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Unconventional Interpretation of Local Scour Downstream a Large Dam Stilling Basin

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ABSTRACT

The stilling basin of Rio Hondo Dam, Argentina, constructed fifty years ago, was designed to contain entirely the hydraulic jump and it was verified by means of hydraulic model tests.

The sediment discontinuity produced by the dam generates a progressive degradation of the river bed elevation downstream the structure, decreasing the bed level 3 m below the original bed. The hydrodynamic study developed shows for flow discharges between 500 m^3/s and 2248 m^3/s (maximum discharge), the Froude Number over the end sill is practically F1 = 1. This condition indicates accelerated flow at the end of the basin.

The local scour downstream this structure was analyzed: a) as a case of scour downstream ski-jump with important energy losses in the way, b) assuming the scour below an horizontal flow jet, and c) as a conventional scour downstream a conventional hydraulic jump stilling basin.

The performance of each one of the eight empirical equations used is considered for the comparison with prototype scour measurements for discharge Q = 1400 m^3/s. All the equations are applied to all discharges, reaching the maximum discharge.

A mixed protection solution (structural and non structural) was proposed. It needs the execution of diverse works and a continuous bed river behavior monitoring.

INTRODUCTION

Río Hondo Dam is located on the Dulce River, in the northern region of Argentina and it has different goals, as flood attenuation, irrigation and hydroelectric power supply.

The discharge structures of the dam are composed by a spillway, two outlet works and four valves for water derivation. The spillway has a free length L = 151 m and design head H = 3m, allowing a maximum spillway discharge of 1,525 m^3/s. The maximum discharge flowing by all the outlet devices is Q = 2,248 m^3/s. The spillway and outlet structures discharge in an horizontal stilling basin with 80 m length, topographic level +245.92, upstream wide 166.5 m and downstream wide 182.5 m. It has inside a line of baffle piers and it finish with a stepped end sill, with topographic top level +249 m.

The dam began to work in the sixties. The “wall” produced an abrupt discontinuity in sediment transport in the river, starting a typical process of “erosion downstream dams in fluvial channels”, developed along several kilometers as a progressive and permanent phenomena, searching in an asymptotic form a new riverbed profile. In the proximity of the stilling basin the bed level decreased from +250 m to less than + 247 m (Figure 1).

Due to this process, the hydrodynamic conditions are extremely modified downstream the stilling basin, and the erosion effect on the riverbed is more severe than the designers considered long time ago. The hydraulic jump cannot be located into the concrete structure and a jet jumps on the end sill to the riverbed, changing the conditions of local scour.

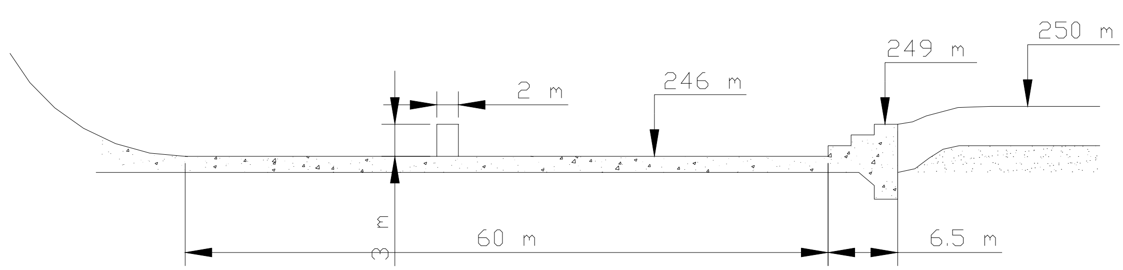

Figure 1: Río Hondo Dam stilling basin

HYDRAULIC ASPECTS

The first verification consisted on the spillway acting alone, with all other outlet devices closed. Then, it is possible to obtain the following data: Q = 1550 m^3/s, Reservoir topographic level: H_0 = +275 m, stilling basin topographic level H_p = +245.92 m, stilling basin wide at the spillway toe b = 166.5 m, and inflow specific discharge to the stilling basin: q = 9.31 m^2/s.

