DESIGN OF THE NEW SPILLWAY FOR THE PANAMA CANAL

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ABSTRACT

When the current spillway of the Gatun Lake of the Panama Canal was designed in the early 1900's, very little information was available concerning the hydrology and hydraulics of the Chagres river. The current spillway capacity was designed based on discharges calculated from historical flood elevations along the river and a maximum operational lake level. In following years, the maximum operating level was raised to satisfy additional water demands and accommodate increasing vessel drafts.

As part of the preparation of an overall Master Plan for expansion of the Panama Canal, the Panama Canal Authority (ACP) studied a number of options to upgrade the Canal infrastructure and existing infrastructure limitations, including the current capacity of Gatun Spillway. After more than one hundred years of operation, a significant amount of hydrologic data has been collected. Different studies have found that the current spillway capacity is insufficient to safely handle maximum floods.

The international standards, for dams such as Gatun, indicate that the spillway must be designed to meet the Probable Maximum Flood (PMF). Therefore, additional spillway capacity was needed in order to meet this criterion and to protect the Lake and the viability of the Canal system itself. This paper analyzes the main hydraulic aspects of the design of the new spillway, including the comparison of the results of a three dimensional numerical modeling and the physical model built for the project.

Keywords: *spillway, capacity, flood, physical and numerical modeling.*

1. BACKGROUND

The Panama Canal Authority (ACP) determined from several studies, that the capacity for flood evacuation of Gatun Lake needed to be increased. Since 1945, different initiatives had taken place to design a new spillway. The final area for the new spillway was decided between the new and old set of locks. In August 2015, INGETEC was awarded the detailed design of new spillway for the Gatun Lake. The design included the analysis of alternatives within the defined area, a thorough geotechnical investigation of the area between old locks and new locks in the Atlantic side of the Canal, and the physical and numerical (CFD) modeling with the final objective to develop the detailed design including construction drawings and specifications for construction.

2. THE NEED FOR A NEW SPILLWAY

2.1 MAXIMUM PROBABLE FLOOD

In 1979, the US Corps of Engineers studied the maximum probable flood (PMF) in the Gatun Lake basin. The analyses resulted in a hydrograph with a maximum inflow of 23,985 m^3/s and a hydrograph duration of 4 days. In 2005 another study showed, by flood transit through the lake, that the existing outlet structures did not have the capacity for managing the PMF.

2.2 LA PURISIMA STORM

La Purisima refers to the storm that occurred in December 2010, between the eight and nine of December and that has been the largest storm since the Panama Canal started operation. The discharge peak in the Gatun spillway reached 5,610 m^3/s and the maximum lake level 26.98 PLD (Figure 1). During this event, the lake level increased for more than 24 hours, even though the spillway was completely open and lock culverts were operating. Fortunately the rain ceased however this event further showcased the need of an additional spillway for Gatun Lake.

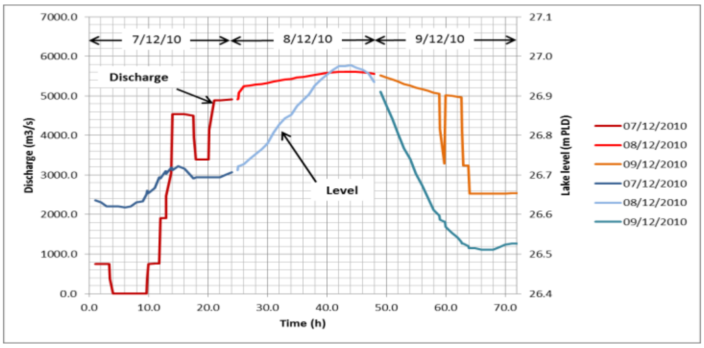

Figure 1. Discharge in the Gatun spillway and lake level during La Purisima event

3. LAYOUT ALTERNATIVES

After the evaluation by ACP of different locations for the project, the final area was established in narrow sector (around 150 m wide) and long (approximately 3000 m) between the old and new set of locks. The crest height was defined at level 21.03 PLD, same as the existing spillway, with a maximum lake level of 27.89 PLD. Considering these restrictions, it was necessary to design an approach channel, 450 m long, between the lake and the control structure. The channel will not only connect the structures, it will allow the necessary space for a natural plug or cofferdam to be used during the construction. The plug would be removed by dredging once the construction is finished.

