XIII ICSMFE, 1994. New Delhi, India / XIII CIMSTF, 1994. New Delhi, Inde

STRESSES AND DEFORMATIONS OF ZAHARA DAM

CONTRAINTES ET DEFORMATIONS DU BARRAGE DE ZAHARA

J.L. Ramirez-Vacas[1] *A. Soriano*[2] *F.J. Sanchez*[3]

[1]Confederación H. del Guadalquivir, Spain
[2]Universidad Politécnica de Madrid, Spain
[3]Ingenieria del Suelo S.A., Madrid, Spain

SYNOPSIS: The paper describes the data observed during the construction of Zahara dam (southern Spain) and its interpretation by means of a finite element model of the main cross section.

Zahara dam has a central core of weathered marls and outer shells of granular alluvium and quarried limestone. Its foundation consists of almost vertical alternating strata of tertiary marls and sandstones.

The mathematical formulation of the finite element model used for the interpretation includes the effect of time for the simulation of construction as well as for the period of observation after the end of construction. It also includes the effect of wetting deformations to predict movements and stresses during the filling of the reservoir which is starting these days.

INTRODUCTION

Zahara Dam, located in the province of Cádiz, Spain, has been built to regulate the upper course of the Guadalete river. The rock formation at the site consists of marls and sandstones of the tertiary age. The behaviour of the marls, when weathered, is that of an expansive clay. The dam axis was situated at a particular place where somewhat thicker sandstone strata appear, making the valley particularly narrow in comparison to other points of the river course.

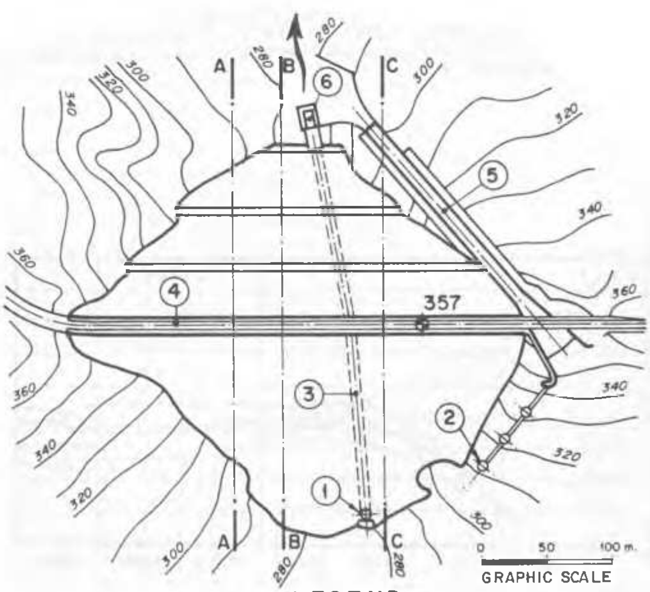

Fig. 1.- Plan view of Zahara dam

Since the foundation rock was rather weak, an earth dam was designed by use of the local marls to form the impervious central core and the river alluvium as inner shells. An outer protection of quarried limestone was provided at both dam slopes. The plan view and the typical cross section of the dam are depicted in Figs. 1 and 2.

The main characteristics of the marls, after excavation from the borrow areas and compacted at the dam core, are as follows: (average values and standard deviations)

- Dry density, γ_d = 16.2 ± 0.8 KN/m^3 (opt. standard Proctor)
- Water content, w = 20 ± 2 (%) (opt. standard Proctor)
- Liquid Limit, LL = 51 ± 4.9 %
- Plastic Limit, PL = 27 ± 4 %
- Effective cohesion, c' = 10 ± 2 KN/m^2
- Effective angle of friction, ϕ' = 22 ± 3 (°)
- Swelling pressure, p_s = 350 ± 100 KN/m^2
- Free swelling, ε_s = 8 ± 3 (%) (under a load of 1 KN/m^2)
- Dispersive nature: Positive under several kind of tests

The upper part of the impervious core has been built with selected marls of limited swelling pressure (p_s < 100 KN/m^2). The alluvial deposits, used as inner shells, is a well graded sandy gravel with a fine content (# 200 ASTM) around 6%. Main characteristics of this material are as follows:

- Dry density (opt. standard Proctor) = 216 ± 8 KN/m^3
- Water content (opt. standard Proctor) = 5.5 ± 0.5 (%)
- Plasticity of fines (IP) = 5 to 12 %
- Effective cohesion (compacted at O.S.P.) = 10 ± 6 KN/m^2
- Effective friction (Id) = 39° ± 4°

The limestone used to build the outer rock shells was rather weak (average porosity of 5 %, average Los Angeles coefficient of 50%) but, as was used under moderate loads, it presents a large angle of internal friction and low compressibility.

