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SALAMONDE DAM COMPLEMENTARY SPILLWAY. DESIGN, HYDRAULIC MODEL AND ONGOING WORKS

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Abstract. *In order to meet the updated requirements of the Portuguese Dam Safety Legislation, Salamonde dam (double curvature arch dam, dated 1955) hydraulic-operational safety assessment has been developed by EDP, including the review of the design flood and the evaluation of safety devices capacity. Thereafter some corrective measures were implemented, including the construction of a new complementary spillway. The paper summarizes the main features of the new spillway design, developed by AQUALOGUS. The selected solution is a gate-controlled ogee crest, followed by a 120 m long curved tunnel scoped in the slope of the right bank designed for a discharge of 1233 m^3/s. The hydraulic performance of the designed structure was analysed by a study developed on a hydraulic physical model, built at LNEC on an approximate scale of 1/50. Part of the results and conclusions that allowed improving the design structure shapes is described. Some aspects of interesting and nowadays challenging ongoing works of this new spillway are presented.*

1 INTRODUCTION

Salamonde dam is located in the Cávado River, which flows through north-western Portugal, being part of the hydro electrical system of Cávado-Rabagão-Homem owned and operated by EDP.

It was built in 1955 and is a double-curvature arch dam with a maximum height of 75 m. For the normal water level (NWL), set at elevation (270.36), the flooded area is 237 ha and the maximum storage capacity is 65 hm^3.

The Salamonde scheme is formed by the dam and respective safety devices, the hydraulic circuit, the underground power plant and the substation. The original safety devices are a spillway and a bottom outlet. The spillway, located in the central part of the dam body, Figure 1, consists of four rectangular orifices equipped with Stoney gates. The maximum discharge capacity is about 1700 m^3/s for the maximum water level (MWL) defined in the original project (270.86).

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The energy dissipation of the discharged free jets is made by impact on the river bed and on the water pool created by a weir built downstream from the dam, Figure 2. This weir is a concrete arch structure with a maximum height of 17.50 m above the river bed, crossed by two equal rectangular outlets with 4.50 m of length and 3.00 m height, equipped with manual flat gates.

Figure 1: Existing spillway and bottom outlet

Figure 2: Weir downstream of the dam

In order to meet the updated requirements of the Portuguese Dam Safety Legislation, RSB (2007)[1], the hydraulic-operational safety assessment of Salamonde dam has been developed by EDP since 2005. The first phase of this assessment included a review of flood studies (in order to validate the previous design flood or to establish a new one), a suitability analysis of discharge devices and the outline of corrective structural measures, as detailed in Oliveira *et al*. (2012)[2]. The second phase consisted in the design and implementation of corrective measures, which included the construction of a new complementary spillway, as described in Oliveira and Dias da Silva (2012)[3]. This paper summarizes the main conclusions of the aforementioned dam safety assessment and presents the main features of the new spillway design, developed by AQUALOGUS (2010)[4].

The hydraulic performance of the designed structure was analyzed by a study on a physical model, carried out by LNEC. This model intended the evaluation of the flow conditions under the operation of the new spillway or both spillways simultaneously and allowed testing alternative shapes to improve the hydraulic performance of the spillway, Couto *et al*. (2014)[5]. The studies in the physical model are mentioned in the paper.

Some aspects of the ongoing works are also referred, namely some of the constructive requirements that became design constraints.

2 THE COMPLEMENTARY SPILLWAY

Beyond the regulatory terms, in the choice and design of the spillway the following essential aspects were taken into account:

- The location of the existing discharge devices and the existing and new hydraulic circuits;
- The topographical and geological characteristics of the downstream valley;
- The need to minimize any interference with the dam’s body (thin arch) without the excessive lengthening of the new spillway structure;
- The maintenance of the traffic between the two river banks during construction period;
- The location of the right bank of the Cávado River inside the Peneda-Gerês National Park.

The adopted solution for the spillway was designed near the right dam abutment and is formed by a gate-controlled ogee crest, followed by a 120 m long tunnel, ending in a ski jump structure.

