Design and Hydraulic Model of Gibe III Dam Spillway

Alessandro Cagiano de Azevedo **Alessandro Masciotta** **Francesca Pianigiani** **Antonio Pietrangeli**

Studio Ing. G. Pietrangeli Srl, Via Cicerone 28, 00193 Rome.

1 - Introduction

This paper describes the design of the spillway of the Gibe 3 dam, a hydropower project located on the Omo river in Ethiopia. The design of the spillway was very challenging in light of the large design floods, its imposing size and construction/operational constraints.
The dam is a gravity structure, about 250 m high, the world's highest using Roller Compacted Concrete (RCC) technology. The spillway is located on the dam crest and controls the discharge of the floods (up to 18'000 m^3/s) through seven large radial gates (12 x 17.5 m). The downstream slope of the dam body hosts the chute with seven independent channels and their flip buckets. A pre-excavated plunge pool controls the scour of the jets in the riverbed and protects the outdoor powerhouse, located downstream.
The spillway and plunge pool design were based on numerical analysis and optimized by means of a physical model (scale 1:60) built at the LACTEC laboratory in Curitiba, Brazil.

While the spillway is currently under construction, and its operation will commence next year, impounding of the reservoir started a few months ago to anticipate the beginning of power production. Construction and impounding programs were studied carefully to allow this early generation program.

This paper covers the hydraulic design of the spillway, the major findings of the physical model and some specific construction issues.

2 - Design Criteria

The spillway allows the safe discharge of the following extreme flows:
- Q = 18'000 m^3/s exceptional flood (Probable Maximum Flood (PMF) after reservoir routing)
- Q = 10'600 m^3/s design discharge for the spillway (10'000-year flood)
- Q = 6'500 m^3/s design discharge for the plunge pool (100-year flood)

The design criteria adopted in the analysis of the spillway vary with the flood type. The exceptional flood, also called "check flood", would be discharged through the seven chutes of the spillway considering a reduced freeboard. The parapet wall (h_{min}= 1 m) above the dam crest would also contribute to control the overtopping. These assumptions take into account that the PMF is an extreme event. The criteria adopted for the design flood, with a return period of 10'000 years, are significantly more stringent. The flows would be discharged through five gates only, considering two gates to be unavailable, with a freeboard of 4 m.

The design criteria for the pre-excavated plunge pool are less stringent. The excavation was designed using a flood with a return period of 100 years (Q = 6'500 m^3/s). In the event of an extreme flood, with a higher return period, some damage could occur to the plunge pool that would require maintenance work.

3 – Spillway Layout

The spillway is located on the central blocks of the RCC gravity dam. The overflow sill, with a total length of about 124 m, is divided into seven bays, each controlled by a 12x17.5 m radial gate. The key elevations are:
- 896 m a.s.l. dam crest
- 892 Full Supply Level (FSL)
- 875 spillway sill

The photo (Figure 1) shows the dam with the spillway on the crest, currently under construction, and the middle outlet opened to control the reservoir impounding.
While the dam axis is straight, the spillway sill is slightly curved to facilitate the concentration of the falling jets into the rather narrow river bed, less than 100 m wide. This arrangement is hydraulically favourable, but complicates the construction of the piers and walls since they are not aligned with the dam blocks.

Fig. 1: Dam and spillway under construction during reservoir impounding.

The gates will be equipped with stop-logs controlled from the dam crest by a gantry crane with a travelling hoist beam. The number of stop-logs will be sufficient to allow one gate to be out of service. The piers are nose-shaped to limit the flow contraction, avoid vortex formation and optimize the hydraulic efficiency of the sill. Several alternative shapes were tested in order to identify the optimum one. The pier joint was especially designed considering the curved spillway sill on the straight dam block. It is also worthwhile recalling that, during the development of the design, the number of gates was reduced from nine to seven to better concentrate the falling jets into the quite narrow plunge pool.