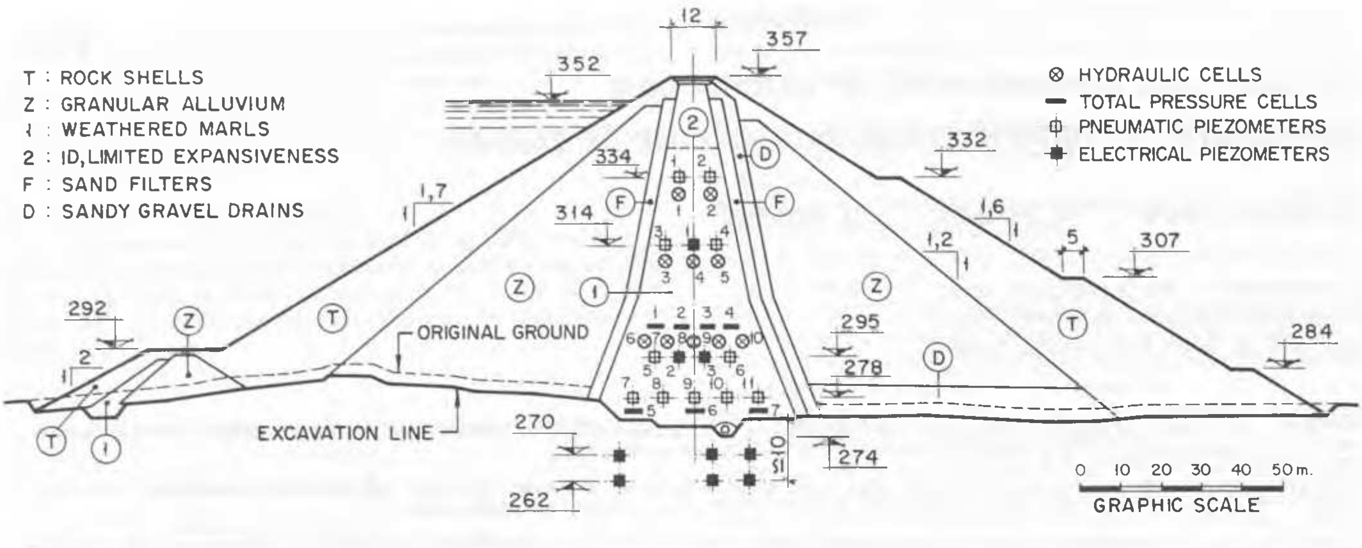

Fig. 2.- Cross section and instrumentation

MONITORING SYSTEM DATA

The monitoring system of the central cross section of the dam is indicated in Fig. 2. Two other cross sections were instrumented in a similar manner.

In addition to the hydraulic settlement cells a continuous topographical leveling of the bottom outlet conduit has been done during and after construction of the dam.

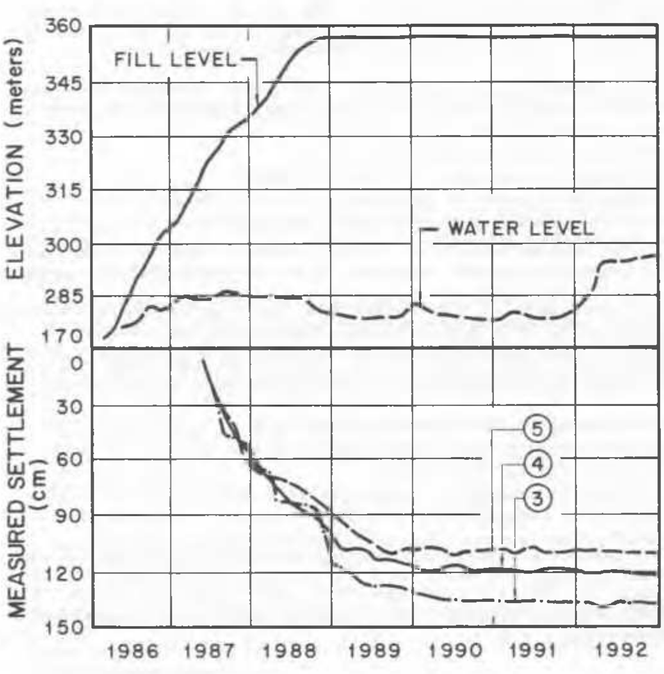

Fig. 3.- Evolution of construction settlements

This control allowed to measure the settlement of the base of the dam along a line from the upstream to the downstream toes.