The excavation of a horizontal approach platform at elevation (253.00), immediately upstream from the control structure, was planned. The ogee crest, at the elevation (258.00), is divided into two equal 6.5 m width bays, fitted with radial gates. The flow separation at the crest is performed using a pier with hydrodynamic shape in plan.

The control structure is followed by a lined concrete tunnel with an approximate length of 120 m and variable cross-section, extending from the elevation (250.00) to the elevation (226.00). The longitudinal profile of the tunnel has two straight stretches, having slopes of 80% and 10%, concordant by a 50 m radius circular curve, Figure 3a. In the horizontal plan, the tunnel has two straight alignments, concordant with a 80 m radius circular curve, Figure 3b. In the initial part of the tunnel, until the end of the curve in plan view, the tunnel section is divided by a septum wall in the continuity of the pier that separates the two spans of the control structure. In this stretch, the cross-section is convergent between approximately 2 x 60 m^2 to a constant section of about 2 x 36 m^2. Downstream from the septum wall the tunnel has a length of about 35 m and is characterized by an area of approximately 80 m^2. For the design discharge, the maximum water level inside the tunnel does not exceed 70% of the cross-section height.

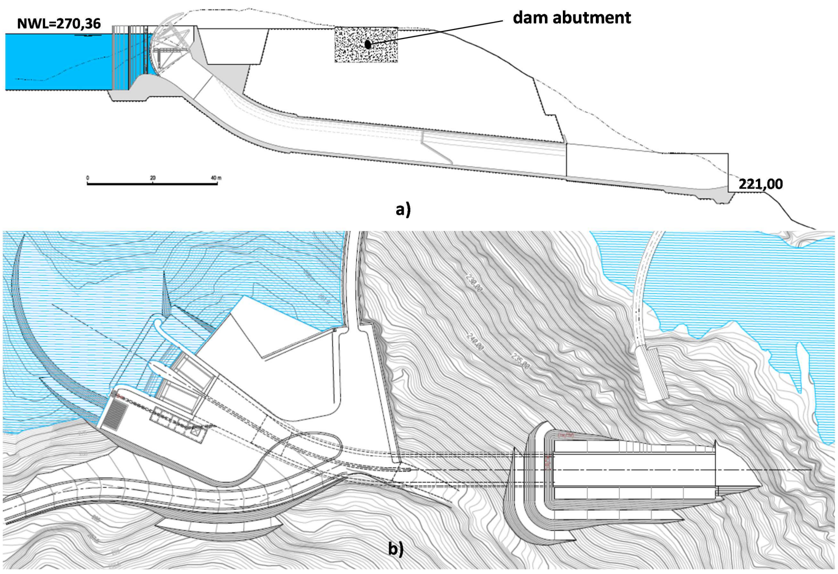

Figure 3: Complementary spillway: a) longitudinal profile view; b) plan view

The downstream part of the spillway, with a length of 50 m, is an open channel, ending by a ski jump structure. According to the design calculations, the impact zone of the discharged jets would be located about 60 m downstream of the terminal section of the spillway.

3 HYDRAULIC DESIGN

The complementary spillway was designed for a flood discharge with a 1000 years return period and the condition of not exceeding the reservoir maximum water level (MWL = 270.86) considering its simultaneous operation with the existing spillway. It was checked for a 5000 years return period flood and for a 100 years flood in the case of one gate being out of service.

Flood routing in the reservoir was simulated using the reservoir storage curve, the spillways rating curves and the flood hydrographs determined in the flood revision (Figure 4). The complementary spillway design discharge was 1233 m^3/s (T = 1000 years), and the checking discharge was 1396 m^3/s (T = 5000 years).

For the 1000 years return period flood, the total flow discharged through both spillways (existing and complementary) would be 2828 m^3/s for the new maximum water level at elevation (270.64).