The spillway control structure was located after the approach channel. The width of the gated spillway was defined by the range of the lake level, between 27.13 PLD (normal maximum) and 27.89 PLD (extreme maximum), and the required crest level. The required capacity of the spillway was determined from an iterative PMF transit, including all available outlet structures, until the maximum lake level was under 27.89 PLD. The existing outlet structures and their capacities are presented in Table 1.

Outlet	Discharge capacity m^3/s
Existing Gatun Spillway	6581
Old lock culverts	1359
New lock culverts	1995

Table 1. Existing outlet structures capacity.

The capacity for the new spillway was established at 4505 m^3/s in order to handle the PMF without overpassing a level of 27.89 PLD in the lake. The total spillway width was defined at 143.8 m, including 14 radial gates (8.6x6.4 m) and 13 piers with a thickness of 1.8 m.

The distance from the lake to end of navigation channel was around 3000 m, the navigation channel is the vessels approach to old locks from Atlantic and has a length of 6500 m between Limon Bay and Gatun locks. The level difference from the lake to ocean had to be dissipated in a conventional structure. Initially the sequence of several short stilling basins, in cascade, was evaluated but the use of only one stilling basin adapted better to the topographical and geotechnical conditions and was the more economical solution because it reduces excavation.

The chute channel, representing the connection between the spillway and the stilling basin, has a decreasing width, of 143.8 m at the spillway structure and 86.6 m at the stilling basin. A convergence channel was introduced for reducing construction quantities and costs. Figure 2 shows the spillway and transition sector.

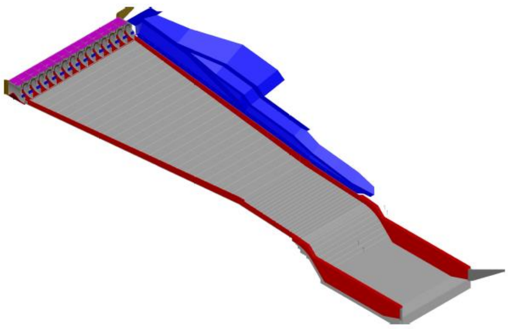

Figure 2. Three dimensional view of the spillway, chute contraction and stilling basin

The stilling basin was designed for a classical hydraulic jump. The addition of concrete blocks at the bottom of the stilling basin for reducing its length, was evaluated, but to avoid risk of cavitation and future repairs under permanent submerged conditions, this option was discarded. The length of the stilling basin structure was defined at 100 m with a sequent depth of 16.21 m that was established by tail water computation from Limon Bay and the lowest tide level.

The tailrace channel has a trapezoidal section with a total length of 2500 m. Its bottom width varies from the stilling basin (86.6 m) to the second reach or Muck reach (145 m). The first reach is located on rock foundation; the second reach is dimensioned for an average flow velocity below critical for muck material. As a result, the tailrace required an expansion, both in width and depth, from first to second reach. Figure 3 shows the Project plan view from Gatun Lake (left) to navigation channel (right).

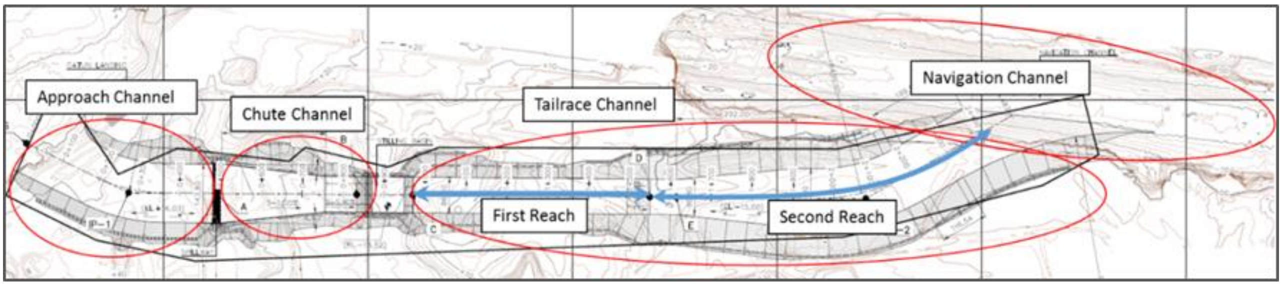

Figure 3. Layout of the project.

