

THE INTERNATIONAL JOURNAL ON HYDROPOWER & DAMS
"MODELLING, TESTING & MONITORING FOR HYDRO POWERPLANTS - II"

CONFERENCE PAPERS. LAUSANNE (SWITZERLAND)

8-11 JULY 1996 (pp. 489-499)

**MODIFICATION OF THE SPILLWAY
ON THE PUEBLO VIEJO DAM (Guatemala)**

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1. Introduction

The Pueblo Viejo - Quixal hydroelectric scheme (Chixoy) is located on the Chixoy river near the city of Coban in the central part of Guatemala. The 300 MW scheme, completed in 1986, is the largest hydroelectric complex of Guatemala. The dam site is located at the entrance of a 150 m deep gorge of sedimentary rocks where limestone, dolomite and limestone-breccia formations prevail. The 130 m high Pueblo Viejo rockfill dam with central clay core impounds a 460.10^6 m³ gross capacity reservoir with a surface of approximately 14 km². A 26 km long headrace tunnel with a design capacity of 75 m³/s supplies the five Pelton units of the Quixal powerhouse. Figure 1 shows a general plan view of the dam and of the ancillary works.

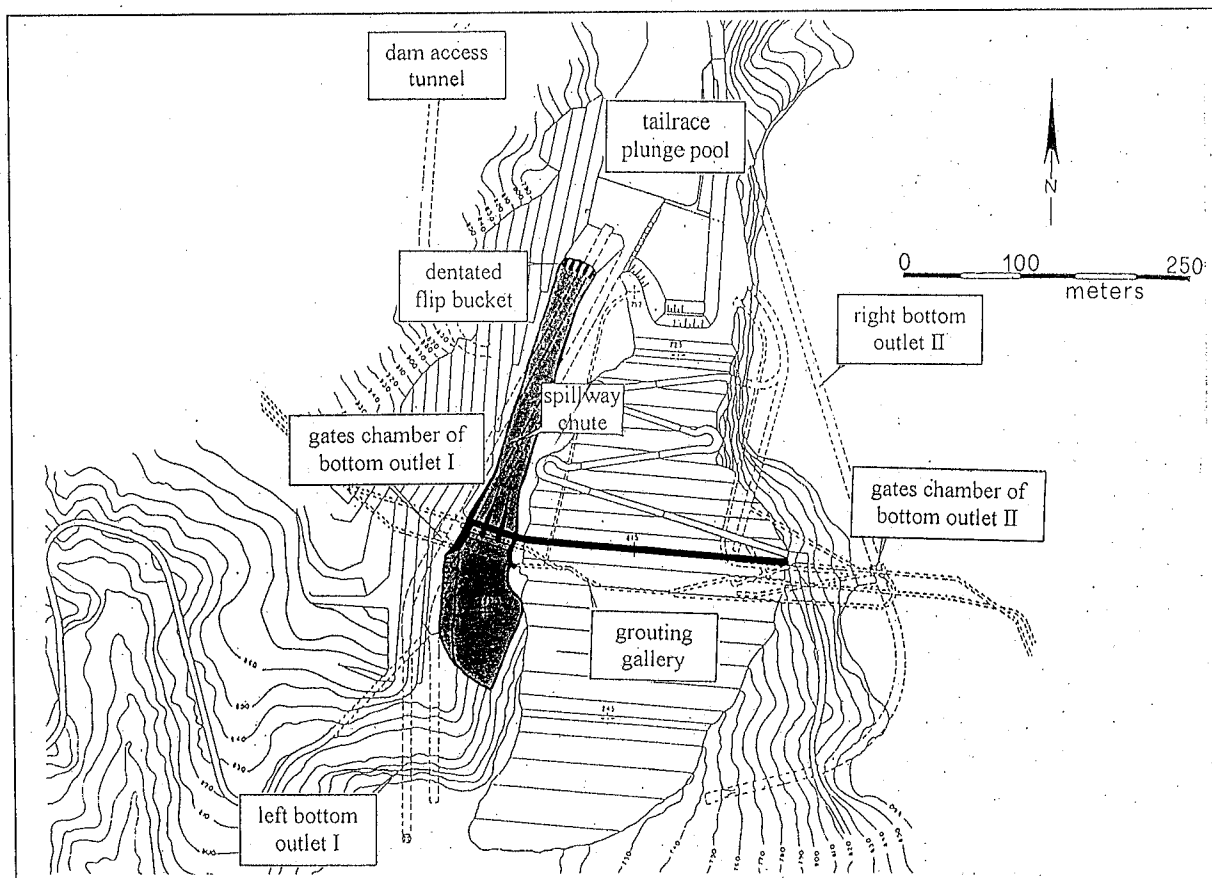


fig.1. - General layout of the Pueblo Viejo dam and ancillary works

The spillway structure, excavated on the left bank, consists of three overflow bays completed further downstream by a spillway chute ending with a dentated flip bucket. The spilling crest designed to be equipped with radial gates has a standard shape corresponding to a 11 m design head. Its maximum capacity of 3890 m³/s is reached with an hydraulic head of 14.0 m resulting in a maximum water level at 814.0 m a.s.l. A general view of the spillway including the inlet works, sill and spillway chute is shown in figure 2.