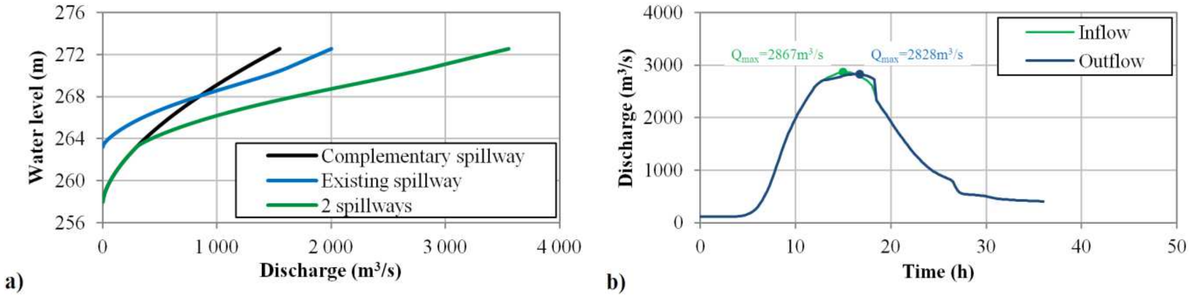

Figure 4: a) Rating curves. b) Design flood inflow and outflow hydrographs (T = 1000 years)

The flow characteristics in the tunnel were analyzed using a 1D model developed to compute gradually varied flow profiles in closed sections. The friction head losses were computed using Manning's coefficients n = 0.0133 and 0.0125. According to the Karman-Prandtl equation, in Quintela (1981)[6], for the tunnel hydraulic diameter (between 6 and 9 m), these values are equivalent to an absolute roughness of 0.2 to 0.4 mm (rough concrete). The water surface super-elevation on the plain curve was approximately determined using the recommended equation [1003], Woodward and Posey (1941)[7] for rectangular sections. A 1.8 m maximum water surface super-elevation was obtained. The air entrainment effect on the water surface was not considered due to the short length of the tunnel. The water surface profile obtained for the design discharge is shown in Figure 5. The maximum water depth in the tunnel was about 69% of the cross-section height for the design discharge (1233 m^3/s) and 77% of the cross-section height for the check discharge (1396 m^3/s). Therefore the tunnel discharge capacity was considered adequate.

Due to the rock quality of the river bed downstream of the dam (Figure 2), a ski jump designed to direct the jet to the river bed was selected. The final geometry of the ski jump was analysed in the physical model.

Considering the possibility of isolated operation of the complementary spillway, the backwater effect up to the existing downstream weir was also studied. With the two weir outlets closed, a water depth of 7 m was obtained, which is about 40% of the weir height above the river bed. In consequence, it was recommended that the spillways operation should start with the existing spillway and that the weir outlets should always be open to prevent the water level upstream of the weir from becoming lower than the downstream level, endangering the weir stability. However, a study of this effect was also carried out in the physical model.

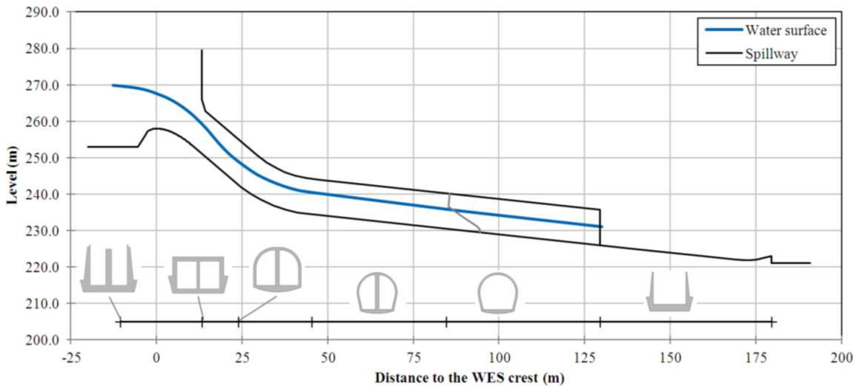

Figure 5: Water surface profile for design discharge (Q = 1233 m^3/s, T = 1000 years) and n = 0.0133

4 PHYSICAL MODEL HYDRAULIC STUDY

4.1 Model description

A non-distorted physical model with a 1/52.08 scale was built at LNEC. The dimensions of the model were 2.40 m height, 6.70 m width and a length of 13.30 m. This model, in addition to the dam and the original spillway, reproduced the complementary spillway and significant reaches of the reservoir and of the downstream valley, including the river bed and the respective banks, to ensure an accurate reproduction of the flow conditions in the river.