The spillway chute is about 75 m long ending in flip buckets with the lip at elevation 800 m a.s.l.. Sidewalls divide the chute into seven independent bays arranged in a slightly convergent plan. This arrangement is hydraulically favourable and each gate can be operated independently, guaranteeing a remarkable flexibility for operation and maintenance. However, the construction of the spillway is on the critical path and the addition of chutes and sidewalls may delay, albeit only slightly, the completion of the project.

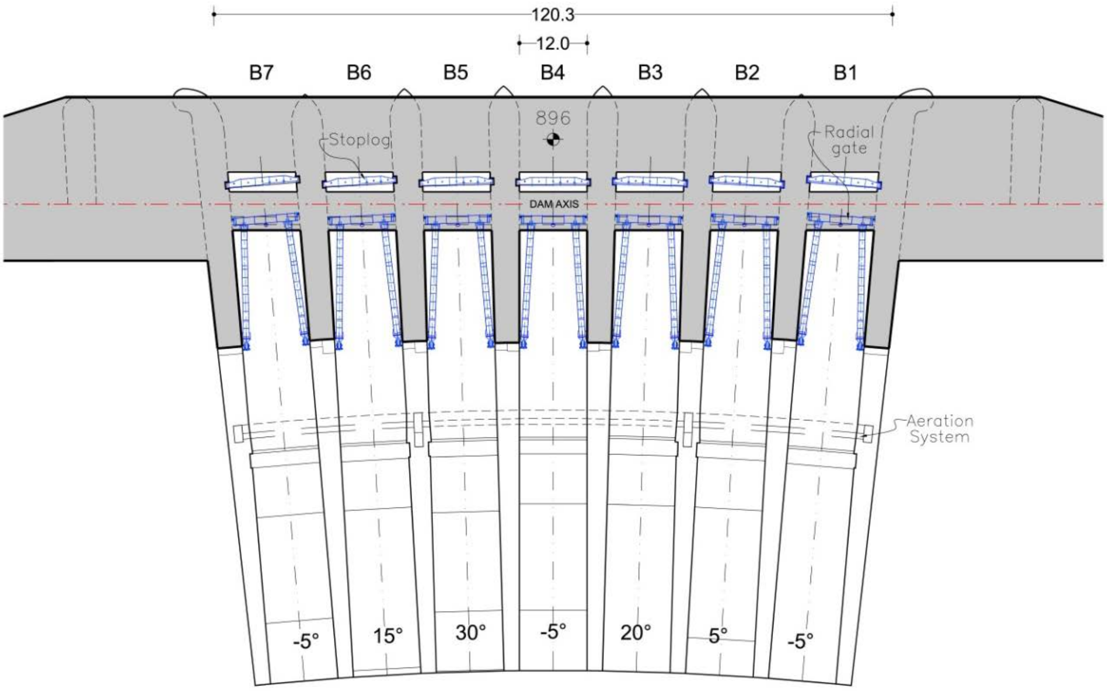

Fig. 2: Spillway plan

Since the velocity of the flows in the lower part of the chute is quite high, more than 40 m/s, one aerator has been added in each bay at elevation 830 m a.s.l. to reduce the cavitation. These aerators will extract air from the hollow lateral piers.

The optimum length of the chute was carefully analyzed to increase the jet length and energy dissipation in the air. The numerical and physical models indicate that scour in the plunge pool can be reduced by spreading the seven jets as much as possible. Therefore, the layout includes seven deflector buckets (at el. 800 m a.s.l., with 25 m radius) with three different lip angles, varying from -5° to +30°, aimed at directing each jet to a specific impact point of the plunge pool. The analysis also showed that the collision of two jets with a small angle has to be avoided since this could cause a significant local deepening of the scour. We also examined alternative layouts with the buckets at different elevations. However, varying the lip angle is more convenient than varying the lip elevation that would complicate the construction of the structure.

The pre-excavated plunge pool is about 300 m long and less than 100 m wide. The pool makes use of the river stretch between the dam and the power house, which is located on the left bank about 300 m downstream of the dam toe. The bottom of the pool is unlined but the slopes are bolted and, above the water level, also lined with shotcrete. The depth and steepness of the slopes have been established considering the geotechnical characteristics of the rock and the energy of the water jets to be dissipated. The lowest elevation of the pool is about 640 m a.s.l., nearly 50 m below the water level corresponding to the 100-year return period flood.