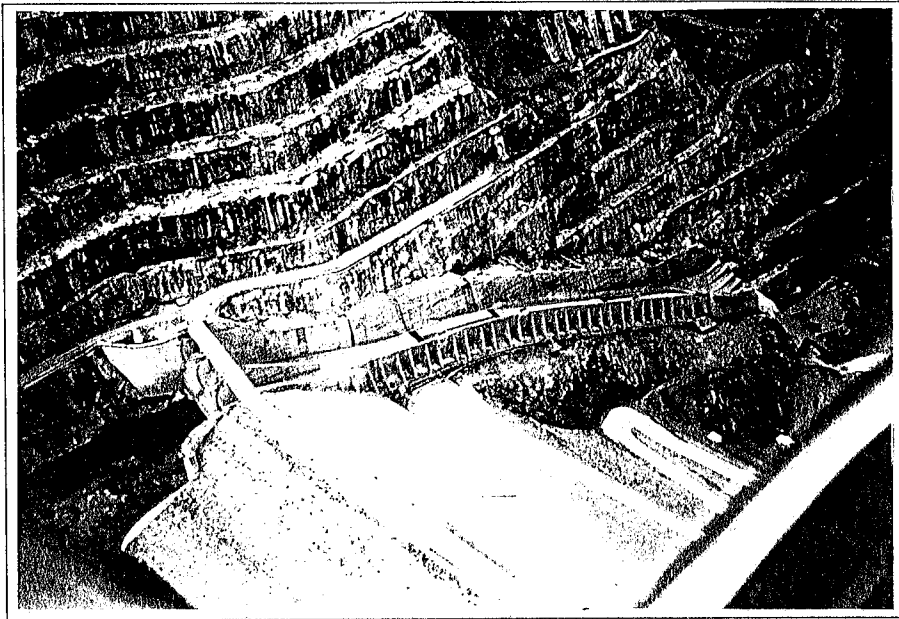


fig.2. - General view of the spillway

The revision of the design flood from a 1000-year to a 10000-year event, during the dam construction, led to the postponing of the gate installation. Namely, it was suggested to install the gates only after the completion of an upstream located hydroelectric scheme leading to a reduction of the peak inflow discharge at the Pueblo Viejo reservoir. Since the construction of the upstream scheme was abandoned by the National Power Authority (INDE), alternative solutions were examined to raise the impounding level at the normal water level considered for the dam design. The reduction of the available flood storage caused by the raise of the normal water level requires an increase of the spillway capacity in order to maintain the present safety during major floods.

The paper presents various alternatives to modify the present spillway in order to increase its capacity. In all considered cases the different parts of the structure have been designed to reach the optimal hydraulic functioning, providing the spillway with gates for a 3.0 m raise of the normal water level. Due to relatively complex approach conditions, a model study was required to investigate the flow conditions at the inlet as well as in the spillway chute. The design of the spillway modification was carried out by Lombardi Engineering Ltd., whereas the model studies were undertaken at the Laboratory of Hydraulic Constructions of the Swiss Federal Institute of Technology, in Lausanne. Parts of the theoretical approaches as well as results of the model tests are presented hereafter.

2. Variant analysis

2.1. General design considerations

Following severe floods during construction, the consultants group revised the design flood increasing the peak flood inflow from 5200 m³/s to 7500 m³/s and the flood volume from 320 hm³ to 450 hm³ maintaining a peak time of 15 hours. Under these conditions the installation of gates on the overflow weir raising the normal water level at el. 803.0 m a.s.l. was considered too risky due to the excessive reduction of the safety margin between the design flood and the flood causing a dam overtopping. It has to be mentioned that the maximum level in the reservoir during extreme flood is very sensitive to any variation of the flood volume rather than to the peak inflow.

Since a preventive drawdown of the reservoir prior to the flood based on a real-time flood forecasting system was not considered as sufficiently reliable in case of extreme events, a raise of the normal water level had to be combined with an increase of the outflow capacity of the dam. Preliminary flood routing analysis indicate a required increase of the outflow capacity between 400 m³/s and 600 m³/s considering a 3.0 m raise of the normal water level. To achieve this 10-15% increase of the outflow capacity, the following basic measures were examined (including their combination) :

- operation of bottom outlets,
- construction of an auxiliary spillway,
- modification of the existing spillway.

The operation of the bottom outlets involves various safety and operational aspects which have to be taken into consideration. Since no reliable flood prediction system would be available, the opening of the bottom outlets shall take place at the beginning of any flood and not only in case of extreme events. Due to the frequent and

prolonged operation (exceeding in average 1 month per year) repair works on the bottom outlets would be needed periodically.

As regards the construction of an auxiliary spillway the limited space available on both abutments would most probably require the excavation of a tunnel. Including the inlet and outlet works, the construction of an auxiliary spillway would therefore involve relatively high costs.

Finally as regards the modification of the existing spillway, two solutions may be considered, namely : lowering of the weir crest elevation to increase the specific discharge or increasing the length of the spillway crest. Both alternatives were examined by model studies as presented in the following paragraphs.

2.2. Increase of the spillway capacity

In order to increase the discharge capacity to 4200 - 4500 m³/s, maintaining an unchanged 1.0 m freeboard during a 10000-years flood both the present and the modified spillway were investigated by model tests. As regards the modified weir, the following two basic solutions were examined:

1. - Lowering the weir crest elevation and modification of its profile ;
 - installation of three tainter gates;
 - reduction of the pier contraction coefficient by increasing the distance between the piers' noses and the spillway crest (variant B1, figure 3.a.) or by modifying the upstream piers' noses, with a more hydrodynamic shape (variant B2, figure 3.b.).
2. - Removing the two central piers without modifying the weir crest elevation and geometry;
 - construction of a single central pier for the installation of two identical flap gates (variant C, figure 3.c.).

