Mularroya Dam: dam core analysis

Le barrage de Mularroya: analyse du noyau

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ABSTRACT: Mularroya Dam is located on the Grío River, in the province of Zaragoza, in northeastern Spain. It is an embankment dam with clay core (ECRD) with a maximum height of 82.6 meters and a crest length of 793.8 meters. The dam body has an approximate volume of 4.8 Mm^3 and is composed of nine different materials. Prior to the beginning of construction a study of available materials was carried out in the area with the result that the volume of clay available for the core was limited, so it was necessary to design a thin core to minimize the clay volume employed. In this article, we summarize the main characteristics of the dam and the materials used for its construction. Additionally this paper describes the procedure used to determine the core width (stress and strain analysis) and the results obtained.

RÉSUMÉ : Le barrage de Mularroya sur la rivière Grio est situé dans la province de Zaragoza, au nord-est de l'Espagne. Il s'agit d'un barrage en remblai, avec un noyau central en argile (ECRD). Il a une hauteur maximale de 82,6 mètres et un couronnement de 793.8m de longueur. Le corps du barrage est constitué de 9 matériaux différents et son volume approximatif est de 4.8 millions de mètres cubes. D'après les résultats d'une étude réalisée avant la construction, pour connaître les matériaux disponibles sur les lieux, le volume d'argiles disponibles pour le noyau était limité. De ce fait il a fallu projeter un noyau étroit pour minimiser son volume. Cet article présente, de façon résumée, les caractéristiques principales du barrage et des matériaux utilisés pour sa construction. La procédure appliquée pour le dimensionnement de la largeur du noyau (étude des tensions et des déformations) est aussi exposée, tout comme les résultats obtenus.

KEYWORDS: Dam, Core analysis

1 INTRODUCTION.

Mularroya Dam is located on the Grío River, in the municpality of La Almunia de Doña Godina, southwest of the Spanish city of Zaragoza.

Figure 1. Location of Mularroya Dam

Mularroya Dam is an embankment dam with a central clay core (ECRD). Table 1 shows the main features of this structure and the generated reservoir:

Table 1. Main data for Mularroya Dam

Height above lowest foundation	82.6 m
Crest length	793.8 m
Dam body volume	4.8 Mm^3
Surface area covered by the reservoir	463.1 Ha
Reservoir capacity (Normal Top Water Level)	103.3 Hm^3
Design Flood	98 m^3/s
Extreme Flood	472 m^3/s

The design of the dam dates back to 2004 and its construction began in 2008. Before the dam body construction started, environmental impact concerns gave rise to delays in obtaining permits to exploit the quarry selected at the design stage for procuring core materials. Consequently, it was decided to study alternate locations within the reservoir area that would supply materials meeting the design requirements for the impervious core. New field investigations were performed and a suitable material for the core found, but its volume turned out to be insufficient to construct the dam with the typical cross-section specified in the design. In order to reduce the amount of impervious material needed, a study was undertaken on a new typical cross-section with a core more slender than the one planned in the construction design. This resulted in revised typical cross-section, which is the geometrical basis on which the dam is presently being constructed.

2 DESCRIPTION OF THE TYPICAL CROSS-SECTION.

The typical cross-section for Mularroya Dam is trapezoidal, with an upper stream slope of 1.6(H):1(V) and 1.5(H):1(V) downstream. Its crest is 10.0 m wide and located at elevation 483.5 m. The outer slope in the upstream shoulder includes two 5m-wide berms, at elevations 423.5 m and 453.5 m. Figure 2 shows the typical cross-section as defined in the construction design.

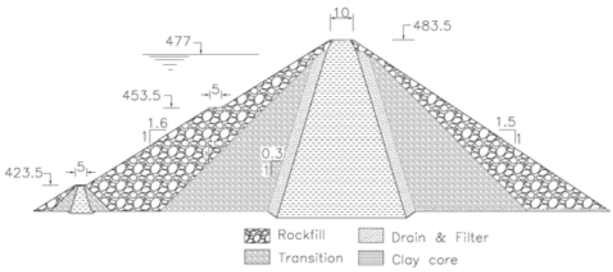

Figure 2. Typical cross-section of Mularroya Dam, as designed

The dam body is mostly made up of five materials: rockfill, transition zone, filter, drain and impervious clay core.