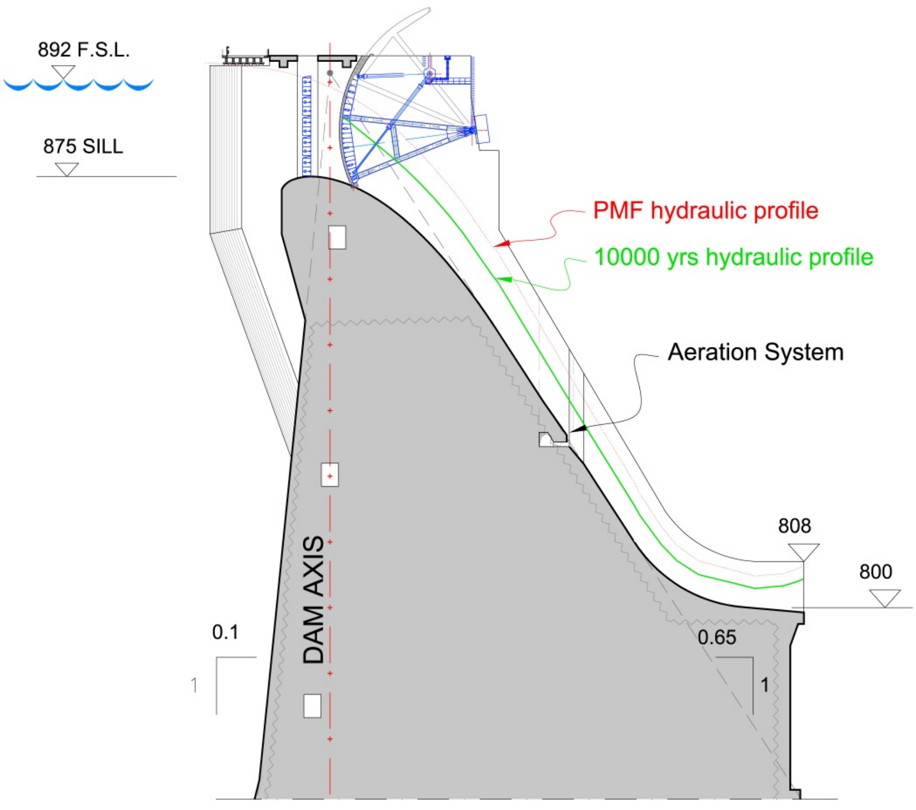

Fig. 3: Spillway, typical section

4 – Hydraulic Model

The hydraulic studies of the spillway and plunge pool were carried out using a 1:60 reduced scale physical model, adopting Froude similarity, built at the LACTEC laboratory in Curitiba, Brazil. The model reproduces part of the reservoir and the dam, the entire spillway (sill, gates, chute, flip buckets), the plunge pool and the powerhouse together with a stretch of the river to set the downstream flow conditions.
The spillway model, built in Plexiglas, was designed to accommodate a variable number of chutes (from 3 to 9) and several alternatives for the flip buckets (varying the lip angle in the range of -5 to 30 °). Erosion in the

plunge pool was studied using a movable bed. The granular material was selected adopting a geometrical similarity to the fractured rock of the river bed. Plunge pool behaviour was also analysed using a fixed bed measuring the pressures of the water jets through electronic transducers.

Numerous tests were carried out modelling several alternatives for the spillway and plunge pool. The findings of these tests substantially helped design optimization as discussed in the next section. The physical model was also useful to visualize the hydraulic behaviour of the entire system and allow the Client and other parties involved in the project to gain confidence with the performance of this large structure.

5 – Spillway Design

The optimum spillway layout comprises seven independent chutes, separated by sidewalls. These separated chutes allow the scattering of the trajectories of the jets in the plunge pool, significantly reducing the scour and regulating the back currents.

