New spillway for La Laye dam, France

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The flood discharge works at La Laye dam in France are to be upgraded, following a recent re-assessment of its design flood. This paper describes the innovative design and model studies for the new emergency spillway, which is to be implemented in 2008.

Located in the South of France near Forcalquier, the 30 m-high La Laye dam is an earthfill structure built in 1962-1964 for irrigation purposes. Associated hydraulic structures include a shaft spillway controlled by a cylindrical gate and a bottom outlet.

The maximum discharge capacity is 380 m^3/s.

As is the case for many dams in France, particularly in the south of the country, the flood estimate was recently reviewed and it was concluded that the design flood should be increased considerably above the original estimate. As a result, an additional spillway is required to bring the project to the required safety level.

Considering the project's features (flood magnitude, dam deformations, geological conditions, narrowness of the valley), the designers proposed an original design for the emergency spillway.

The project was approved by the Comité Technique Permanent des Barrages (French Public Authority in charge of dam safety). To complete the final design, a hydraulic model test was recently carried out to analyse the hydraulic behaviour of the spillway with a focus on the dissipation issues.

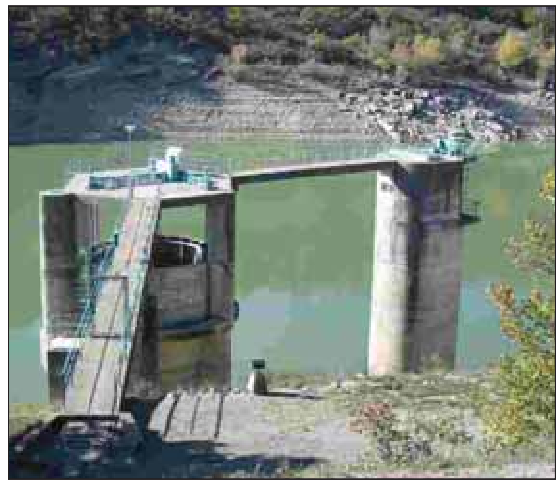
1. La Laye dam

Controlling a 3.5×10^6 m^3 reservoir, La Laye earthfill dam was mainly built for irrigation, as well as some water supply to the Forcalquier urban area and industries.

The dam was completed 1964, and reservoir impounding took place in 1965. The initial NWL was raised by 3 m as a result of the installation of a cylindrical gate on the circular sill of the spillway.

The 30 m-high dam is an earthfill structure with a thick core, and is shown in the photograph above.

Downstream view of La Laye dam in France.



Intake/bottom outlet tower and the shaft spillway.

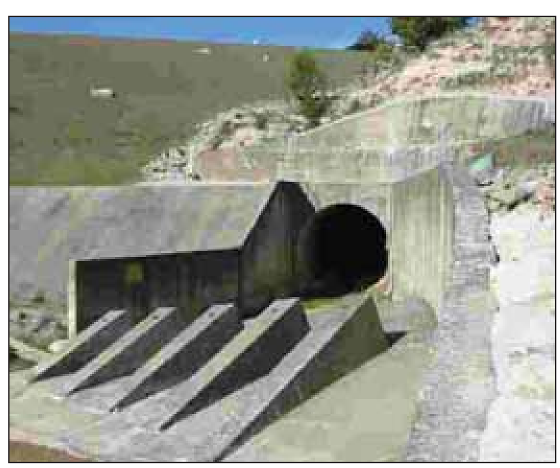
The ancillary works consists of:

- an upstream intake tower;
- a bottom outlet at the toe of the intake tower;
- an independent shaft spillway; and,
- a single gallery collecting the flows from the bottom outlet and spillway.

The combined maximum capacity of the spillway and bottom outlet is close to 380 m^3/s; any elevation of the reservoir level will have only a very marginal effect on this figure.

2. Hydrological assessment

A detailed hydrological assessment was carried out and this led to the selection of the following flood data:



The spillway outlet.

- 1150 m^3/s (Gradex method) for the design flood (T = 10 000 years), to be released below the MWL; and,
- 1400 m^3/s (Gradex method) for the safety flood (maximum flood figure given by several approaches) to be released below the dam crest level.

3. Selection of the spillway type

The discrepancy between the existing spillway capacity and the calculated flood figures underlines the need to design and construct a new spillway.

The design criteria for the new spillway can be summarized as follows:

1. The new spillway will be an emergency structure, operating very seldom and for short durations.
2. The flow figures are very large (820 m^3/s for a reservoir at MWL, 1600 m^3/s for a reservoir at crest level).
3. The dam has not shown any displacement, either vertically or horizontally for many years.
4. The river valley is narrow, and both banks have steep slopes.
5. The geological conditions (with a substratum strike oriented slightly downstream) have to be taken into account, as the dam stability could be threatened by any excavation or scour at the toe of the dam.

Several alternatives have been discarded :

- increasing the existing spillway capacity;
- heightening of the dam;
- provision of a new spillway, excavated on a bank; and,
- implementing gated spillways.