Main data of settlement obtained from the hydraulic cells within the clay core indicated, as shown in Fig. 3, that the maximum construction settlement was 1.2 m and occurred at elevation 315 m (approx.), that is at a height from the foundation close to 1/2 of the total core height. Some increase in settlement has been observed after completion of the dam body, although that additional settlements did not go farther than 0.1 m at any point of observation.

Total vertical pressures within the dam core, monitored by vibrating wire pressure cells, indicate, as shown in Fig. 4, that a good part of the weight of the core was supported by the shells since the ratio of the total pressure to the geostatic vertical pressure has been recorded, as average, on the order of 50 %.

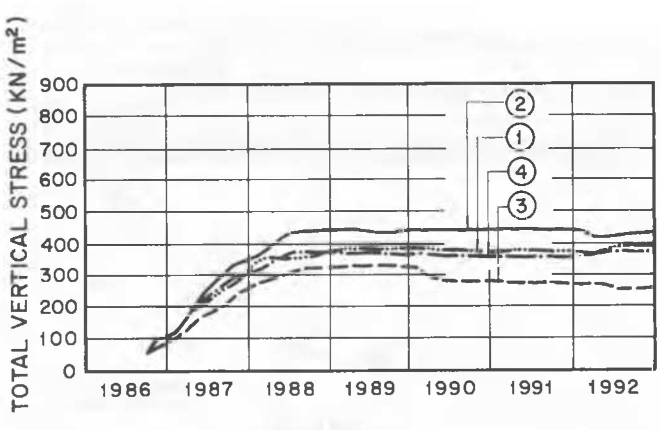

Fig. 4.- Evolution of total vertical stresses

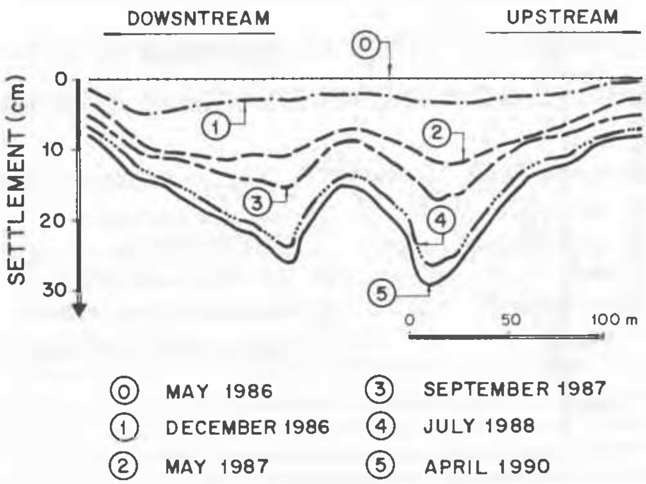

Fig. 5.- Settlement of the bottom outlet conduit

Settlements along the bottom outlet, indicate the same effect, as shown in Fig. 5. At the end of construction the settlement measured at the central part of the core was about one half of that measured at the contact of the core and the inner shells.

Most piezometers placed within the core of the dam indicated no generation of pore-pressures during construction. Only two of them, located at the deepest part of the core, indicated some pore-pressure generation during construction although that pressure did not amounted to more than 250 KN/m^2. The core was place at a water content somewhat lower that the optimum and at a density close to the maximum of the standard Proctor. The average degree of saturation after compaction was on the order of 80%.

The piezometers placed at the foundation indicated that no excess pore pressures were induced by the construction of the dam but at some local points. Foundation piezometric levels were controlled by the variable water level of the small reservoir created by the cofferdam.

BACK ANALYSIS

In order to help to the interpretation of the observed data, a finite element model of the dam main cross section was prepared as shown in Fig. 6

The stress-strain law used to simulate the construction of the dam, layer by layer, has been the well established hyperbolic formulation although two main modifications have been introduced to include the experience of the authors on the observation of dam behaviour.

The first modification refers to the time effects and although for this particular dam it has been observed of minor magnitude, it is worthwhile to be considered. For this purpose, it has been postulated that deformations within an element are given by a log function:

$$\varepsilon(t) = \varepsilon_o \left(1 + \alpha\, L_n \frac{t}{t_o}\right) \qquad (t > t_o)$$

To evaluate the parameters of this equation (ε_o, α, t_o) the behaviour of the materials under compression has been investigated by laboratory testing. Under oedometric conditions the decrease of the void ratio of the core clay when doubling the vertical load can be approached by:

$$\Delta e(t) = C_c\, lg_{10}\, 2 + C_a\, lg_{10} \frac{t}{t_o} \qquad (t > t_e)$$

In this equation "C_c" and "C_α" are the well know coefficients of initial an secondary compression and "t_o" is the time for which initial compression is thought to be finished.