Table 2 presents the approximate volume of each material needed for the dam body:

Table 2. Approximate volume for each material

Volume	m^3
Rockfill	1,553,086
Transition	1,596,949
Filter and drain	432,017
Impervious core	1,208,243

The specification of technical requirements in the final dam design states some characteristic values that each of these materials must comply with. Specifically, the core material is required to show the main characteristics presented in Table 3:

Table 3. Requirements for the core material

Passing 2-inch (50 mm) sieve	100%
Passing #200 ASTM (75-µm) sieve	>35%
Liquid limit	$20 \leq W_L \leq 50$
Plasticity index	> 10
Soil organic matter	< 1%
Soluble salt content	< 1%
Maximum dry density (Standard Proctor)	≥1.65t/m³
Permeability coefficient	$\leq 10^{-6}$ cm/s
Internal fiction angle	≥ 25°
Pinhole test	ND

3 STUDIES FOR SELECTING CORE MATERIALS

The final dam design stipulates that impervious clays for the core shall be obtained from a quarry near Morata de Jalón (municipality close to the dam). However, when the works started, two issues arose:

a) The available clay volume from the planned borrow might not be enough for the dam construction and specially

b) Administrative problems related to environmental permits could imply a substantial delay affecting the works schedule.

These reasons led to the decision of studying alternate borrow pit locations within the reservoir area.

A clayey Keuper formation was found there and samples taken in order to perform laboratory tests. The results indicated that this material complied with the technical requirements for the core as specified in Table 3.

According to the field investigations, this material could be exploited in a volume that was around 40% of the clay needed to build the dam with the cross-section defined in the final design.

On the other hand, the reservoir area could also supply some Tertiary clayey gravels that comply with all requirements of Table 2, except for the maximum size (4% retained on the 2-inch sieve) and permeability ($k_{average}$ = $4 \cdot 10^{-6}$cm/s). An estimation was made that these gravels could be obtained in a volume representing 80% of all the clay needed for the construction of the dam with the typical cross-section specified in the final design.

Considering these facts, several alternatives were analyzed: a) waiting until authorities government granted a permit to use the originally planned quarry; b) studying how to construct the core with clayey marls from the reservoir area, provided that an additional source was found in order to guarantee the needed volume; or c) designing a zoned core that would consist of a central zone of Keuper clays, surrounded by clayey gravels.

In the end, the possibility of constructing a zoned core was chosen and analyzed. As the clay volume is limited, the inner core must be thin; and this fact together with the uneven deformability of the different materials in the core could entail a potential decrease in vertical stresses in the center of the core (silo effect). Such low vertical stresses might give rise to hydraulic fracturing in the core, which is why the decision was made to perform stress-strain analyses of the dam body.

4 ANALYSES OF THE TYPICAL CROSS-SECTION

The commercial finite-difference program FLAC v5 was used to compute stress-strain states in the dam body, of which 2-dimensional models and calculations were made.

Computations were performed on simplified cross-sections with four different widths for the inner clay core, as Figure 3 shows.

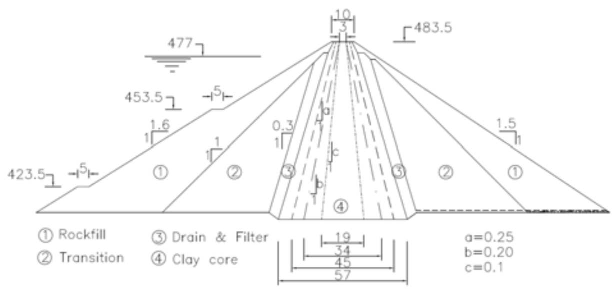

Figure 3. Geometry of cross-sections in 2D models

Table 4 provides the geometrical data for each case (I-IV) analyzed with FLAC:

Table 4. Geometric data for the simplified cross-sections analyzed in computations (dimensions of inner clay core)

	Core width at the crest (m)	Core Slopes
Case I	10	0.3(H):1(V)
Case II	6	0.25(H):1(V)
Case III	3	0.2(H):1(V)
Case IV	3	0.1(H):1(V)

In Cases II, III and IV, clayey marls from the reservoir area are considered to be placed between the Keuper clay core and the upstream and downstream filters (see Fig. 3).

Computations have been made considering two different material models for the soil: Mohr-Coulomb (Itasca Consulting Group Inc. 2008) and Duncan-Chang (Duncan, J.M. and Chang, C. 1970)

3 .1 *Analyses with the Mohr-Coulomb model*

The material parameters needed for using this soil model were obtained from laboratory tests performed on material samples taken from the field investigations and are shown in Table 5:

Table 5. Soil parameters for the Mohr-Coulomb model

	g (t/m³)	c (t/m²)	f (º)	E (kp/cm²)	n
Rockfill	2.1	0.1	45	1000	0.3
Transition	2.1	*0.1*	*35*	800	0.3
Filter+Drain	2.1	*0.1*	*35*	400	0.3
Outer core	2.1	*5*	*28*	400	0.3
Inner clay core	2.0	*5*	*25*	150	0.4

These parameters are assigned to the soils in the model and the 2D numerical models reproduce the geometries from Fig. 3. Then the construction of the dam body is simulated by sequentially enabling the zones corresponding to each dam lift, to eventually obtain stresses and strains at the end of construction for each case. This soil model has been used to analyze Cases I, II and III. By way of example, Figure 4 represents vertical stresses at the end of construction for Case III.