4. KEY HYDRAULIC ISSUES

The design of the new spillway contained the following hydraulic issues:.

a) Determination of the spillway rating curve definition using numerical and physical modeling. The methodology traditionally used to define the spillway crest geometry and its discharge coefficient have been applied successfully for decades, however, the approach conditions can generate deviations and therefore, the capacity has to be validated by modeling. In particular, the width of the structure is large, and flow depth is relatively low; therefore, a uniform flow distribution is not guaranteed.

b) Uniform Flow distribution in the chute channel from spillway to stilling basin. The chute channel is convergent to save costs, the angle of contraction was defined to reduce the cross waves transmission downstream and to avoid a choking condition. However, this was based on a theoretical approach and the interaction with spillway piers can only be properly evaluated by modeling.

c) Energy dissipation from stilling basin. The energy dissipation depends on the flow distribution in the basin entrance and the residual energy would indicate the length of protection downstream. Pressure fluctuation has to be confirmed to avoid cavitation and structural damage in the basin.

d) Uniform and slow flow in the tailrace channel to limit muck erosion. Muck material is present in about 60% of the tailrace length and along the navigation channel. This material is cohesive but very soft, cannot support heavy protection, and its correct installation under a submerged condition cannot be guaranteed. Modeling muck erosion is no possible, but theoretical flow distribution has to be confirmed by modeling.

e) Reduction of the effects on navigation channel and Limon Bay. The navigation channel starts in Limon Bay, that is, isolated from ocean by breakwaters. The behavior of the flow inside the bay, and the interaction with breakwaters openings, can only be analyzed by numerical modeling. Level variations at Atlantic Ocean are low, but the backwater could affect the hydraulic jump stability at the stilling basin.

4. PHYSICAL MODEL

The required prototype/model scale was 1:40. INA's laboratory (Instituto Nacional del Agua), located in Buenos Aires, Argentina, had the capacity for building and testing BEC's such a large size of the model (around 90 m long). Construction started with preparation of the area where the model was to be installed, known as the "deep water area building" with an indoor area of 2,450 m^2. Due to the considerable size of the model, some portions were constructed outdoors of the building, such as the pumping system, which was already in place.

The main objectives of physical modeling were:
- Capacity curve of the new spillway
- Pressure field on the spillway crest and stilling basin
- Cross waves pattern in the convergent chute channel
- Tailrace channel flow behavior and velocity field
- Flow Interaction at the joint tailrace-navigation channel
- Review of general flow behavior and correction alternatives

Any change in the design was first evaluated in the numerical model and later tested in the physical model. This way of working allowed to test quickly several alternatives only confirming the best solution in the physical model, saving time and costs.

5. NUMERICAL MODEL (CFD)

The numerical model was built using Flow3D software that can manage two and three dimensional flows. A hybrid 2D-3D model was used in order to break down the numerical domain on several subdomains. Different hydrodynamic models were implemented according to requirements for computation precision and representation of turbulence. This procedure allows a proper representation of the phenomenon in a computationally efficient way.

In the subdomains where low velocities and hydrostatic pressures are expected, the flow was represented by a shallow water model (2D). In the subdomains where high velocity and turbulence effects are expected, 3D Navier-Stokes equations, combined with the RNG version of the k-e turbulence, were implemented. Always a single-structured mesh was applied, and independence of results from mesh size was ensured by mesh interdependence analysis for all cases (Aydin & Ozturk, 2009).

A 2D model with shallow water conditions was used to analyze the flow on the navigation channel and Limon Bay. That model was built in Delft 3D because it was considered more suitable, this study area was not included in the physical model due to its size.

The numerical model was very useful for detecting problems and evaluating different solution alternatives that were then test and optimized in the physical model. Main findings from the numerical modeling are detailed explained below.