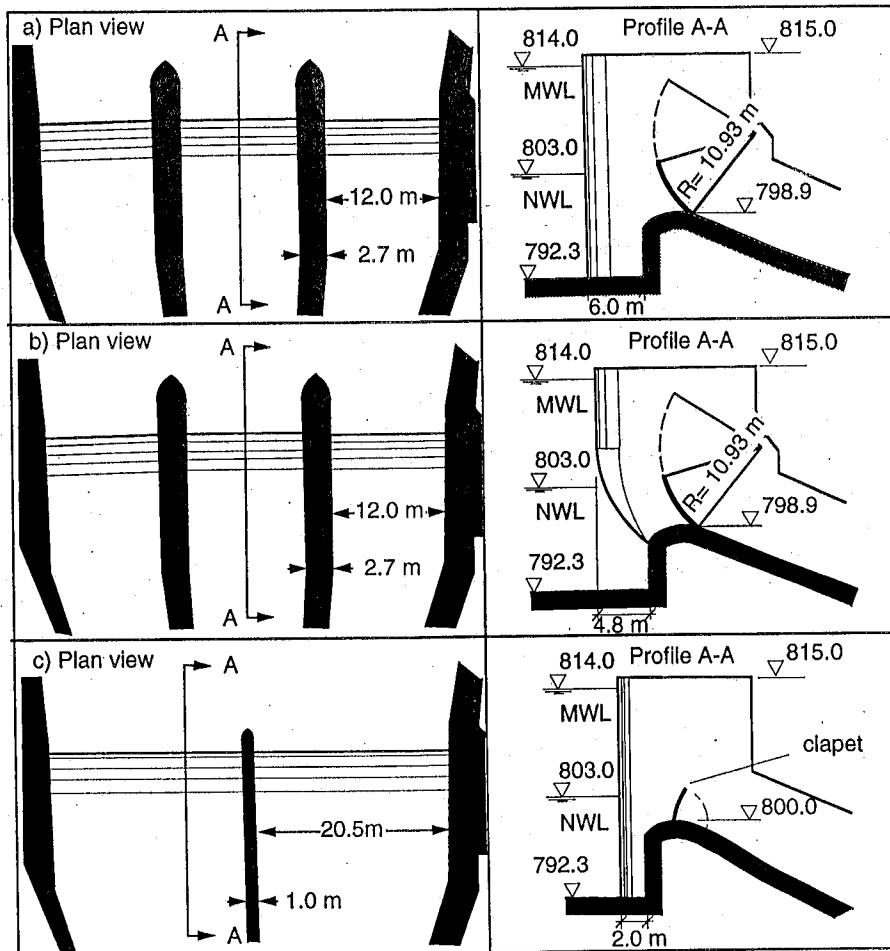


fig.3. - The examined solutions : a) variant B1; b) variant B2; c) variant C.

Concerning variants B, flood routing calculations have been carried out to determine the optimal elevation of the weir crest for different operational conditions. For these calculations, the required freeboard of 1.00 m was respected for a 10000 year flood. The maximum water level in the reservoir is therefore limited to 814.00 m a.s.l. Figure 4 shows the relation existing between the normal water level, the elevation of the weir crest and the maximum outflow while respecting the minimum freeboard. It is assumed that during floods the gates will be

operated to maintain a constant water level in the reservoir up to the complete opening of the gates. Lowering the crest level from 800.00 m a.s.l. to 798.90 m a.s.l. results in a maximum spillway capacity of 4370 m³/s corresponding to a 12% increase. It is felt that this value will not require a complete reassessment of the hydraulic characteristics of the spillway. Moreover, a possible malfunction of the gates has to be considered keeping in mind access difficulties and the severe seismic conditions of the region.

As shown in figure 3, in variant C, the two central piers are replaced by one central 1m large pier, resulting in a nearly 14% increase of the effective crest width. In this case, the weir can be maintained at the present elevation limiting the demolition works to the excavations required for the installation of the flap gates.

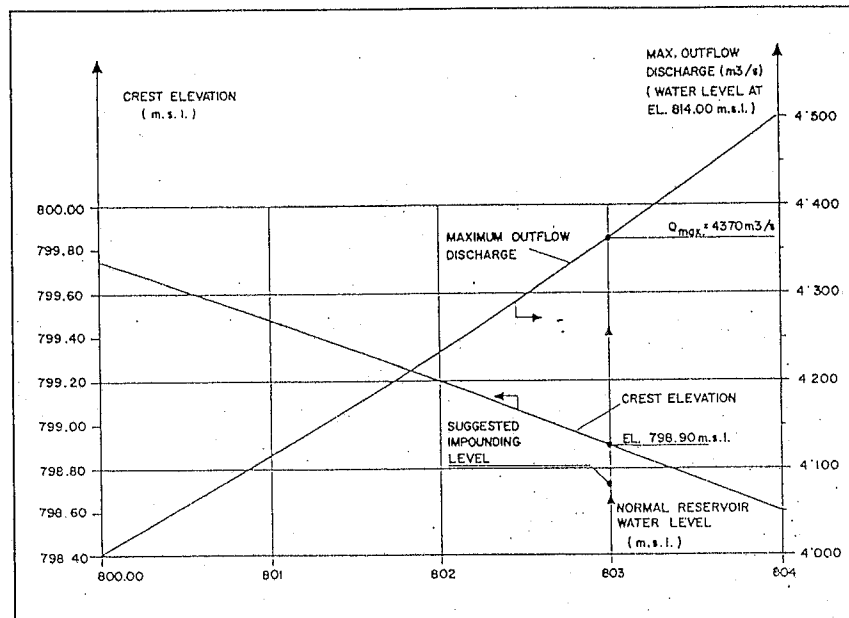


fig.4. - Lowering of the spillway crest (variants B1 and B2): relation between the crest elevation, the normal water level and the maximum outflow discharge with a minimum freeboard of 1.00 m (powerhouse in operation)

In order to evaluate the capacity of the actual spillway and of the suggested alternatives, the classical approach relation has been adopted considering the relation: $Q = C_d B H \sqrt{2gH}$, where Q is the total discharge over the spillway crest, C_d the discharge coefficient, H the energy head on the crest, and B the effective crest length. The last is expressed by the equation: $B = L - 2(2k_p + k_a)H$, where L is the net length of the crest, k_p is the pier contraction coefficient and k_a is the abutment contraction coefficient. The following assumptions were made for the calculations: approach depth effects on the discharge coefficient are based on the low ogee crest case; the pier contraction coefficient is assumed as $k_p = 0.04$ for the actual situation and $k_p = 0.00$ for the suggested solutions. Finally, the abutment contraction coefficient is considered equal to $k_a = 0.04$ both for the actual and the modified spillway geometries.

2.3. Cavitation risk

As part of the hydraulic analyses carried out both on the actual and the modified spillway, it was felt useful to examine the risk of cavitation on the spillway chute. It has to be mentioned that the chute is equipped with three aerators located respectively 80, 110 and 175 m from the spillway crest. The last aerator is located immediately upstream of the dentated flip bucket. Although no cavitation damages were registered during the more than 1000 days of spillway operation, a cavitation risk analysis was carried out in order to identify the most critical flow conditions. The main purpose of the analysis was therefore to verify that the flow conditions caused by the gates installation may not introduce any additional cavitation risk on the chute and in particular on the flip bucket.

The theoretical approach used is based on the USBR monograph "Cavitation in chutes and spillways" [4]. The chute has been modelled to compute the hydraulic and cavitation characteristics of the flow for various discharges. The prediction of cavitation occurring at the boundary is made by comparing the cavitation index of the flow to the cavitation indices of the surface irregularities which may be anticipated.