Figure 6 includes the upstream view of the model, with part of the reservoir, the upstream face of the dam and the control structures of both spillways. Figure 7 includes a downstream view of the complementary spillway and the downstream weir.

Figure 6: Hydraulic model. Upstream view

Figure 7: Hydraulic model. Downstream view

Both spillway structures were moulded in cement and sand mortar. The border areas in contact with the flow were coated with cement paste, which simulates the roughness of the concrete surfaces of the prototype. The top of the channel was built with transparent acrylic material ("Perspex") to enable the visualization of the flow and measuring water levels achieved therein. Alternative shapes were moulded using gypsum and wood pieces. After the trials, the proposed shapes were reconstructed with cement mortar in order to withstand more prolonged use in tests without deterioration. In the downstream reach of the model, where scour of the river bed can occur, mobile material was used. The mean diameter of the gravel, (D_{50} = 22 mm), simulates approximately the expected probable fracturing of existing rocks.

The water intake is integrated in the experimental pavilion network, where a 500 mm diameter pipe has regulation valves and electromagnetic flowmeters. The water flows to the model through a tranquilization system. The water levels upstream and downstream from the dam were controlled by two hydrometers, one upstream, inside the reservoir and another downstream, close to the boundary of the model. Water levels were measured in the reservoir model at a section located 100 m upstream from the spillway crest. For the tested flow rates, the water levels downstream of the dam were reproduced in the model using a plane gate located in the downstream area of the model. The reproduced flow levels were based on the natural rating curves.

4.2 Objectives and tested conditions

The main objectives of this study were the analysis of the complementary spillway design shapes, the hydraulic performance and definition of alternative shapes, if necessary, in order to have more favourable technical solutions.

Hence, the use of the physical model included: (a) analysis of the general flow conditions in the approach zone, on the control structure, inside the tunnel, over the ski jump structure and in the outlet zone; (b) determination of the spillway stage-discharge curves for different openings of the gates; (c) analysis of the simultaneous operation of the two spillways, in particular regarding the approach conditions and possible influence of the new spillway release with the existing spillway releases damped by the downstream weir; (d) measurements of the water levels around the weir.

To achieve these objectives, tests with different discharge were performed, namely design and check flood discharge in complementary spillway (1233 and 1396 m^3/s) or simultaneous operation of both spillways (2828 and 3205 m^3/s). Tests were also run with 616 m^3/s over the complementary spillway through one of the gates, simulating the case of one gate being out of service.

4.3 Preliminary design structures performance

The first tests on the hydraulic model, with the preliminary design shapes, allowed the conclusion that the flow conditions were generally acceptable. Nevertheless, some efforts were done to improve these conditions wherever possible.

The results of the test with the design discharge near the control structure are represented in Figure 8.

The results of the same test, concerning the design discharge release over the ski jump structure, are represented in Figure 9. This structure created a very compact jet that would cause excessive erosion on the river bed at the impact zone. An evaluation of the erosion caused by the jet in the river bed was performed through a 14-hour long test flood in the prototype, situation in which a maximum scour depth of -17 m was measured.

Figure 8: Control structure with design shapes
Design discharge (2828 m^3/s)

Figure 9: Ski jump structure with design shapes
Design discharge (2828 m^3/s)

4.4 Modifications to the spillway design shapes

The elimination of the flow contractions initially observed near the control structure was accomplished by testing four modifications. Figure 10 represents some of the alternative shapes tested, namely the extension of the left end wing wall and the upstream extension of the central pier.

Both bank erosion and riverbed scour could be minimized through series of testing several alternative shapes for the ski jump structure that orientated the jet to a more convenient impact zone. In Figure 11, one example of these tests is presented, where improvement in the jet dispersion and impact zone can be visualized.

Figure 10: Control structure with one of the alternative shapes. Design discharge (2828 m^3/s)

Figure 11: Ski jump structure with one of the alternative shapes. Design discharge (2828 m^3/s)

4.5 Proposed structures performance

The main modifications in the control structure within the final proposed shape include: extension of the left wing wall; extension of the pier; redefinition of the hydraulic shape of the right wing wall and inclusion of the cofferdam abutment in the left wing wall.