Fig. 4: Physical model (scale 1:60): test of the design flood with seven chutes

An alternative scheme with three chutes and curved buckets was also tested. The reduction of the number of side walls would significantly facilitate the construction, but while the scour would be quite similar to the seven chutes alternative, operating the spillway would be significantly more complex since unless all three gates were open simultaneously, the flow would be asymmetrical and the water jets would hit the river banks.

The pier types selected for the first tests were USACE’s type 2 and 2c. A significant instability of the flows was observed, especially with partial gate opening, with the formation of intense vortices near the gates. The surges on the pier were about 1 m high and the vortex entrained air and caused unacceptable perturbation in the flows along the chute. Tests showed that changing the gate position or the pier length made little difference so a trial run was carried out with a vortex suppressor device which eliminated the vortices improving flow conditions along the chute. A shape similar to USACE pier type 3 was therefore tested for the pier heads. The vortexes disappeared and pier type 3 was adopted in the design.

Selection of the optimum layout for the flip buckets required the comparison of several alternatives using different lip angles. The schemes were compared using the movable bed to model the scour in the plunge pool. The optimum solution was reached by analysing the resulting erosion/deposition achieved at the end of these tests. The following photo (Figure 5) shows the trajectories of the jets, varying the lip angles of the buckets from –5 ° to 30° while maintaining the lip elevation unvaried at 830 m a.s.l.. The scattered trajectories of the jets not only favours the energy dissipation but also better regulates the backward currents, reducing the maximum scour and avoiding local potential concentration of erosion. The sidewalls of the chutes converge in order to better direct the flows towards the centre of the river bed, within the pre-excavated plunge pool.

The measured flow velocity reaches maximum values of 40-45 m/s, near the end of the chute. Analysis of the cavitation indicates the need for an aerator since, without it, the cavitation index would fall below 0.25 at about elevation 830 m a.s.l.. An aerator was placed at 830 m a.s.l., about 45 m far from the spillway sill.

Fig. 5: Scattered jet trajectories, varying the lip angle of the buckets (from -5° to 30°)

6 – Design of the Plunge Pool

The project includes a pre-excavated plunge pool to control the scour caused by the plunging jets from the spillway and protect the powerhouse. The space available for the plunge pool is limited by the narrow width of the valley and by the powerhouse, located just downstream. Therefore, pre-excavation of the pool was envisaged to increase the water cushion and the volume of water able to dissipate the energy, better controlling the scour process.