6. FLOW SEPARATION AT PROJECT ENTRANCE

The approach channel showed inefficient hydraulic behavior in the numerical model, confirmed also by the physical model. There was a significant flow separation towards the left bank that reduced the effective cross-section area and increased the flow curvature. This hydraulic pattern affected mainly the operation of bays one and two, located at left side of the control structure.

The best solution alternative was obtained with the inclusion of a groin in the left bank (Figure 4). The groin reduced the flow separation zone and improved the approach to bays one and two of the spillway. The reduction of the separation zone was relevant as the flow rotation in that area could gradually affect the slope stability and promote the accumulation of floating debris.

The groin implementation represented an increment in cost but produced a more reliable hydraulic behavior of the structure. The physical model showed that the capacity of the spillway was increased by 1.5% when including the groin.

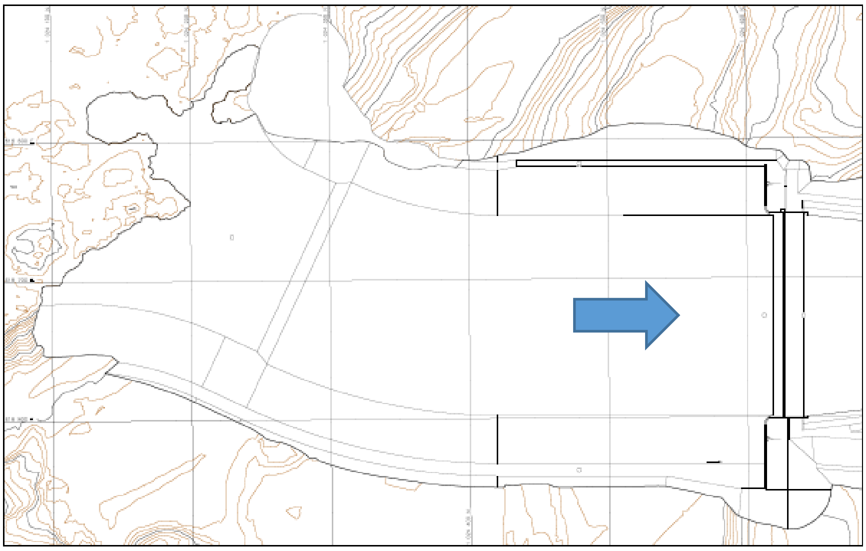

Figure 4. Entrance and approach channel, including groin.

7. ASYMMETRIC FLOW IN THE TAILRACE

The larger portion of the tailrace channel does not have erosion protection at the invert due to construction difficulties under a submerged condition. As a consequence, the tailrace was expanded to the original width in order to decrease flow velocities as much as possible.

The initial results obtained from simulations of the tailrace channel, showed that the flow is not well distributed over the entire channel width after the expansion and the transition was not effective. As a consequence, the flow turns into a jet that diverts towards the left bank. An additional concern was that the jet continues straight on, and impacts the right bank, at the end of the tailrace channel, where Atlantic muck is present. Additionally, the velocity vectors showed a recirculating pattern that represents potential erosion.

The phenomenon of asymmetric flow in symmetric expansions, has been studied since 1940, but is only found in a few references, and research on this subject has not been published in the more well-known references. Halim (1966) dealt with expansions in subcritical flow with width ratios of two and three and different shapes, in search of a simple and short expansion. Asymmetric flow was observed in his results, but no attention to the cause was given. He found good results from piers and mentioned that full-depth piers were very effective in Yu's experiments, due to the advantage of influencing the flow to its entire depth. Austin et al.(1970) mentioned an interesting evaluation of the lateral eddies in expansions presented by A.R. Thomas in 1940. However, this research was focused on head loss calculation along transitions. Graber (1982) presents a quite interesting analysis of asymmetric flow in symmetric expansions. He alleges that asymmetry is caused by differential pressure between lateral eddies that forces the main flow towards any one side. This unbalanced force is due to flow instabilities and occurs for width ratios above 1.5. Graber presents two options to solve the problem: 1) communication between eddies, to balance out the pressures and 2) a central bay to avoid pressure differences. Fani et al. (2012) presented an extended analysis of asymmetric flow in expansions with laminar flow. They found that the incorporation of a cylinder stabilizes the flow due to the inclusion of a drag force.

