of clay from 40 to about 50%.

With this slight correction the mixture adopted was as follows:

normal portland cement	80 kg
borrow material	850 kg
water	65 to 70%

The possibility of using a plastifier in the actual work was also admitted.

Another aspect considered in the study of the constitution of the wall filling was its capacity to withstand tangential stresses. These may be calculated, although in an approximate manner, by adopting an elastic solution in which the stress caused by the enbankment is computed considering a symetrical vertical triangular load distribution. With this hypothesis, and for the location of the wall, the maximum tangential stresses is given, according to Poulos and Davis (1951), by $\tau_{max}/P=0.238$ where the corresponding maximum principal stresses are $\sigma_1/_p = 0.638$ $\sigma_3/_p = 0.162$

where

$$p = \gamma H = 2 \times 37 = 74 \text{ t/m}^2$$

Thus:

$$\tau_{max} = 0.238 \times 74 = 18 \text{ t/m}^2$$
$$\sigma_1 = 0.638 \times 74 = 47 \text{ t/m}^2$$
$$\sigma_3 = 0.162 \times 74 = 12 \text{ t/m}^2$$

The mean stress will thus be

$$\sigma_m = 29{,}5 \text{ t/m}^2$$

As referred earlier in this paper, the study of the composition of the wall filling material led to the conclusion that, for a cement ratio of 80 kg/m^3 the strength of the mixture was characterized at 28 days by cohesion values of 1.250 and a friction angle of 15^o. Thus, for a mean stress of 29.5 t/m^2 the shear strength available in the material would be:

$$\tau = 12.5 + 29.5 \text{ tg } 15^o = 20 \text{ t/m}^2$$

6 - OBSERVATION OF THE DIAPHRAGM WALL

To study the behaviour of the diaphragm wall, neutral stress cells, open piezometers and settlement measuring devices were installed in the foundation, downstream of the wall.

The neutral stress cells were of the vibrating wire type. Twenty-four cells were installed, distributed along a plane parallel to that of the watertight wall, at a distance of 8 meters from this wall.

The open piezometers were placed along two profiles parallel to the axis of the dam (fig. 4), their ends being at about elevation 17.5 m (fig. 5).

The settlements of the foundation are being observed by batteries installed for measuring the vertical displacements of the dam embankment. In fact, in these batteries, it is possible to determine at any moment, starting at the upper end of the battery, the elevation of the different cross-arms that have been put in place as the embankment was being constructed. The assessment of the settlements of the foundation can be made with some certainty through the results concerning the evolution of the first cross-arm. Fig. 4 and 5 show the positions in plan and in transverse cross section of the first cross--arms of the 6 batteries installed.

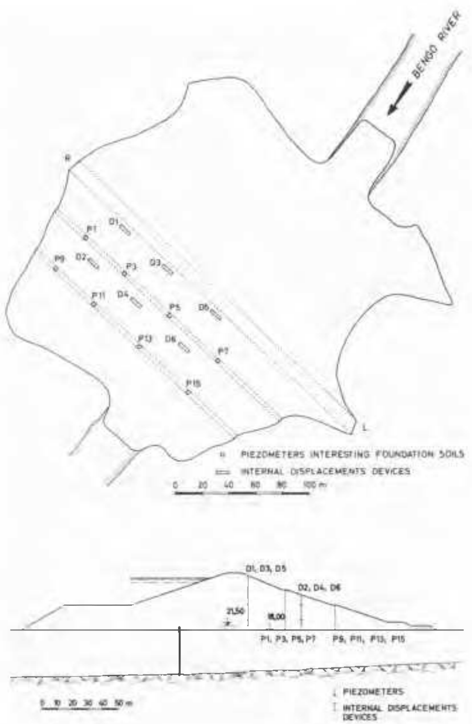

Fig. 4 and 5 - Position in plan and profile of observation devices

All this equipment was installed during the months of June and September 1973 and the results given hereafter concern the first year after installation, that is, up to August 1974.

Fig. 6 shows the diagrams of the evolution of the elevations of the embankments and of the storage of the reservoir. The same figure shows, as an example, the evolutions of the neutral stress in two cells (cells Nos. 2 and 8), of the water level in a piezometer (piezometer No. 3), and of the settlements of the foundation in the vertical direction of the point where a battery has been installed (No. 2).

From the analysis of the results obtained with the piezometers it can be concluded that the evolution of the water levels inside them is very similar, so that the behaviour of piezometer P_3, indicated in the figure, can be considered to be representative of the whole set.

It can thus be said that towards the end of July 1974 the mean of the values observed was very closely to the groundwater level downstream of the dam.