The computer program used for the study [4] was developed to calculate the hydraulic and cavitation properties of free surface flows. The program is based on the determination of the water surface profile using the standard

step method [5], which assumes gradually varied flow in a uniform channel. Algorithms are used to account for changes in shape of the channel and curvilinear flow. Although the simplifications are somewhat crude, the algorithms produce results having sufficient accuracy for the engineering analysis purpose. The program output includes the usual flow parameters as well as the cavitation characteristics of the structure [4].

Several parameters have been developed to predict the potential cavitation damage on hydraulic structures. It is generally recognised that the cavitation index of the flow, the surface execution quality and the time of exposure are the most important parameters. These parameters are evaluated as follows :

- Cavitation index of the flow σ :

$$\sigma = \frac{P_r - P_v}{\rho V_r^2 / 2}$$

where P_r is the reference pressure, P_v the vapour pressure of water, V_r the reference velocity and ρ the of water density.

The cavitation index indicates the state of cavitation in hydraulic structures. Although the cavitation process starts at smaller values, significant damage necessitating repair has been observed for $\sigma \approx 0.2$.

- Cavitation index for uniform roughness σ_u :

$$\sigma_u = 4f$$

where f is the Darcy-Weisbach friction factor.

If the cavitation index of the flow σ is lowered below the cavitation index for uniform roughness σ_u , cavitation will occur in sheets within the flow body and be caused by the roughness of the structure surface.

- Damage potential D_p and damage index D_i :

$$D_p \propto \left(\frac{1}{\sigma_s}\right) \left(\frac{\sigma_s}{\sigma} - 1\right) \left(\frac{V}{V_r}\right)$$

$$D_i = D_p \cdot \ln(t - t_0)$$

where σ_s is the cavitation index for the initiation of damage, σ the cavitation index of the flow, V the flow velocity, V_r the reference velocity, t the time of exposure and t_0 the integration constant.

While damage potential D_p describes the aggressiveness of cavitation, the damage index D_i includes the time of exposure to measure the severity of the cavitation damage.

The cavitation characteristics of the structure were studied for three different discharges and for the actual and the modified spillway configurations. The cavitation index of the flow in the channel is presented in figure 5 for the most critical flow condition, at different values of the concrete roughness k_s .

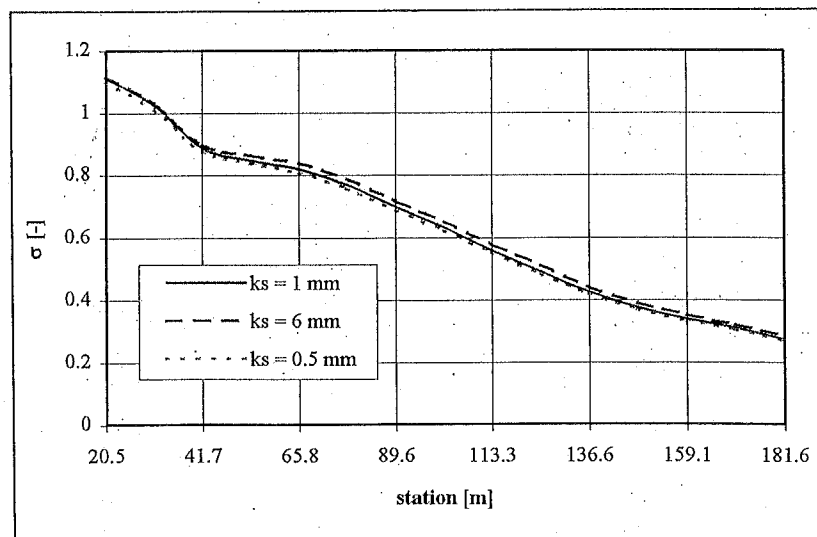


fig.5. - Cavitation index of the flow for different values of uniform roughness k_s (actual situation; $Q = 1500$ m³/s)

Both cavitation index σ and damage potential D_p , calculated for six types of surface irregularities, never reach the critical value significant of major damage at any point in the channel.

Moreover, the turbulent layer is not fully developed and the flow is not self aerated.

Nevertheless, in some points of the spillway chute the velocities reach values greater than 20 m/s and the Froude Number can be larger than 4. In addition, the dentated flip bucket at the end of the spillway chute needs a well-aerated flow in order to avoid the risk of damages. According with the present analysis, the modification of the spillway may not negatively affect the cavitation risk on the structure.

3. Experimental tests on the hydraulic model

3.1. Model description

In order to ascertain the hydraulic feasibility of the planned modification works on the spillway and identify the most suitable layout, it was felt necessary to carry out a model study.

The main purposes of the experimental study may therefore be summarised as follows :

- Quantify the influence of the approach conditions on the discharge characteristics of the spillway weir.
- Evaluate the flow patterns in the spillway chute and examine the risk of lateral overtopping.
- Determinate the velocity profiles on the upstream dam face area close to the spillway inlet.

According to the previous requirements, the model included the spillway inlet, the overflow weir and the spillway chute at a geometric scale of 1 : 60. The upstream dam face located close to the spillway inlet was equally modelled as shown in figure 6. All parts reproducing concrete structures were carried out in the model by PVC elements.