The concept of adding stabilizing forces by incorporating cylinders is comparable to the piers tested by Halim, and was incorporated in the numerical model. In total, six alternatives to improve flow distribution in the tailrace were tested in the numerical model, but the inclusion of cylindrical piers produced the best results and later was tested in the physical model. Main results are shown in Figure 5 and Figure 6.

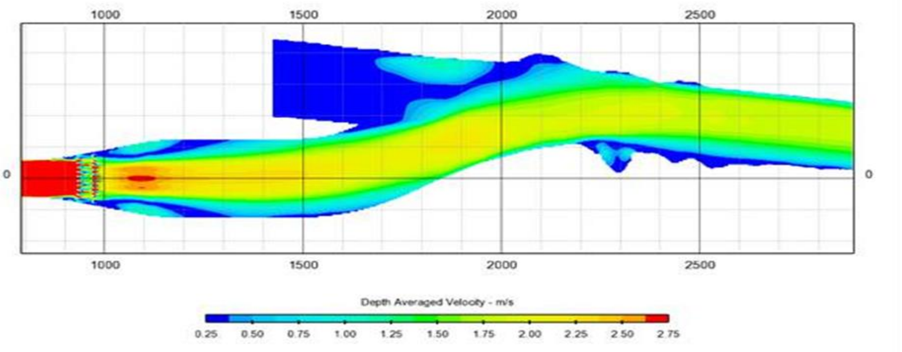

Figure 5. Flow distribution after piers inclusion.

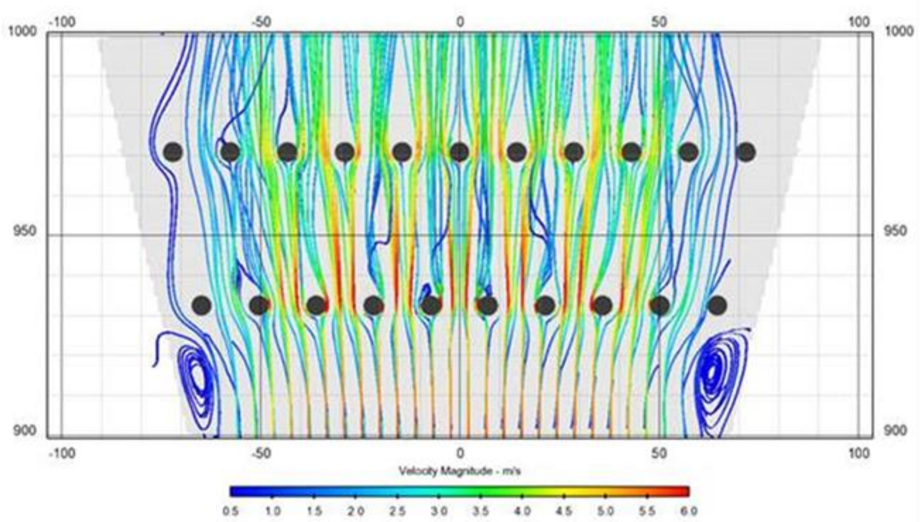

Figure 6. Flow between piers.

8. COMPARISON OF THE RESULTS BETWEEN PHYSICAL AND NUMERICAL MODELS

The comparison of results between models started with the approach channel tests. The numerical model showed a flow separation since the beginning of the left bank. The flow separation affected the channel hydraulic behavior and mainly Bays one and two of the spillway (Figure 7). The same behavior was observed later in the physical model (Figure 8 and Figure 9).