By comparison of both equations it can be postulated:

$$\alpha = \frac{C_a}{C_c} \cdot \frac{\log_{10} e}{\log_{10} 2} = 1.44 \frac{C_a}{C_c}$$

Investigation of the ratio C_a/C_c in a good number of materials (Mesri et al (1987), Soriano et al (1989)) indicates that, for most compacted clayey materials, it is close to 0.030. Hence, values of α are to be expected close to 0.043.

Calculations are carried out by computing instant deformations and simulating the increase of time by the method of the "initial stresses". The "free deformations" from time t_1 to time t_2, if no new loads are applied, are calculated by the equation:

$$\Delta\varepsilon(t_1, t_2) = \varepsilon(t_o) \cdot \alpha \cdot L_n \frac{t_2}{t_1}$$

DEFORMATION PARAMETERS

	K	n	Rf	ν	c (KN/m2)	Ø (°)	α (%)
1	1000	0,8	0,7	0,3	200	25	4
2	75	0,8	0,9	0,4	120	25	5
3	2000	0,6	0,7	0,3	40	35	2
4	2500	0,6	0,7	0,3	0	45	2

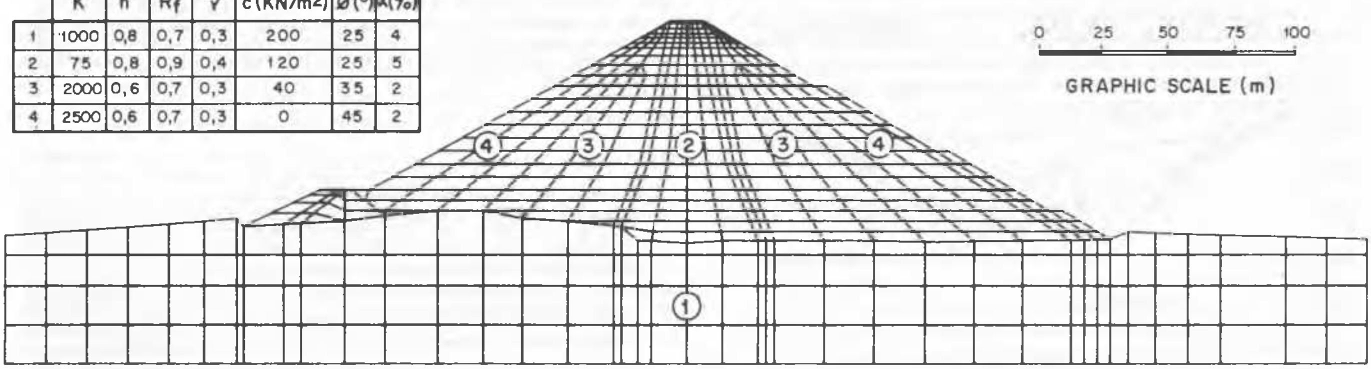

Fig. 6.- Finite elements mesh and deformation parameters

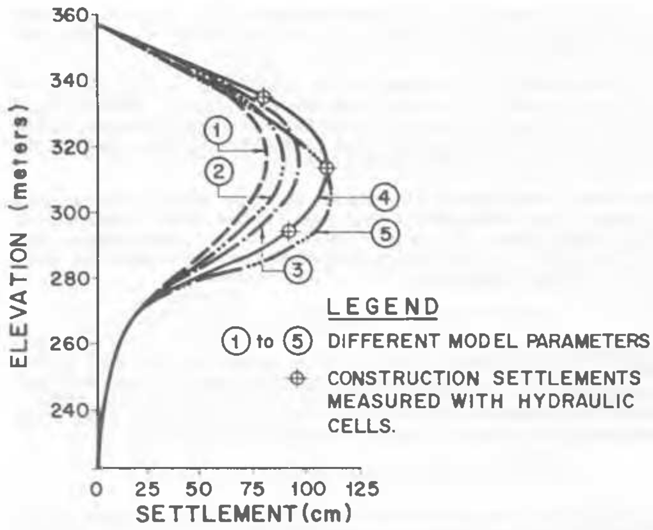

Fig. 7.- Back calculated settlements at dam axis

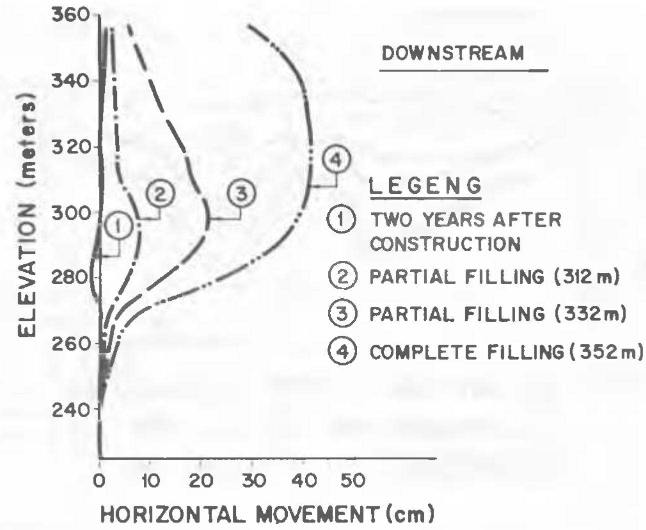

Fig. 9.- Back calculated horizontal displacements

The equivalent residual stresses are: $\Delta\sigma = D\ \Delta\varepsilon$
and the corresponding nodal forces: $\Delta F = E\ \Delta\sigma$

being **D** and **E** the corresponding finite elements matrices of the stress-strain law and element stress-nodal force correlation.

A second modification has been included to simulate the increase of compressibility of the granular materials upon saturation. The approach is explained elsewhere in detail (Soriano et al (1990)) and it is already a rather common practice.

RESULTS OF THE INTERPRETATION

Some results of the interpretation are given in Fig. 7 where settlements along the vertical axis of the core are given for different model parameters. Several trials have been made in order to match, as close as possible, the observed movements. The final set of deformation parameters which are thought to represent the actual behaviour are given in Fig.6.