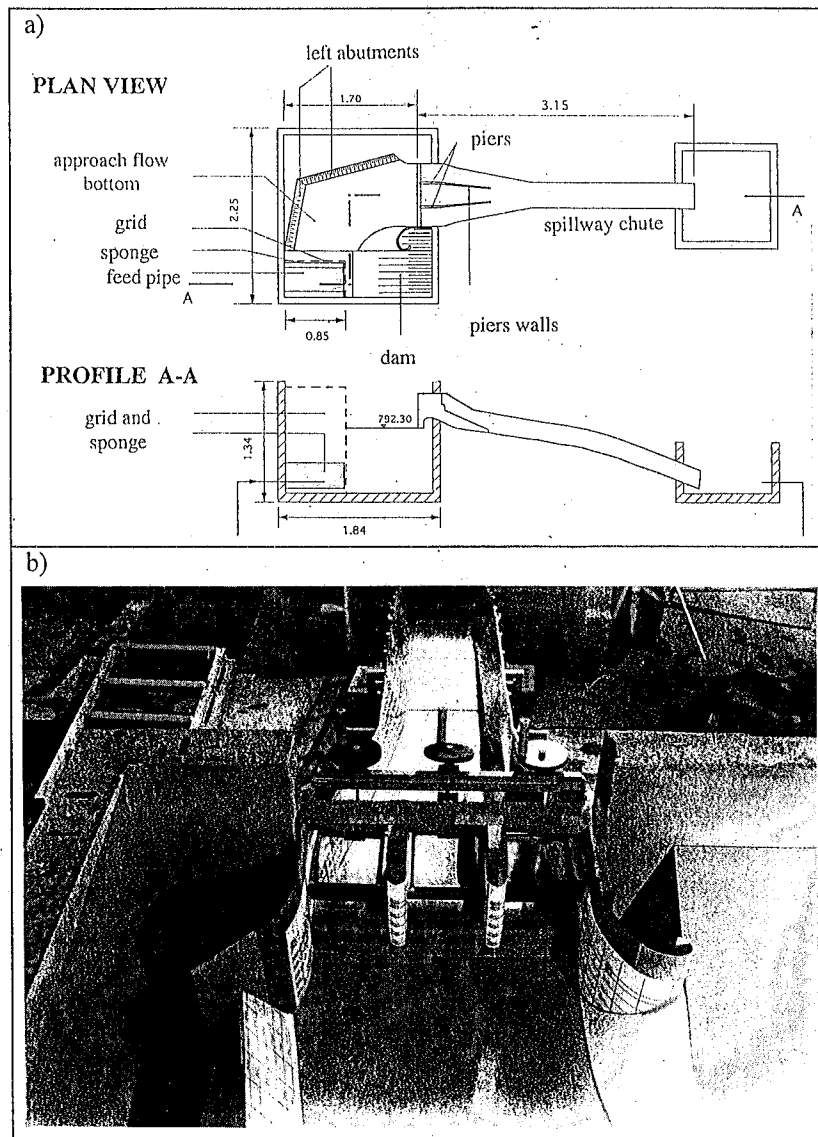


fig.6. - a) General model scheme; b) Upstream view of the spillway weir with tainter gates.

Free surface flows similarity only happens when the load losses are reproduced at the same scale as that of the model. In order to respect the load losses similarity (referring to the Darcy-Weisbach and Colebrook-White

relations) both model and prototype must have the same friction factor. The roughness of the PVC model is difficult to be adapted in order to meet the previous requirement. Therefore, for each configuration the Reynolds number and the relative roughness k_s/D were calculated for the model and for the prototype at different discharge values in order to determine the friction factor in each case. The results show a certain difference between model and prototype but the model is rougher than the prototype. This fact put the model on the safety side when considering the flow profiles in the chute. Obviously, the air entrainment will increase the water depth on the prototype.

For each spillway configuration, the following hydraulic characteristics were determined : approach velocity distribution; head-discharge relations for the free overflow, for outflow controlled by gates and for various gate opening combinations; flow profiles through the bays and along the chute axis and the lateral walls; flow profiles and velocities in some critical cross sections of the chute.

3.2. Approach flow

In order to investigate the approach flow distribution, velocity measurements were made in the reservoir upstream of the spillway inlet for a 4000 m³/s discharge. At 40 m from the gates, the velocity field is highly asymmetric with a dominant stream on the left side. This asymmetry vanishes progressively at the approach of the inlet and is quite homogeneous with velocities of about 4 m/s at a distance of 15 m. Measurements made on the upstream dam face show a well-graded velocity distribution (between 1.5 and 2.5 m/s) inversely proportional to the water depth. At the interface of the dam slope and the right lateral pier, a significant flow velocity increase was observed. The maximum measured values reach 6.3 m/s, on the slope bottom.

3.3. Hydraulic capacity (gated and ungated flows)

The results obtained for ungated flows are shown in figure 7.