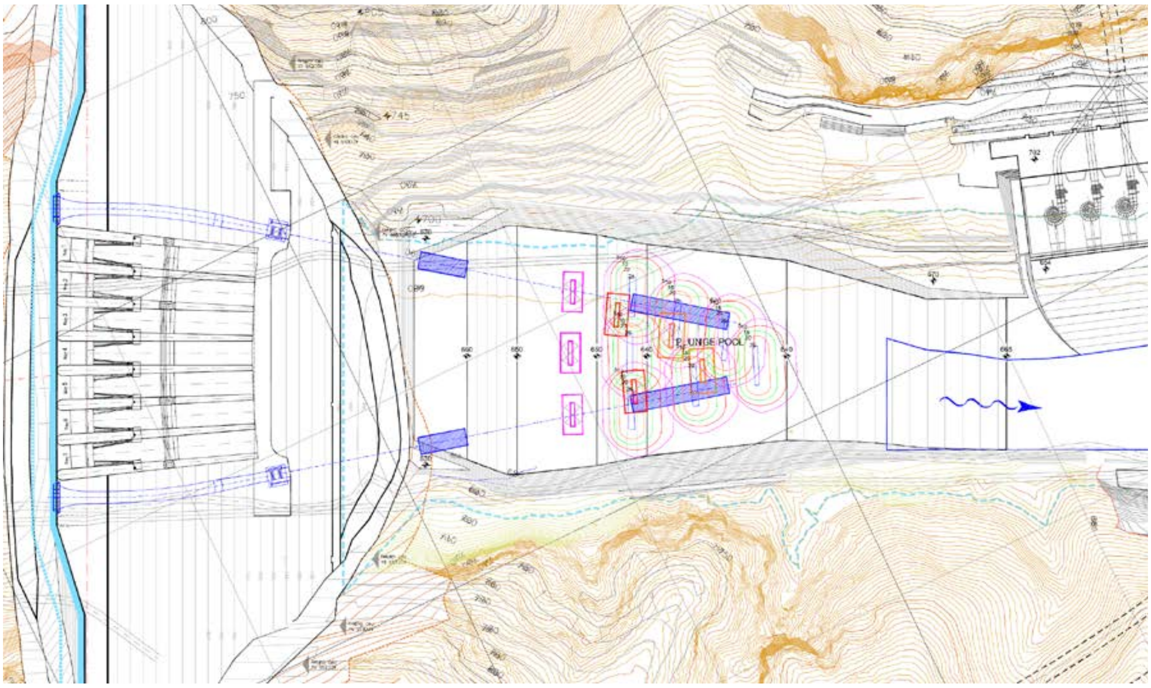

Fig. 6: Layout of the spillway and plunge pool (the impact areas of the spillway jets are indicated in magenta and those of the two middle level outlets in blue).

A deflecting wall upstream of the powerhouse will convey the flows into the downstream stretch avoiding interference with the hydraulic operation of the turbines.

The plunge pool design is based on the outputs of numerical analysis carried out using several independent approaches and then verified and refined using the physical model. The first analysis carried out was based on empirical equations using the modified Veronese formula (adapted by Yildiz in 1994), Mason's formula and Brito's equation. The excavation level of the plunge pool was preliminarily set at 640 m a.s.l,, slightly above the value obtained with the modified Veronese equation (636 m a.s.l.).

The impact of the plunging jets on the bottom of the plunge pool bottom was then evaluated using the Erodibility Index Method proposed by Annandale in 2006, based on the specific stream power of a plunging jet transmitted to the rock. The distribution of the dynamic pressures on the plunge pool bottom were examined and compared with the rock erosion threshold value. The break-up length of the jet was calculated and found to occur before the impact on the water surface implying that a certain amount of energy will be dissipated in the air. The index indicates an erodibility threshold of 550 kW/m^2 for the pool bottom, the same order of magnitude (slightly lower) as the stream power of the impingement jets on the pool bottom set at 640 m a.s.l., evaluated with different methods to be in a range of about 700 kW/m^2.

We then used the physical model with movable bed to assess the pattern and maximum depth of the scour, together with the flow recirculation in the pre-excavated plunge pool.