Applying the energy conservation and continuity principles it is possible to estimate the inflow water depth h_1 = 0.44 m, and the inflow velocity U_1 = 21.16 m/s, giving an incident Froude Number F_1 = 10.3.

The length of the stilling basin is L_c = 80 m, with baffle piers and end sill. This stilling basin was designed for hydraulic jump location and energy dissipation.

For this purpose, the flow must change from supercritical to sub critical in the structure, avoiding the macrotubulent transition on the riverbed because the possibility of severe local scour.

With the calculated values it is possible to estimate the “sequent depth” of a free hydraulic jump h_2 = 6.2 m. With the addition of this value up to the stilling basin bed level the downstream water level needed for the hydraulic jump stable condition will be +252.12. When the riverbed level was +250 m (at the beginning of the dam operation) the downstream level was clearly over that value.

According with Smetana, the length of hydraulic jump can be estimated by L = 6 (h_2 - h_1) = 34.56 m, that implies L/ Lc = 0.43. However, local scour was detected downstream the structure.

The maximum measured discharge through the spillway along forty years of operation was Q = 1400 m^3/s, with a downstream level of +250.8 and it occurs recently.

The maximum local scour at the bed reaches the level + 245.5 m. the topographic level of the top of the end sill is + 249 m, the stilling basing wide at the end sill location is B = 182.5 m, the specific discharge over the end sill is q = 7.67 m^2/s and the mean velocity over the end sill was U_d = 4.26 m/s.

For different upstream reservoir levels (Hups) and the downstream water levels (Hdown) calculated by means of the HEC-RAS model it is possible to calculate the hydrodynamic conditions over the end sill for discharges between Q = 500 m^3/s and Q = 2248 m^3/s (Table 1).

In Table 1 it was also calculated the specific inflow discharge "q", the specific discharge over the end sill "q_d", the velocity "U_d", the water depth over the end sill "h_d" y and the Froude Number in this section "F_d".

TABLE 1: HYDRODYNAMIC CONDITIONS OVER THE END SILL

Hups (m)	Hdown (m)	Q (m3/s)	q (m2/s)	qd (m2/s)	Ud (m/s)	hd (m)	Fd
273.41	249.90	500	3.00	2.74	3.04	0.90	1.02
273.59	250.05	600	3.60	3.29	3.13	1.05	0.98
273.77	250.13	700	4.20	3.84	3.39	1.13	1.02
273.93	250.25	800	4.80	4.38	3.51	1.25	1.00
274.09	250.37	900	5.41	4.93	3.60	1.37	0.98
274.24	250.45	1000	6.01	5.48	3.78	1.45	1.00
274.53	250.65	1200	7.21	6.58	3.99	1.65	0.99
274.80	250.82	1400	8.41	7.67	4.21	1.82	1.00
275.00	250.93	1550	9.31	8.49	4.40	1.93	1.01
275.00	251.13	1800	10.81	9.86	4.63	2.13	1.01
275.00	251.27	2000	12.01	10.96	4.83	2.27	1.02
275.00	251.50	2248	13.50	12.32	4.93	2.50	0.99

The condition $F_d = 1$ implies that in this section a critical flow is produced and it will be considered as a control section (hydraulic upstream conditions are independent from downstream conditions). Downstream the end sill the flow must to be accelerated, with severe instability, waves formation and a new jump process over the riverbed. This phenomena contributes on the local scour process downstream the structure.

LOCAL SCOUR DOWNSTREAM SKI JUMP STRUCUTRES

The maximum depth of scour "y" , measured from the tailwater level, (Figure 2) is a function of the water density ρ, the viscosity μ, the grain representative diameter d_s of the riverbed material, the grain density ρ_s, the specific discharge "q", the fall distance between the reservoir level to the tailwater level ΔH, the tailwater depth h_r, the submerged gravity g (s - 1), where $s = \rho_s/\rho$, and the angle α of the jet.