The existing shaft spillway is saturated for a discharge close to 400 m^3/s. The required capacity, which corresponds to four times the present one cannot be obtained.

Dam heightening could have been an attractive solution if the reservoir area and volume were large enough. The part of the design flood which is not released by the existing spillway represents a volume of 10×10^6 m^3, to be compared with the volume stored in the reservoir for each metre above NWL, about 300 000 m^3. A comparison of both figures shows that heightening the dam could not contribute to solving the problem.

Neither bank offers an option for construction of a new spillway. The left bank is already occupied by the existing spillway, while the right bank has sub-vertical slopes just upstream of the dam. The magnitude of the design discharge is such that a cut with a 15/20 m depth and 20 m width should be created in the dam's longitudinal section.

The option of equipping the emergency spillway with flap gates (three flap gates 20 m wide × 3 m high) was also discarded for the following reasons:

- very large flows would be released in the event of any operational problem;
- the existing spillway is already gated and the emergency spillway was considered to be operating in an autonomous way;
- the emergency spillway will operate very rarely, and operation of the gates would result in large floods; and,
- the cost savings would not be substantial.

All previous considerations led to the proposal to locate the emergency spillway on the dam, and to install fusegates to obtain the required spilling area. The section along the spillway axis is shown in Fig. 1.

The spillway will include:

- a horizontal platform supporting the fusegates (the first fusegate will collapse for a flood with a return period of between 200 and 500 years);
- a concrete chute, founded on the dam fill; and,
- a downstream dissipating structure with a ski-jump and a reinforced concrete apron founded on sound bedrock.

Before developing the final design, a hydraulic model was built to determine the design dimensions with a focus on the downstream dissipation structure.

4. Hydraulic model

4.1 Description

The hydraulic model extends from 130 m upstream of the spillway sill to 150 m downstream of the ski-jump. The total length of the model is about 350 m, including the spillway. The length was chosen in accordance with the topographical data, and the need for accurate representation of the hydraulic phenomenon.

The flows studied are mainly free surface flows, meaning that a Froude similarity must be used to describe the hydraulic behaviour of the spillway. This similarity law implies that the Froude number must be the same in both the prototype and in the model. This dimensionless number compares the relative values of inertia forces and gravity forces in the flow, and is expressed by the following relationship: ,

$$Fr = V / \sqrt{gH}$$

where V is the velocity, g the gravity acceleration and H the head. As a result, similarity rules are as follows:

- geometrical scale: $1/e$;
- velocity scale: $1/e^{0.5}$; and,
- discharge scale: $1/e^{2.5}$.

As the maximum discharge on the platform is limited by the water supply network, the scale is chosen according to this limit. The results are:

- **geometrical scale 1/35**

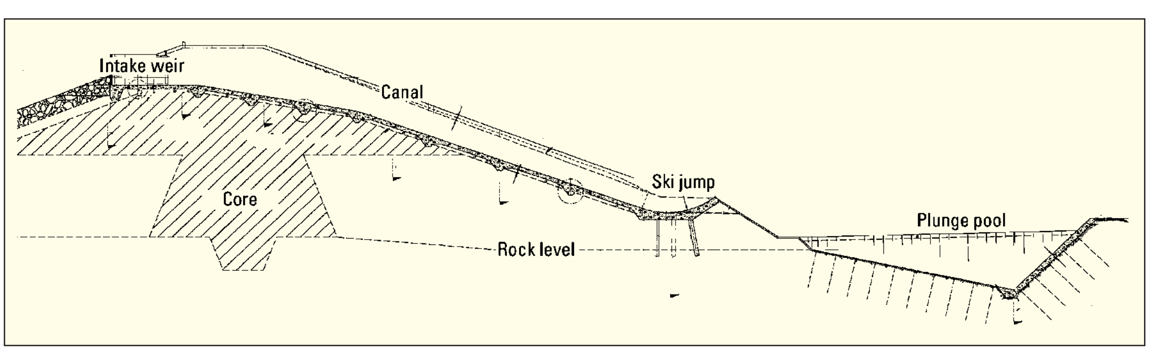


Fig. 1. Longitudinal section along the new spillway.

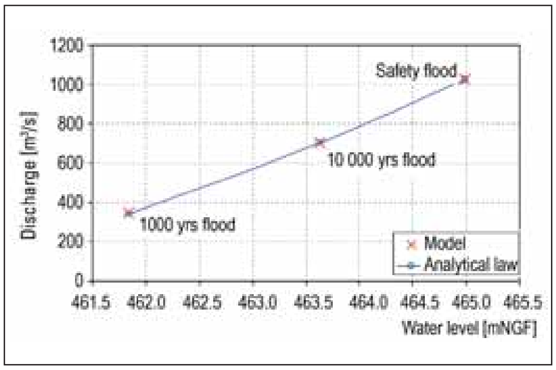


Fig. 2. Measured and analytical sill law.