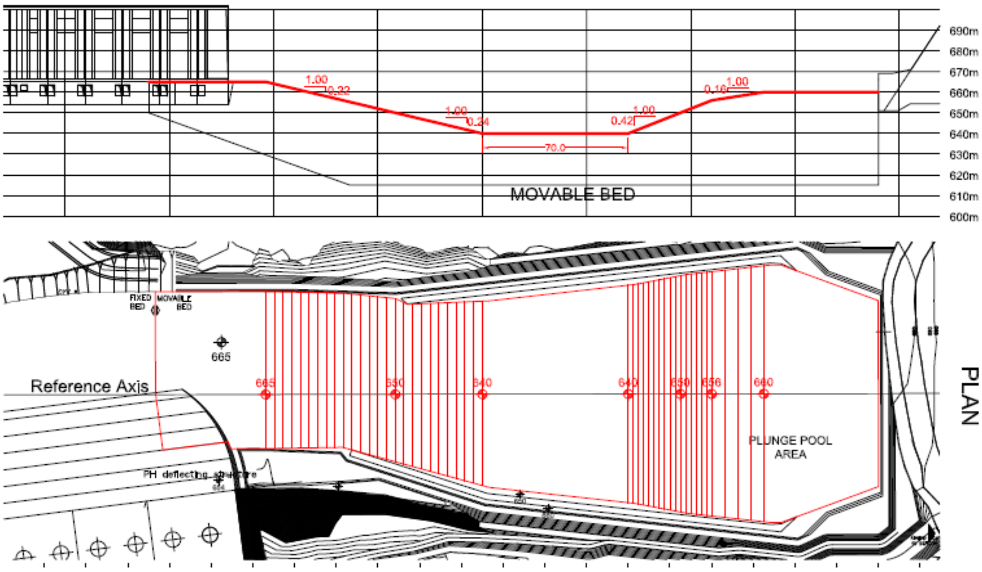

Fig. 7: Plan and profile of the pre-excavated plunge pool

The findings of the movable bed model showed that the pre-excavated plunge pool is effective. After the occurrence of the design flood (Q = 6'500 m^3/s, Tr = 100 years) the scour is quite small, a few meters only, and the eroded material remains within the plunge pool so operation of the powerhouse will not be affected by erosion in the plunge pool.

Fig. 8: Physical model with movable bed: bottom of the plunge pool after scour

The depth and steepness of the plunge pool slopes were evaluated according to the geotechnical characteristics of the rock and considering the energy to be dissipated from the water jets, taking into account the footprint of the impingement jets on the bottom of the plunge pool.

Extraordinary situations were also simulated in the model and results of these tests indicated that the spillway is able to operate in many adverse conditions. Higher discharges, up to 10’000 years’ flood, could carry material out of the plunge pool depositing it in front of the powerhouse, but significant damage to the structure would happen only for extremely high discharges.

A comparison of the maximum depth of scour using the empirical equation is given in the following table.

			Max. Scour Level		
Ret. Period	Discharge	Water Level	Physical Model	Veronese’s Formula (adapted by Yildiz)	Mason’s Formula
[years]	[m^3/s]	[m a.s.l.]	[m a.s.l.]	[m a.s.l.]	[m a.s.l.]
100	6’500	688	633.2	636.6	634
10000	10’600	695	626.7	626.3	616

Table 1: Plan and profile of the pre-excavated plunge pool

The maximum scour measured with the physical model is very similar to that obtained with Veronese’s modified formula and, for the pool design flood, also to Mason’s equation. As shown in Annandale’s index, some scour occurred on the bottom of the pool pre-excavated at 840 m a.s.l.. The results of the physical model confirm the findings of the basic numerical analysis carried out using the methods indicated.

An additional analysis of the stability of the plunge pool bottom, assessing the resistance of the rock to the dynamic pressures induced by the falling jets, was conducted applying Bollaert-Schless equations and Häusler theories. The results were used to design the angle of each flip bucket to obtain, by means of separated and adequately distanced jets, an almost uniform distribution of the pressures on the bottom of the pool. The fixed-bed physical model tests substantially confirmed the results of the numerical analysis which, albeit affected by some intrinsic unpredictable aspects, also allowed us to discover critical operating conditions leading to the definition of specific criteria for the spillway gate operation.

The best hydraulic performance of the spillway and plunge pool is achieved with all seven gates operating at the same time to have well distributed and reduced fluctuating dynamic pressures at the bottom of the plunge pool. Under such conditions, the adopted water cushion in the plunge pool significantly diffuses the jets’ energy before reaching the bottom of the pool. The maximum measured dynamic pressures are around 120 m of water head, with a 10’000 years’ flood, including the water cushion and the additional dynamic pressure caused by the jet (about 50 m).

The edge of the pool, near the dam toe, was reinforced and protected to prevent regressive scouring due to back-currents which could not be significantly reduced given the pool's limited width. Pressure contour lines on the pool bottom were identified for design flow conditions, and used to check the excavation supports of the pool abutments.

7 – Construction Issues

The construction of the spillway is challenging especially because of its size, the steep slopes of the dam and the height of its walls and piers. Moreover, while the spillway is currently under construction and its operation will commence next year, impounding of the reservoir started a few months ago to anticipate the beginning of power production. In order to facilitate and accelerate construction, as much as possible, several specific design arrangements were adopted as discussed here below.