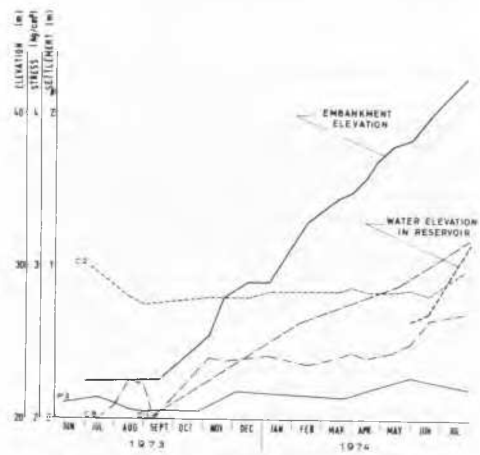

Fig. 6 - Evolution of observed data

As regards results of the neutral stress cells, two types of behaviour can be considered. In a group which includes the majority of the cells, it was found that the value indicated by them for the neutral stress shortly after their installation has remained practically constant. Considering the elevations of the groundwater levels deduced through the piezometers, it can be seen that the stresses read by these cells are very similar to the water pressures corresponding to the depths at which the cells have been placed. An instance of this kind of behaviour is given by cell No. 2 and is represented in figure 6. There is however a second group, constituted by only 5 cells (Nos. 6, 8, 21, 22 and 23) in which the rise in neutral stress linked with the growth of the embankment is very marked as can be seen from the behaviour of cell No. 8, which is also shown in fig. 6. It can be seen that for this group of cells the stresses recorded are sligthly greater than the water pressure corres - ponding to the elevation at which they were installed. At most this difference is about 0.7 kg/cm^2 in cell No. 8. This difference in behaviour must be due to the characteristics of the environment of the cells. Thus, those of the first group are probably situated in rather permeable sandy zones whereas those of the second group are situated in clay zones in which the dissipation of neutral stresses generated by the increase in total stress due to the growth of structure has been rather difficult. Fig. 7 shows the values of the neutral stresses by the end of July

1974.

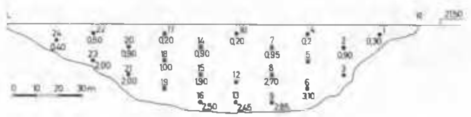

Fig. 7 - Pore-pressures downstream of the wall

Taking for instance cell No. 8, it can be seen that according to the geologic cross-section it is situated in a layer with clay characteristics and has a thickness of about 20 m. The elevation where it was installed was 0.5, the neutral stress, read immediately after installation, was 2.0 kg/cm^2, which corresponds approximately to the hydrostatic pressure for the existing groundwater level.

During the period mentioned in Fig. 6, while the embankment rose from elevation 22.5 m to elevation 42.5 cm, that is 20 metres, the neutral stress went from 2.0 to 2.7 kg/cm^2. Therefore, for an increase in total stress of about 2 x 20 = 40 t/m^2 the increase in neutral stress was only 7 t/m^2, that is to say that simultaneously the pore pressure has undergone a dissipation of 40 - 7 = 33 t/m^2.

Starting from these results of the observation of neutral stresses it is possible to derive the values of settlements and to compare them with the values that were actually obtained. In fact, settlement H undergone by a layer of thickness H can be calculated through expression

$$\Delta H = H \frac{c_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_o}{\sigma'_o}$$

With the material in question the values obtained in the tests for parameters c_c and e_o were respectively, $C_c = 0.25$ $e_o = 1$.

Thus

$$H = 1 \text{ m}$$

By comparing this value with the settlement actually observed during the same period at the point corresponding to displacement battery D1, which lies comparatively near the vertical that contains cell C_8, it is found that they are of the same order of magnitude. Thus, as in fig. 8, the settlement observed in D1 was 0.91 meters.

On looking at the results of the observations with piezometers and neutral stress cells it can be said that in this initial phase the behaviour of the diaphragm wall seems to have been satisfactory. In fact, although the water level of the reservoir has risen some metters, pressures downstream of the wall correspond very closely to the position of the groundwater level, which lies near elevation 22.

As regards the settlements of the foundations, the predictions of the design have on the whole been confirmed. Large settlements have in fact been observed, which, as can be seen in fig. 8, already reach 1 meter. It has also been confirmed that maximum settlements are located near the right bank and that differential settlements are considerable. For the moment and for the points observed they already reach values of about 0.60 meters.

Fig. 8 - Observed settlements

7 - CONCLUSIONS

As said at the beginning of this paper, the results of the observations analysed concern a time during the construction stage of the work, in which the embankment only reached about 70% of its maximum height. Nevertheless storage was already taking place and by the end of the time considered it amounted to about 40% of the total load.

It is not possible as yet to form conclusive opinions about the behaviour of the work, specially of the diaphragm wall. It can only be said that during the period considered the behaviour is normal, both as regards watertightness of the wall and settlements of the foundation. Thus, in the observation scheme installed downstream of the wall no pressures indicating anormal percolation through the foundation were detected, although there was already considerable difference in pressure between the upstream and downstream side of the dam.

As to the settlements that are being obtained, they are within the order of magnitude predicted in the design, both as regards absolute and differential values.