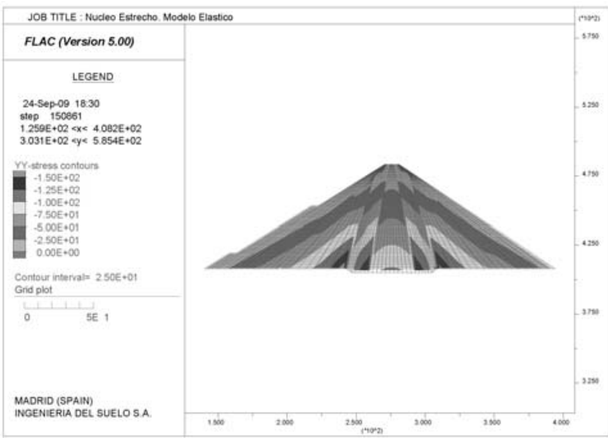

Figure 4. Vertical stresses at the end of construction (t/m²). Case III. Mohr-Coulomb

The silo effect that develops between materials with uneven deformability can be seen clearly in this figure, as vertical stresses in the inner core decrease due to arching.

Figure 5 shows vertical stresses at the base of the core for Cases I, II and III, when using the Mohr-Coulomb soil model.

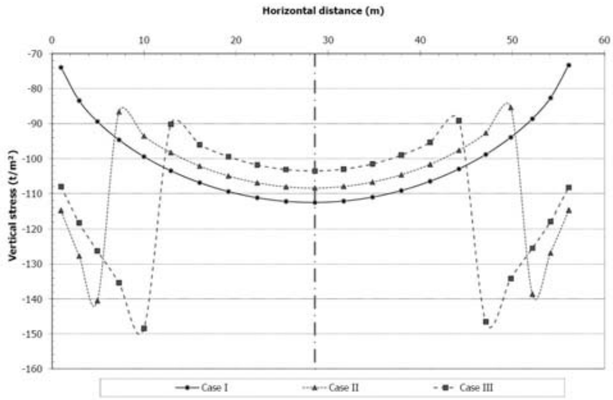

Figure 5. Vertical stresses at the core base. Mohr-Coulomb model

As the inner core becomes thinner, the vertical pressure that it transmits to the foundation decreases. The difference is not very significant, with values ranging from 11 kp/cm^2 for a base width of 57 m to 10 kp/cm^2 when the inner core is 34 m wide at its base.

3 .2 *Analyses with the Duncan-Chang model*

Using this soil model requires parameters that control deformability (k, n, Rf) in addition to the density and shear strength (friction and cohesion) parameters already presented in Table 5. For computations with this model, the Poisson ratio (ν) has been considered to remain constant and equal to the values from Table 5.

As with the previous model analyses, the parameters employed have been obtained from triaxial tests performed on samples taken at the borrow area. Table 6 shows the different values for each material.

Table 6. Parameters exclusive to the Duncan-Chang model

	k	n	Rf
Rockfill	1000	0.4	0.5
Transition	800	0.5	0.7
Filter + Drain	400	0.5	0.7
Outer core	400	0.6	0.8
Inner clay core	150/100	0.8	0.9

All four Cases (I-IV) from Table 4 have been analyzed using this soil model, simulating the dam construction by successive lifts and obtaining the stress and strain states at the end. In Cases I and II, a value of k = 150 has been considered for the inner core, whereas k = 100 has been used in Case IV for the same material. However, Case III has been analyzed with both k values (Case III, k=150; Case IIIb, k=100). By way of example, Figure 6 shows the vertical stresses at the end of construction for Case IV.