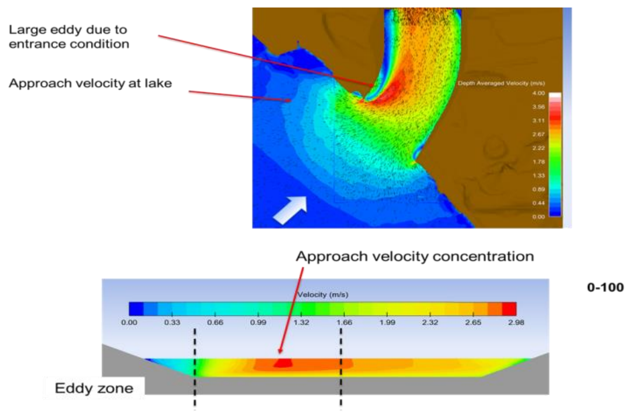

Figure 7. Velocity field and flow separation at project entrance from CFD

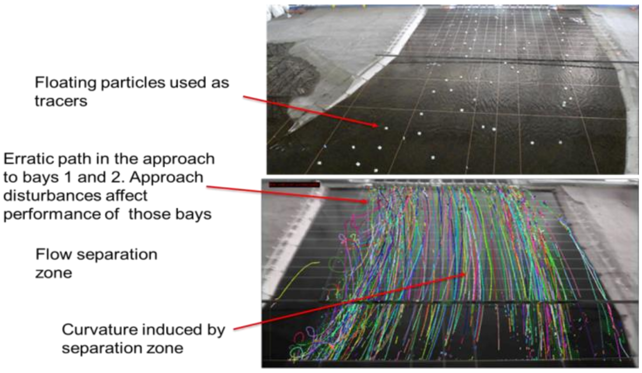

Figure 8. Flow pattern observed in the physical model

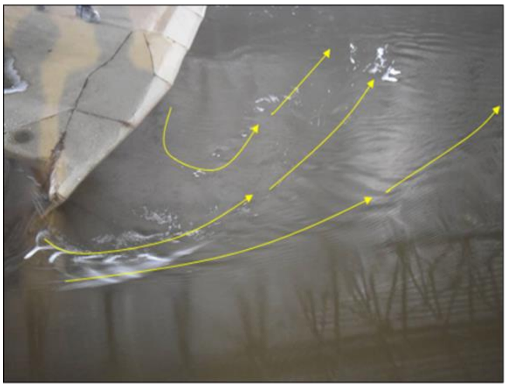

Figure 9. Flow separation at physical model

Although the model reproduced very well the theoretical capacity curve, there was a difference between the results from the numerical and the physical model. The physical model showed a smaller capacity of the control structure. It is possible that piers required a smaller mesh size but physical model showed an important effect on the prototype discharge for only a few millimeters in the model lake level, therefore measurement precision might be an issue. However, it was decided to select the more conservative data that came from the physical model.

In the chute channel, the general behavior observed in the numerical model was also observed in the physical model, however, the cross waves from piers, in the numerical results, and did not project until the vertical curve (compare Figure 10 and Figure 11). In the physical model, the cross waves from piers and contraction interact and create a complex flow towards the stilling basin. The complex flow found in the physical model affected the behavior of the hydraulic jump when it is not submerged.

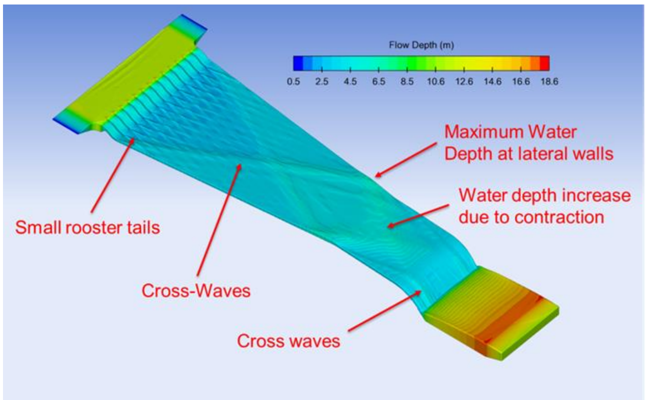

Figure 10. Cross waves in the chute channel from CFD.