- velocity scale 1/5.9
- discharge scale 1/7247

The model includes six main elements:

- an upstream stilling basin receiving the water flowing from a 200 mm pipe;
- a basin of the width of the dam and including the upstream abutment and the intake of the existing spillway;
- the new spillway, including the sill, the chute and the ski jump;
- a pipe with a regulating gate, supplying the existing stilling basin; and,
- a basin following the natural topography downstream of the dam, including both existing and new stilling basin.

The discharge is measured with a 200 mm electromagnetic flow meter on the supply pipe. This equipment has an uncertainty value of less than 0.3 per cent. Water levels are measured with a point gauge with a precision of 1/10 mm. Flow velocities are measured with an electromagnetic two dimensional velocity meter connected to a computer via a data logger.

4.2 Boundary conditions

The upstream discharges are given by three different floods:

- the 1000 years flood peak discharge: 730 m^3/s
- the 10 000 years flood peak discharge: 1080 m^3/s;

Inlet of the sill before and after the flow optimisation (1000 year flood).

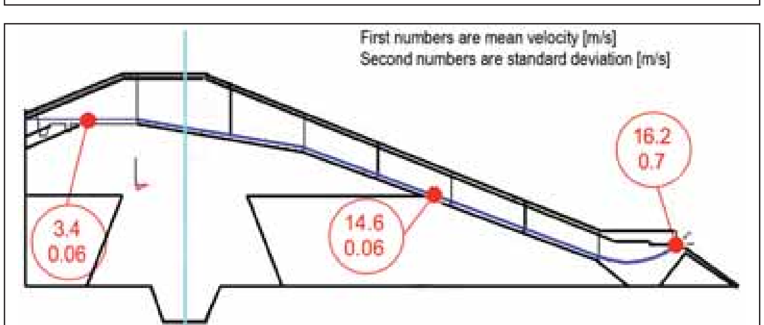


Fig. 3. Velocities at three points on the spillway for the 1000 year flood.

Model of the downstream area for the 1000 year flood.

and,

- the safety flood peak discharge: 1400 m^3/s

For each discharge, 380 m^3/s will be evacuated by the existing shaft spillway, the remaining discharge flowing through the new structure.

4.3 Main results

4.3.1 Flow description

The flow through the sill and in the steep slope canal is generally regular, except next to the wing wall where the water is not very well guided at the inlet. These perturbations induce small stationary waves which are reflected on to the walls but which never produce overflows outside the canal.

Differences between the computed and measured water heads in the steep canal are always less than 50 cm, and no overflows are observed in this part of the spillway.

Flow detachment at the outlet of the ski-jump appears for a 100 m^3/s discharge and the impact in the stilling basin occurs 40 m farther.

Despite a relatively low volume, the stilling basin is efficient as a result of the large water cushion (4 m) present over the basin. The total volume for energy dissipation, including the basin and the cushion, is thus about 12 500 m^3, giving an efficiency of 6 Cv/m^3 for a 1000 years flood and 15 Cv/m^3 for a 10 000 year flood, commonly accepted values for such a structure.

Flows from the existing spillway and the new spillway show little interaction between them.

4.3.2 Sill law

A water level–discharge law was estimated from the measurements, and an adjustment was made to the discharge coefficient C_d from the following analytical law, giving the discharge as a function of the weir length L and the water head above the crest H:

$$Q = Cd \cdot L \cdot \sqrt{2g}H^{3/2}$$

The computed value for the discharge coefficient is: C_d=0.39

4.3.3 Flow optimization

As mentioned above, significant swirl occurs adjacent to the wing wall at the inlet of the sill. It is possible to suppress this by changing the design of the inlet, to enhance the guidance of the flow path. The optimal inlet is obtained by an elliptical design. This solution was tested on the model, which demonstrated that it is possible to suppress most of the swirl and to increase the discharge coefficient to 0.4 (+2.6 per cent) for the 1000 year flood.

4.3.4 Velocity measurement

The velocity was measured at five points along the spillway from the flat upstream platform to the end of the ski-jump, and at seven points around the stilling basin. For each point, a series of more than 30 measurements was taken. The result was presented as a mean and standard deviation of measured sample. Fig. 3 shows the main results for the 1000 year flood.

4.4 Contribution of the hydraulic model to the design

The hydraulic model study has shown that the new spillway will operate well.

In particular:

• The discharge coefficient has been measured, and the inlet shape was optimized.
• Flows in the chute are satisfactory, and stationary waves observed have never induced overtopping.
• The ski-jump works well, and flows are regularly distributed along its width.
• The stilling basin is efficient, and the exiting flows are stabilized well. The dissipation rate varies from 6 to 15 Cv/m^3, depending on the discharge.
• Velocities near to the toe of the earthfill dam are about 3 m/s.

5. Next stages

The final design of the new spillway has recently been completed. Tender documents for construction are to be issued at the beginning of 2008, so that construction can begin during the second half of 2008. ◊

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