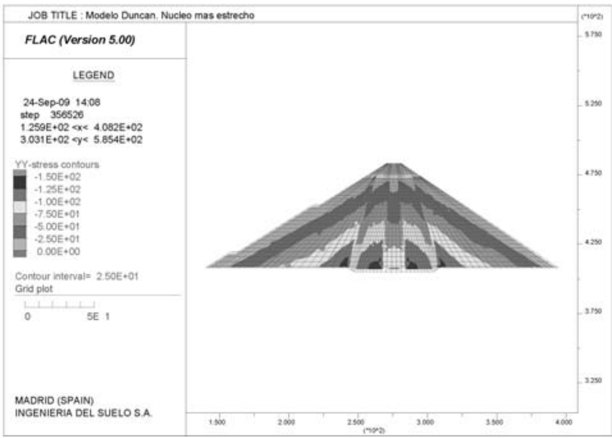

Figure 6. Vertical stresses at the end of construction (t/m²). Case IV. Duncan-Chang

Figure 7 represents the vertical stresses in the inner core at the foundation level, for each case analyzed.

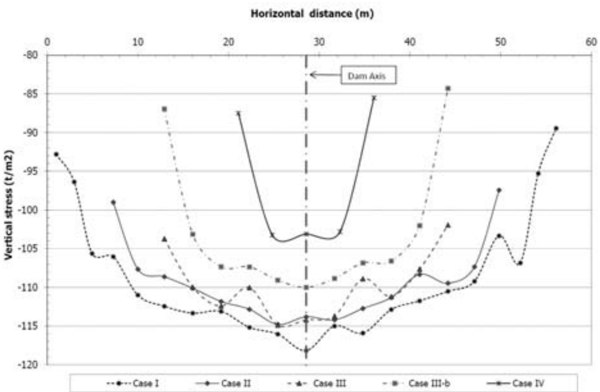

Figure 7. Vertical stresses at the inner core base. Duncan-Chang

It can be seen that the results for vertical stresses with this soil model are similar to those obtained using the Mohr-Coulomb model. So, the conclusion can be drawn that the choice between the two constitutive laws does not affect significantly the outcome in this specific problem.

Figure 8 summarizes the results of vertical stress at the core base for all cases analyzed.

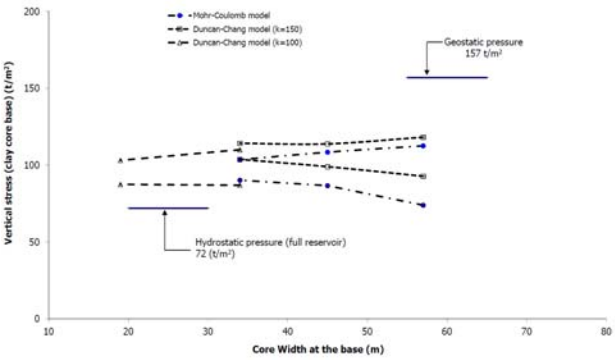

Figure 8. Summary of results

This figure represents all values of the vertical compressive stress on the core foundation, for each soil model and inner core geometry. It also represents reference lines for the hydrostatic pressure for full reservoir and the geostatic pressure corresponding to the assumed densities. As the pressure on the core base is not uniform along its width, Fig. 8 also shows the range of variation between the maximum (at the center) and the minimum (at the edges) values. It can be observed that this range of variation increases as the core becomes wider.

5 CONCLUSION

On the basis of the computations performed, it can be concluded that –at the end of construction- total stresses in the core are higher than the future hydraulic load due to a full reservoir. This hold true for each case analyzed. For the thinnest core of the study, the range of total vertical stress on the core base is higher than the full reservoir load for the Normal Top Water Level, namely from 120% to 130% of that value.

Water impounding will induce only slight changes in total vertical stresses, as such stresses are governed by the overall dam self-weight, which hardly varies in the core during impounding.

For these reasons and according to existing references on core widths (Soriano Peña and Sanchez Caro, 1997) the decision was made to build the dam with a thin core. The value adopted was the lower of those analyzed and the resulting typical cross-section is shown in Figure 9.

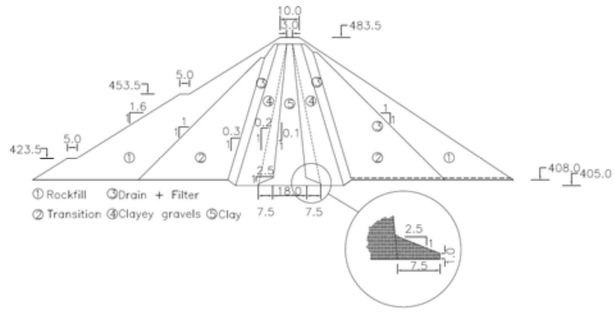

Figure 9. Revised typical cross-section, as adopted for construction

Mularroya Dam is currently under construction and the installed monitoring system provides data on stresses and strains measured in the dam body that do not deviate substantially from the predictions resulting from the numerical model analyses performed.

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