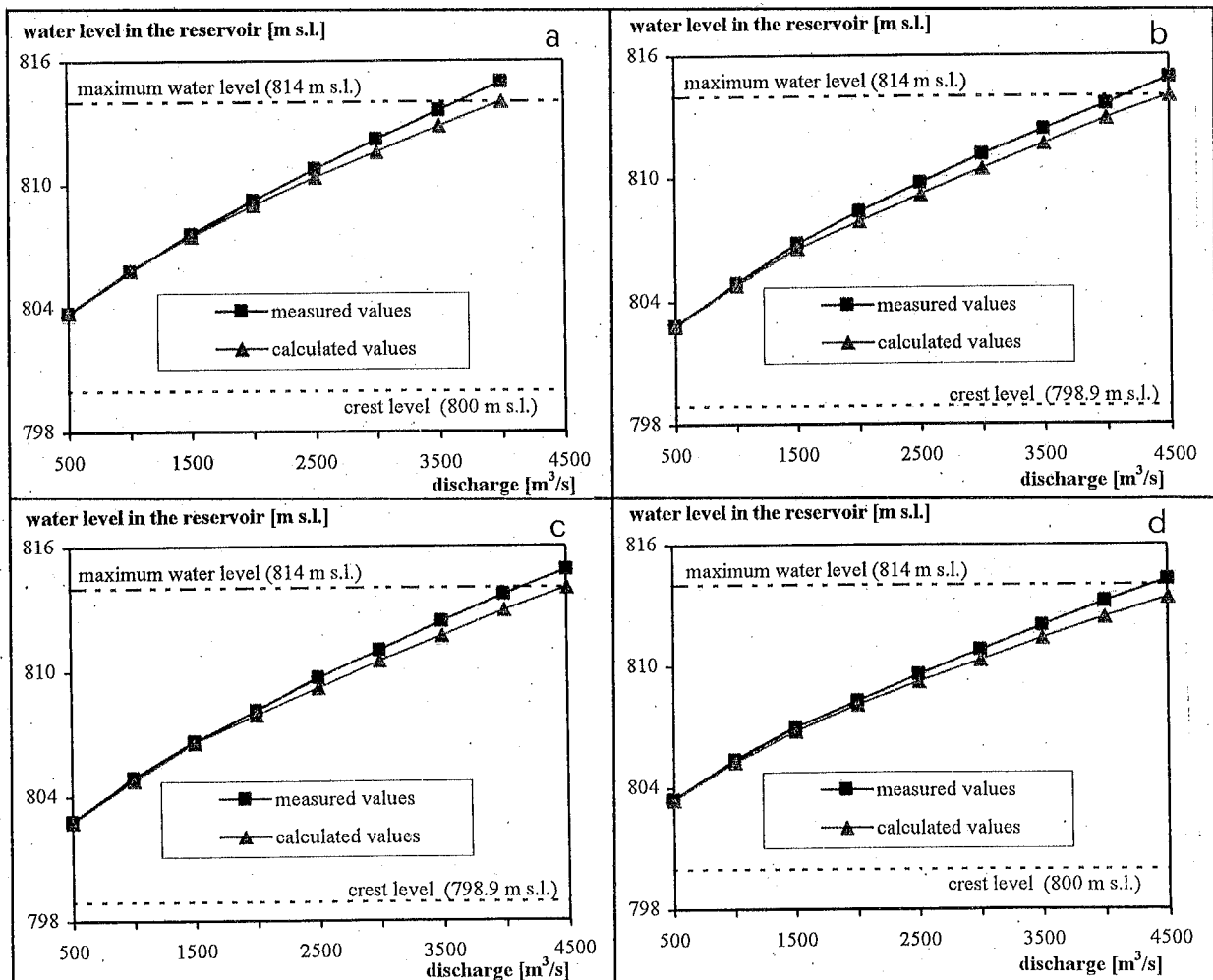


fig.7. - Comparison between calculated and measured values of the outflow capacity : a) actual situation; b) variant B1; c) variant B2; d) variant C.

The difference between the calculated and measured outflow capacity curves may be explained by the fact that in the theoretical approach the abutment contraction coefficient was probably underestimated ($k_s = 0.04$). It could be observed on the model that the right abutment curvature influences significantly the outflow behaviour : the standing waves caused by the wall itself are bigger than those caused by the central piers. Furthermore, close to the right abutment the flow no longer follows the wall profile, showing a relevant flow contraction.

3.4. Flow patterns on the chute

Water level measurements on the chute correspond to average values, with no indication of the turbulent fluctuations of the flow.

A first set of measures was dedicated to the water line profile on the chute walls and along the central axis. According to the results obtained with various discharge, an example of which is given in figure 8, the following main aspects may be pointed out:

- generally, a fairly good agreement exists between the calculated (using USBR program [4]) and the measured average flow profile (figure 8a);
- for low discharges ($1500 \text{ m}^3/\text{s}$) the flow profile measured is fairly symmetric. By increasing the discharge, significant differences are observed between the two lateral walls. At extreme values of $4000 - 4500 \text{ m}^3/\text{s}$ (figure 8b), this phenomenon leads to intermittent water overtopping by the right wall of the channel.

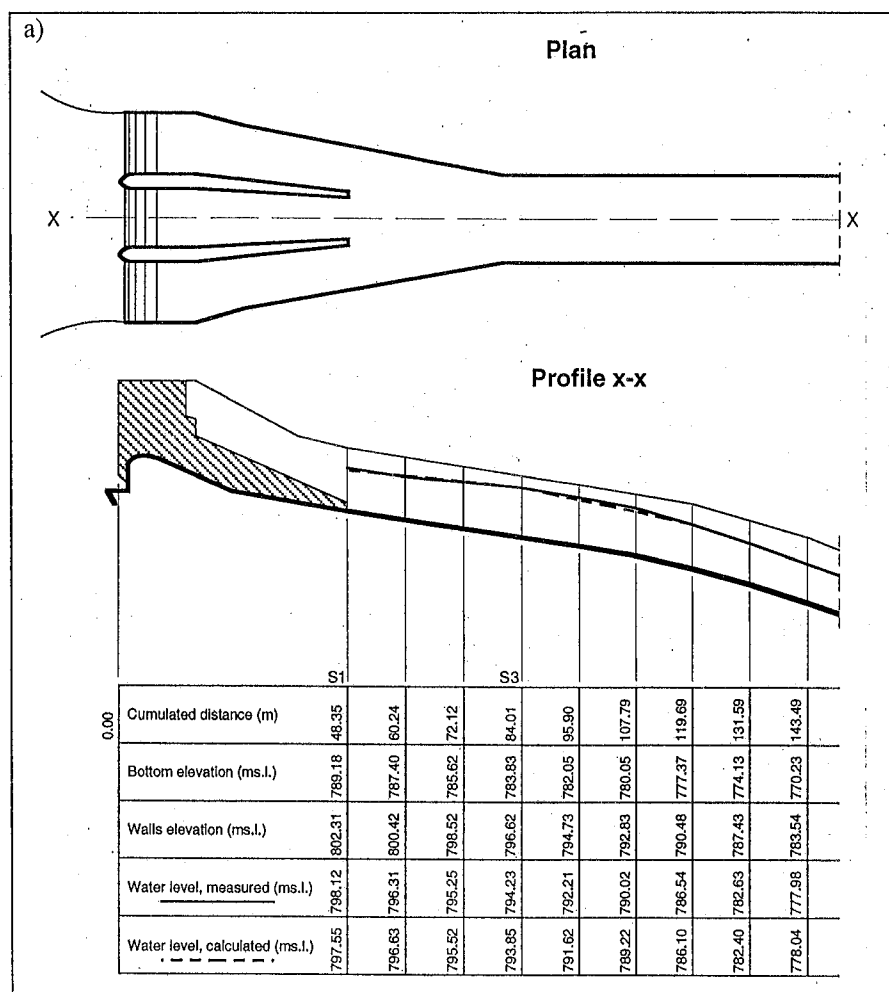


fig.8.a. - Flow profiles on the chute axis (Variant B1; $Q = 4000 \text{ m}^3/\text{s}$) : comparison between the calculated and the measured average profiles.

In order to describe the flow configuration on the chute, a second set of measurements have been made in some characteristic cross-sections. At low discharges, it was observed, that the velocity profile is rather symmetric and homogeneous in each section.

The velocity profile at the measured cross sections becomes increasingly irregular when the discharge is raised. Figure 9.a. shows the results obtained for two cross sections in the existing design for various discharges.