Consideration of part of the cofferdam permanently integrated in the structure was decided upon economic and constructive constraints, but the tests showed that this solution could turn out an improvement in flow conditions. Some tests in the model also considered the final shape of the bank in the approach zone, which resulted in a reduction of excavation volumes, with all its advantages in terms of volume of works and costs.

The proposed overall shapes allow fewer contractions. Additionally, it was concluded that no significant changes in the stage-discharge curves could be measured. In Figure 12 the results of the test with the design discharge is included, where a significant improvement in terms of visualized contraction from Figure 8 can be highlighted.

Regarding the flow inside the tunnel, tests revealed that the free surface never reaches the design criteria of 70% of free height of the tunnel.

The proposed changes for the ski jump structure include: redefinition of the shape of the left wall; reduction of the right wall height and redefinition of its shape and reduction of the lip width.

The jet obtained for the design discharge is illustrated in Figure 13. This causes shallower holes and the effect of its dispersion is evident. The jet impact, with the design discharge, extends from 49 to 91 m.

After defining spillway shapes, tests were performed to measure the water levels around the existing weir faces, within a safety analysis of this structure when both spillways become operational. For an opening of 0.5 m in two of the existing spillway gates, the water levels upstream from the weir were higher than the downstream levels, whatever the discharge flow from the complementary spillway was. Thus, the stability of the weir is guaranteed with the opening of 0.50 m in two of the four gates in the existing spillway.

Figure 12: Control structure with proposed shapes. Design discharge (2828 m^3/s)

Figure 13: Ski jump structure with proposed shapes. Design discharge (2828 m^3/s)

Several tests with combinations of different gate openings in the new spillway were performed. It was concluded that the successive symmetric opening of both gates is more favourable. In case the operation is performed through levels from alternate openings of 0.5 m in each gate, the opening of the right gate should be higher. According to the tests, if one of the gates is eventually not functioning, no critical situation was detected and the operation of the working gate can proceed until the necessary opening.

Additionally, flow depths were measured inside the tunnel in ten cross-sections for five different gate openings. The results were used to calibrate a numerical model, namely the commercial computational fluid dynamics “FLOW-3D”, used to simulate the flow characteristics along the new spillway, as described in Silva *et al*. (2014)[8]. Figure 14 and 15 illustrate the flow inside the tunnel in the physical model and the results of the numerical model simulation, which gave underestimated flow depths in some regions with reduced flow thickness.

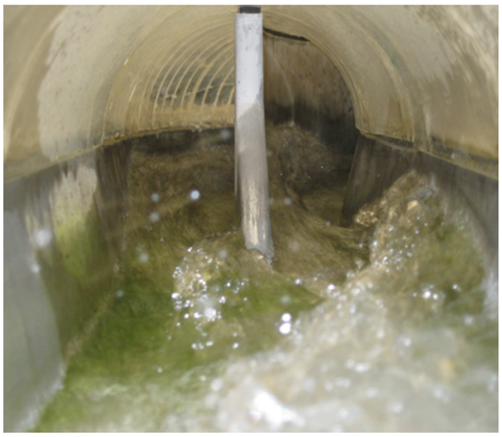

Figure 14: Physical model flow inside the tunnel

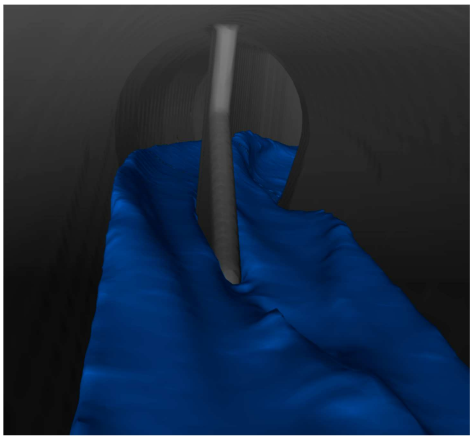

Figure 15: Computational model flow inside the tunnel (in Silva et al. (2014)[8])

5 CONSTRUCTION PHASE

The construction of the new Salamonde dam spillway started in March 2011 and will be completely finished in October 2014. The spillway construction was included in the contract for the Salamonde Repowering Project (Salamonde II), whose hydraulic circuit intake and powerhouse are located in the left bank, near the similar structures for the existing hydraulic circuit. Figure 16 illustrates an overview during construction.