Placement of spillway concrete proceeds using cranes installed on the dam and double-step pumps from the downstream face together with ramping formworks on the upstream face, where the spillway piers will need extensive reinforced connection with the RCC dam body. In the meanwhile rapid placement of the RCC proceeds independently over the entire dam length, thanks to the RCC conveyors bridging the spillway piers.

The downstream face of the dam was retroceded in the spillway zone, separating the construction of the dam body from the spillway.

Connecting bars for the reinforced concrete slab of the spillway chute are placed in the upper layer of each 3 m high step of RCC, bent to sew the slab orthogonally and to avoid crossing the formworks while a porous concrete kerb along the downstream face of the RCC acts as a drain below the final concrete slab.

Fig. 9: Spillway chute under construction

The radial gate tendons will be installed directly from a dedicated chamber inside the spillway piers, facilitating access to the tendons. The concrete surface of the chute requires a particular finishing suitable for the high velocity flows. Therefore, a second stage, 20 cm thick slab made of high durability and resistance concrete (C40 D20 with silica fume) was conceived to be built separately, without construction joints, poured continuously with a sliding formwork.

8 – Conclusions

The design of the Gibe III spillway was very challenging in view of the large design floods (up to 18’000 m^3/s), the imposing size of the dam (H = 250 m) and the fact that impounding of the reservoir started well before completion of the spillway.

The layout of the spillway includes seven independent bays arranged in a slightly convergent plan. This arrangement is hydraulically favourable and each gate can be operated independently, guaranteeing a remarkable flexibility for operation and maintenance. The numerical and physical models indicate that the scour can be reduced by spreading the jets as much as possible and so the layout includes seven deflector buckets with different lip angles, varying from -5° to +30°, to direct each jet to a specific impact point in the plunge pool. The scattered trajectories of the jets not only favour energy dissipation but also regulate better the backward currents, reducing the maximum scour and avoiding potential local erosion concentration.

Since the velocity of the flows in the lower part of the chute is more than 40 m/s, aerators were added in each bay at elevation 830 m a.s.l. to reduce the risk of cavitation by extracting air from the hollow piers.

The space available for the plunge pool is limited by the narrow width of the valley and by the powerhouse, located just downstream. Therefore, pre-excavation of the pool was envisaged to increase the water cushion thereby better controlling the scour process. The findings of the movable bed model confirmed that the pre-excavated pool is effective since after occurrence of the design flood (Q = 6’500 m^3/s, Tr = 100 years) the scour was quite small, only a few meters, and the eroded material remained within the plunge pool. Powerhouse operation, therefore, will not be affected by erosion in the plunge pool.

The Authors:

A. Cagiano de Azevedo, obtained his degree in civil hydraulic engineering from the University of Rome “La Sapienza”. He works for Studio Pietrangeli as senior project manager on important projects such as Gibe II, Gibe III, GERDp, Batoka and Namakhvani cascade. His expertise, including project management, design coordination, budgeting and scheduling activities and cost assessment, ranges from the phases of design to supervision of construction of dams, hydropower plants and large hydraulic works.

A. Masciotta, obtained his degree in Civil & Hydraulic Engineering from the University of Rome “La Sapienza”. He has more than 20 years’ experience as a designer olarge civil works including dams and hydropower projects. He has a remarkable expertise in the design of large hydraulic structures thanks to his knowledge both of hydraulic and structural engineering. He has been involved for Studio Pietrangeli in the design of several of the firm’s most important projects.

F. Pianigiani, obtained her degree in Civil Engineering from the University of Perugia. She has been working for Studio Pietrangeli as senior hydraulic engineer for five years and has participated in feasibility studies, detailed design and supervision of construction of large dams and hydropower plants in various African countries.

A. Pietrangeli, obtained his degree in civil hydraulic engineering from the University of Rome “La Sapienza”. In recent years, as managing partner of Studio Pietrangeli, he has been directly responsible for the technical direction and overall management of many of the firm’s projects covering more than 30 large dams (up to 250 m high) and 16 large hydroelectric plants (totaling more than 10’000 MW) in Africa, Europe and South America.