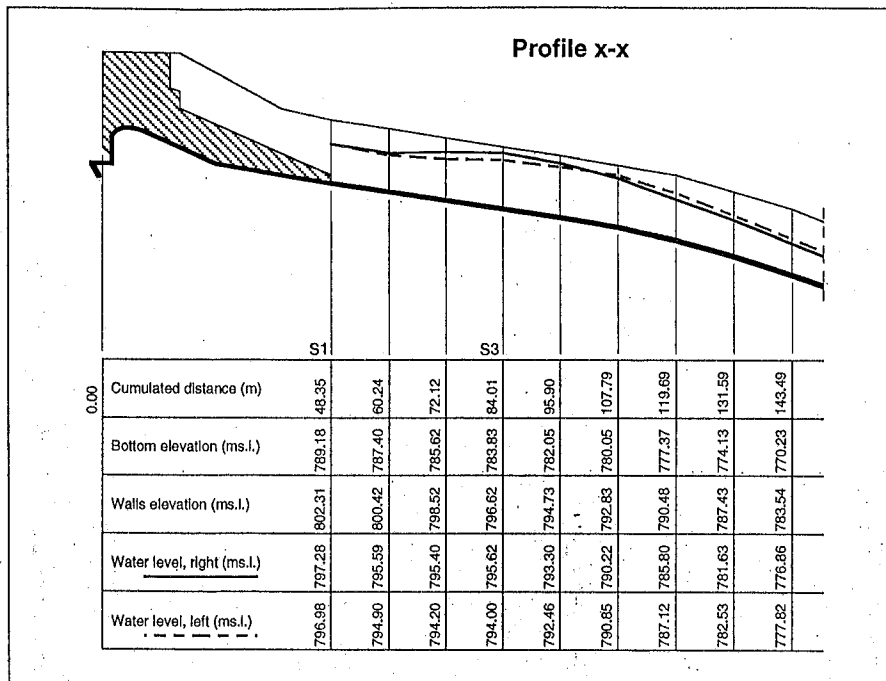


fig.8.b. - Flow profiles on the chute (Variant B1; $Q = 4000 \text{ m}^3/\text{s}$): comparison between measured values on the right and left side walls of the chute.

Just downstream of the spillway, the outflow from the right bay enters the chute a bit before the outflow from the left bay. This difference, due to the asymmetrical geometry of the inflow, amplifies the shock waves meeting the lateral walls at different points along the chute. This phenomenon causes the water overflow by the right wall at section S3 (figure 8b).

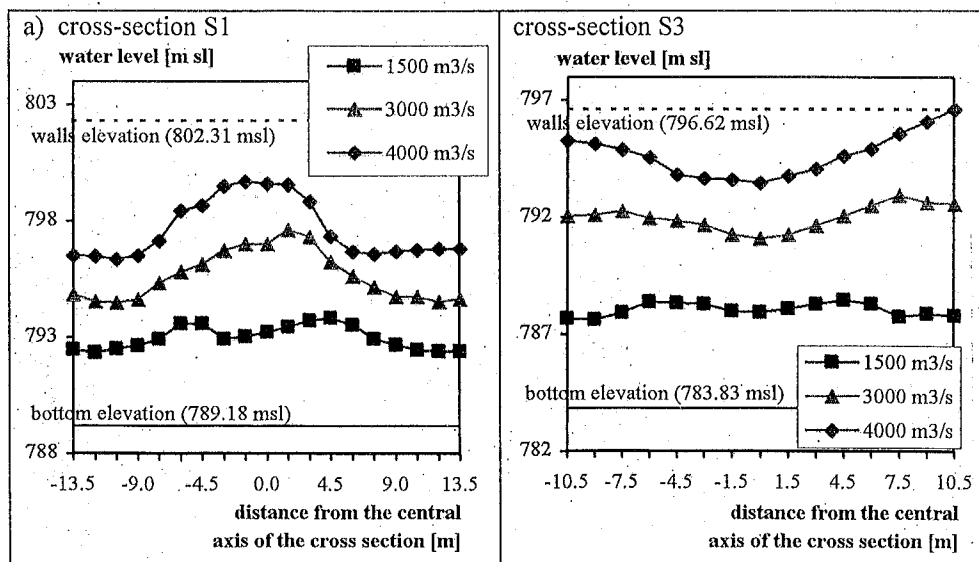


fig.9.a. - Flow profile in two different cross sections of the chute for three different discharge values before modification of the central piers (actual situation)

In order to solve the problem of overflow, new pier's geometries were tested on the model for the actual configuration and for the B variants, increasing and decreasing the relative distance between the wall themselves, both symmetrically and asymmetrically. These attempts were unsuccessful in stopping the overflow from occurring.

Another attempt was made by progressive increase of the downstream height of the left central pier, in order to improve the convergence of the outflows on the centre line. The goal was reached by raising the left pier

extension by 2.4 m (solution A in figure 10). With this modified configuration, the flow symmetry and the water profile fluctuations decrease considerably.

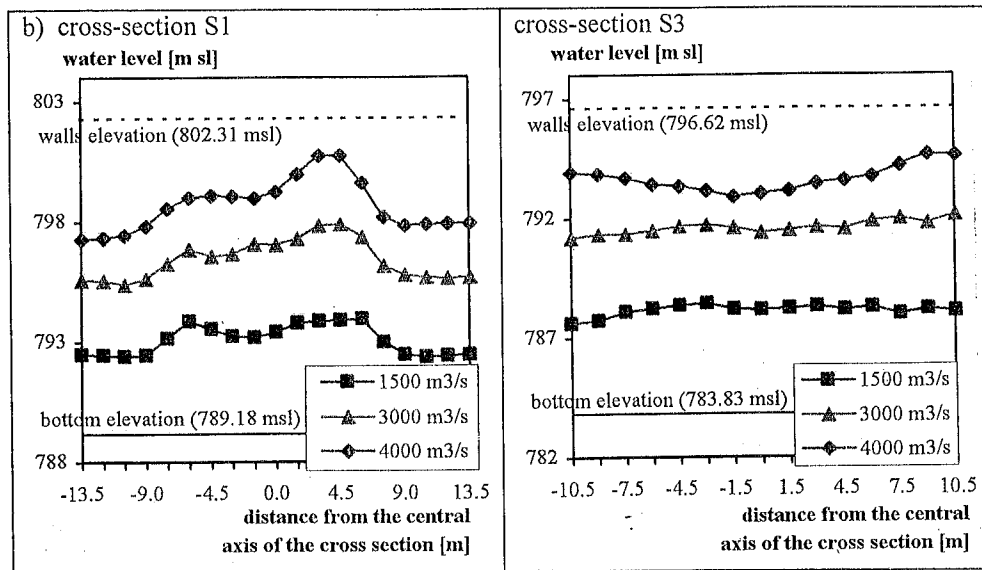


fig.9.b. - Water levels in two different cross sections of the chute for three different discharge values after modification of the piers walls (actual situation, solutions A and B)

A simpler solution could be found by modifying the tops of the central piers walls, as shown in figure 10 (solution B). The result obtained is similar to that of solution A (figure 11).