Figure 16: Overview during construction (February 2013)

The inclusion of the works of the spillway in the Salamonde II contract would allow a significant reduction of the disturbances/impacts associated with the construction. In fact:

- By matching the needed lowering periods for both projects, it would be possible to avoid any additional lowering in the Salamonde reservoir, as well as all the disturbances associated with that type of action;
- Both the debris deposit and industrial facilities are common for the two projects and so, better synergies and construction facilities and equipment optimization can be achieved.

The construction of a cofferdam (Figure 17) was decided to ensure the needed protection during construction of the spillway control structure.

The concrete cofferdam is a circular arch, with its crest at (271,50) and a maximum height of 23,5 m. Its construction occurred between May 1 and July 15, 2011, taking advantage of the anticipated lowering of the Salamonde reservoir, due to the construction of the cofferdams for the Repowering Projects of both Salamonde II and Venda Nova III. The demolition of the cofferdam occurred in August 2014, during another lowering of the reservoir scheduled due to the same repowering Projects.

In Figure 18 it is possible to see the ski jump structure already built.

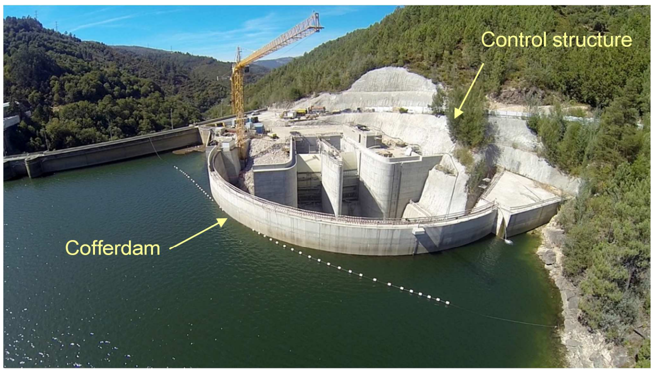

Figure 17: Control structure (September 2013)

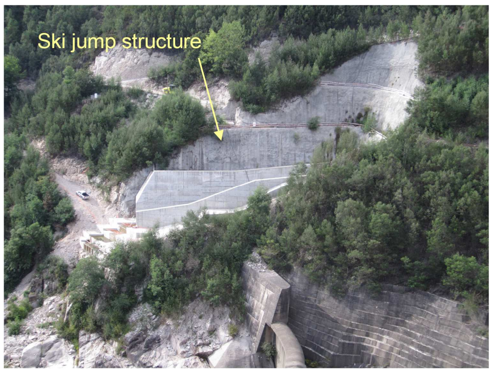

Figure 18: Ski jump structure (August 2014)

6 CONCLUSIONS

After the safety analysis carried out by EDP, it was concluded that an additional discharge structure would be necessary, namely the complementary spillway for Salamonde dam. This new structure, designed by AQUALOGUS, is a gated spillway, controlled by an ogee crest, followed by a tunnel with rather complex geometry, designed for free surface flow, and a ski jump which directs the jet into the river bed.

The hydraulic performance of this new spillway was studied in a physical model built at LNEC with a 1/52 scale (Figure 6 and Figure 7).

Based on the model testing facilities, some modifications from the preliminary design were considered, namely in the control structure wing walls shape, on the right bank shape and volume of digging upstream the intake structure, incorporation of part of the cofferdam and central pier extension to upstream. This model allowed particularly testing several alternative shapes or the detailed definition of the ski jump structure, minimizing the impact on the downstream banks and the scour on the riverbed. The study included a safety analysis of the dam downstream weir when the operation of both spillways will occur. Additionally, several scenarios were tested with alternate and different gate openings of the complementary spillway.

Some measurements were undertaken specifically to calibrate a CFD numerical model, used to simulate the flow along the complementary spillway (see ex. Figure 14 and Figure 15). Although it could be considered that the numerical model accurately simulates the flow features along such a spillway with rather complex geometry, it became proved that the use of a physical model still reveals to be an extremely powerful tool.

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