This set of parameters hav_ been used to predict the behaviour of the dam during the first filling, which is starting these days (end of 1992). Main features of stresses and movements are indicated in Fig. 8 (total vertical stresses) and Fig. 9 (horizontal movements).

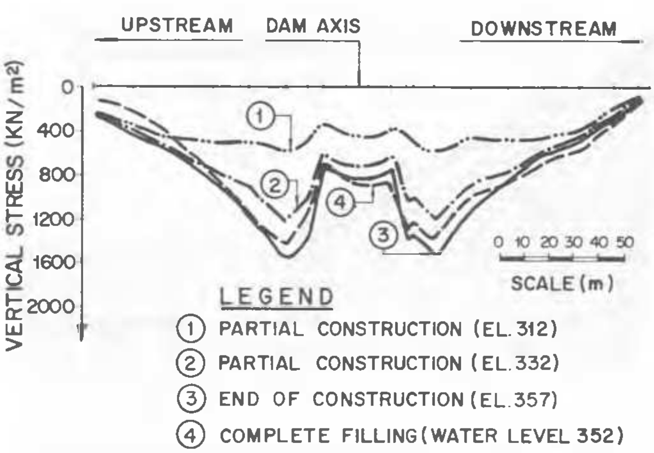

Fig. 8.- Back calculated vertical stresses (El. 290)

It is worthwhile to give some additional explanation about the deformation of the foundation. The observed settlement at the base of the dam has been quite large and the back calculation of the modulus of the foundation can not be done by a two-dimensional finite element model. For this particular purpose a simplified three dimensional calculation has been done to estimate that the average modulus of elasticity of the foundation, needed to explain the observed settlement, is about 700 MN/m^2.

CONCLUSION

The paper describes the main data related to the observation of movements and stresses within the core of Zahara dam as well as a simple numerical model that reproduces the main features of the observed behaviour.

The postulated model behaviour, once is calibrated to match the recorded data, is used to predict future behaviour when filling the reservoir, although it is known that the mechanism of deformation during filling of the reservoir can be quite different from that controlling construction deformations. Nevertheless, up to this point of the observation process, the main patterns of the recorded data can be explained by use of material parameters which are in reasonable agreement with those that were derived from the laboratory test which were performed at the design stage and during the construction of dam.

REFERENCES

Mesri, G. y Castro, A. (1987) *"C_α/C_c Concept and K_o during secondary compression"* Journal of Geotechnical Engineering. ASCE. Vol. 113 pp 230-247.

Soriano, A. (1989). *"Puesta en obra y compactación de rellenos en obras de infraestructura viaria"*. Simposio sobre el Agua y el Terreno en las Infraestructuras Viarias. Torremolinos, Spain. pp 129-150.

Soriano, A., Sánchez, F. and Serrano C. (1990). *"Simulation of wetting deformations of rock fills"*. Proc. Second European Specialty Conference on Numerical Methods in Geotechnical Engineering. Santander. pp 495-517.