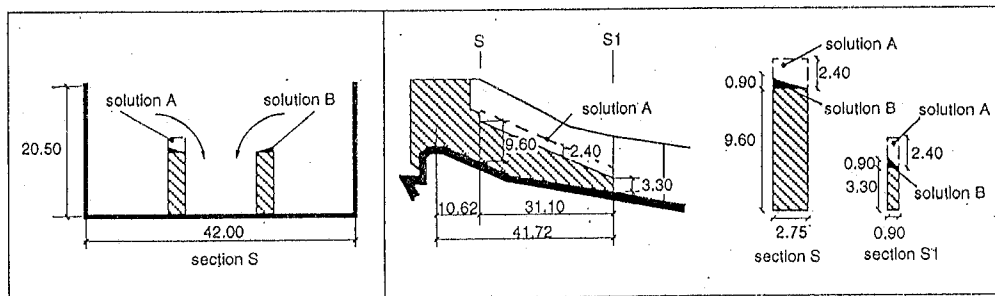


fig.10. - Modification of the piers' downstream extensions in order to eliminate the water overflow at the right wall of the chute (solution A: left extension raised by 2.4 m; solution B: lateral inclination of the tops of the downstream pier extensions)

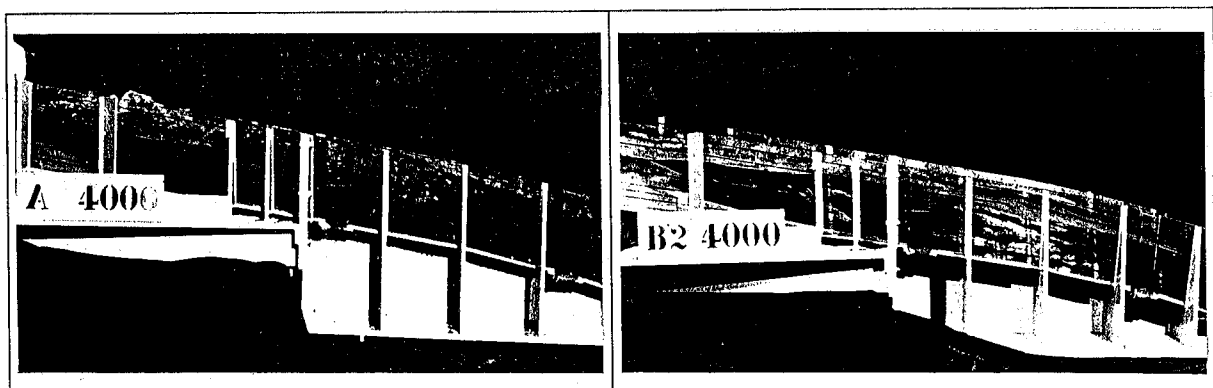


fig.11.- Overflow at the right chute wall (actual situation): a) $Q=4000 \text{ m}^3/\text{s}$ before modification of the piers' extensions; b) $Q=4000 \text{ m}^3/\text{s}$ after modification of the piers' extensions (solutions A and B).

5. Conclusions

The hydraulic analysis of the actual spillway of the Pueblo-Viejo dam and of some design alternatives allowed the detailed investigation of the general hydraulic behaviour of the spillway. The emphasis was placed on the increase of the outflow capacity, on the improvement of the flow conditions in the structure, and on the assessment of cavitation risk in the chute.

The lowering of the weir crest and the optimisation of the piers' geometry (variants B1 and B2) showed the possibility of increasing the spillway capacity with limited modification of the present spillway. As model testing cannot be used to predict the cavitation risk, a theoretical approach was undertaken to compute the cavitation characteristics of the chute. In all cases, it was concluded that the cavitation risk is low both for the actual and the modified spillway geometries.

The flow behaviour in the chute was investigated under steady flow conditions. The narrowing of the channel and the piers produce shock waves which increase with discharge. Asymmetric inflow conditions accentuates these effects at the right side of the chute. For the maximum discharge, overflow of the right side wall can be observed. Three different solutions have been tested to solve this problem. All of them consider modifying the downstream piers' extension walls with no channel modification. A particularly simple effective solution could be found by modifying the shape of their tops.

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Biographical details of the authors

J. L. Boillat was awarded a MSc in Civil Engineering at the Swiss Federal Institute of Technology Zurich, Switzerland, in 1972. In 1980 he obtained a PhD in Civil Engineering from the Swiss Federal Institute of Technology Lausanne, Switzerland. Between 1972 and 1989 he worked as a Consulting Engineering for hydraulic construction projects as well as an independent Resident Engineer involved in the design and construction in civil engineering. Then he was engaged as a Consulting Engineer by the City Council of Lausanne and charged with urban storm drainage and sewerage systems management as well as a responsible for water treatment plant and environmental protection. Since 1989 he has been lecturing regularly on hydraulics at the Swiss Federal Institute of Technology Lausanne also supervising and managing numerous studies in the field of hydrology, hydraulics and hydraulic structures.

He has published and presented at International Congresses 30 technical papers devoted mainly to studies in hydraulics. He is also the author of 70 reports on different hydraulic studies and model tests carried out by the Laboratory of Hydraulic Construction.

R. Bremen was awarded a MSc in Civil Engineering at the Swiss Federal Institute of Technology Lausanne, Switzerland, in 1987. In 1990 he obtained a PhD in Civil Engineering from the same institute. Since 1990 he is working at Lombardi Engineering Ltd in Minusio, Switzerland. He is involved in the design and rehabilitation of dams and hydroelectric plants in Europe, Central and South America.

V. Feci graduated in Hydraulic Engineering from the University of Florence (Italy) in September 1995. In the context of the exchange program between European Universities, as Erasmus student, she has made her final work at the Swiss Federal Institute of Technology Lausanne, Switzerland. From November 1995, she is student in Master in Energy at Swiss Federal Institute of Technology Lausanne, Switzerland. At the same time, she works as civil engineer at Laboratory of Hydraulic Structures, Swiss Federal Institute of Technology Lausanne.