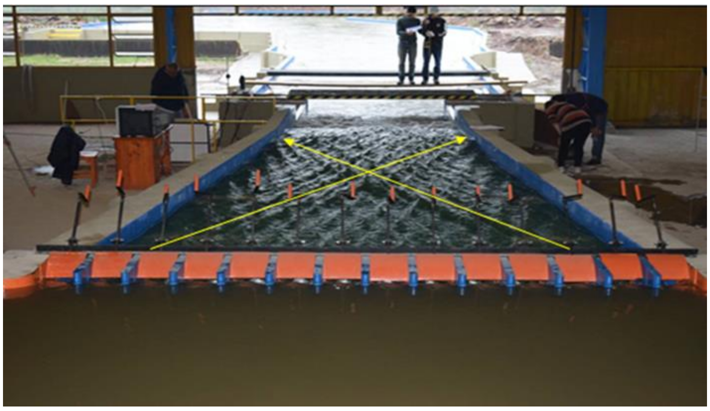

Figure 11. Cross wave in the chute channel, physical model.

The numerical model showed an asymmetric flow downstream the channel expansion (Figure 12). The asymmetry was validated in the physical model (See Figure 13).

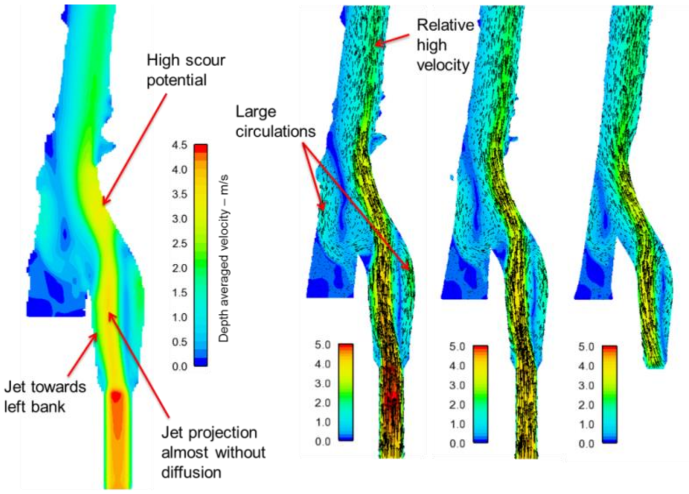

Figure 12. Asymmetric flow in the tailrace channel, CFD.

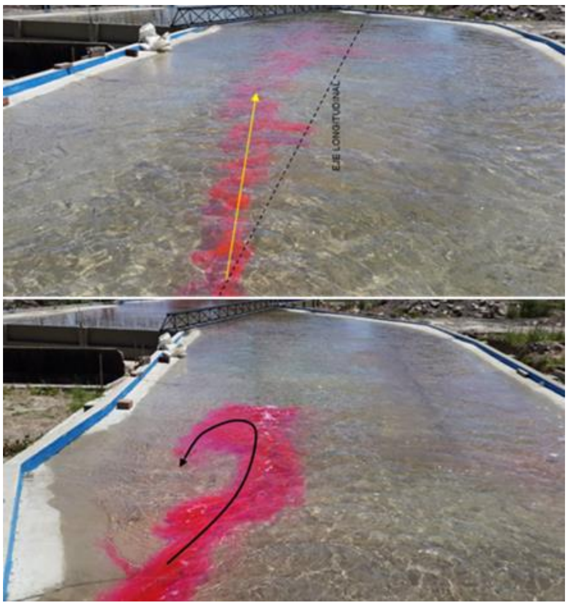

Figure 13. Asymmetric flow in the physical model.

9. CONCLUSIONS

Design of the BEC spillway has followed international guidelines and design procedures. The design evolved from its conceptual and preliminary design to detailed computations by use of numerical and physical models and the required adjustments from the geotechnical investigations program. Numerical and physical models complemented each other well and allowed to produce a robust design for the BEC spillway. Results from both models showed consistency and comparable values, except for the spillway capacity and the complex flow entering the hydraulic jump. Numerical model was very useful to correct the theoretical design where flow deviates from 1D behavior and to check changes before implementing them in the physical model. The physical model helped understand the complex flow in the chute channel where cross waves from piers and contraction interact.

Numerical modeling has advanced amazingly in the last 10 years and is approaching levels of confidence comparable to those of physical modeling; however, the magnitude of structures like the Panama Canal greatly benefit from the effort of complementing numerical and physical modeling since the first allows for fast and low cost changes. The implementation and analysis of numerical and physical models resulted in a robust design for the BEC spillway.

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