$$y/\Delta H = F\left(q/(g\ \Delta H^3)^{0.5},\ h_r/\Delta H,\ d_s/h_r,\ \alpha\right),$$

and if Z* is called the "fall Number" $Z^* = q/(g\ \Delta H^3)^{0.5}$, the expression can be reduced to:

$$y/\Delta H = F\left(Z^*,\ h_r/\Delta H,\ d_s/h_r,\ \alpha\right). \qquad [1]$$

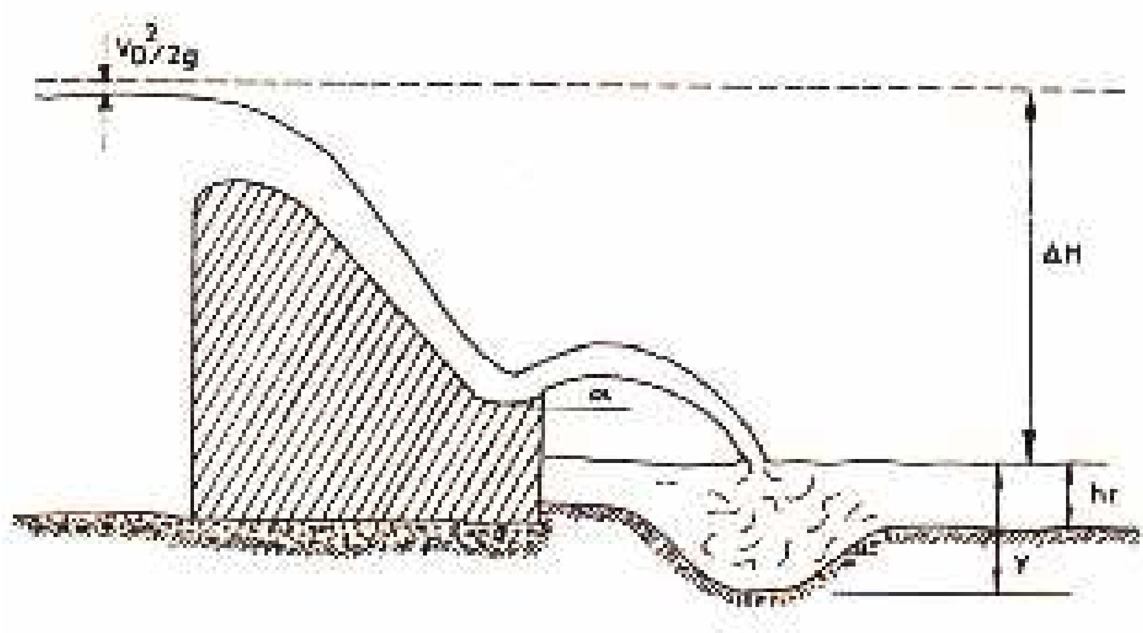

Figure 2: Ski-jump local scour notation

There are a lot of equations proposed by several authors (1) with the objective to give an experimental relationship between the variables involved in [1] with more or less complexity and simplification. An intentionally oversimplified formula for the estimation of the maximum depth of scour downstream ski-jump spillways, including only y, q, g y ΔH, or in other words the parameters $y/\Delta H$ and Z^*, is known as the INCYTH equation (2), published in 1983, in order to be of use as an initial estimation.

This equation can be expressed as:

$$y/\Delta H = K\, Z^{*0.5}\ , \qquad [2]$$

and it is it has a mean value of K = 2.5 and a maximum value for K = 3.25 (3).

The equation was developed by means of laboratory tests carried out by the authors and sixty six laboratory experimental results of other authors (with standard deviation of 18%) and seventeen prototype results (with standard deviation of 26%).

The INCYTH equation requires only the knowledge of the unit discharge and the fall height from the reservoir level to tailwater level. It should be noted that during the design stage many parameters are usually unknown, such as the size of blocks formed by fracture of the rock at different depths near the jet impact. For prototype preliminary calculations, this equation demonstrates acceptable performance, taking into account prototype results obtained from other authors, as the published data of Colbún (4), Tarbela (5), Cabora Bassa (6) and several dams in China (7).

LOCAL SCOUR DOWNSTREAM STILLING BASINS

The hydraulic jump is usually designed to be on a stilling basin with concrete floor, in order to avoid severe local scour of the riverbed downstream the spillway. Nevertheless, there are numerous references showing erosion and damages downstream hydraulic jump stilling basins. The macroturbulent flow generated by the jump has influence on this process.

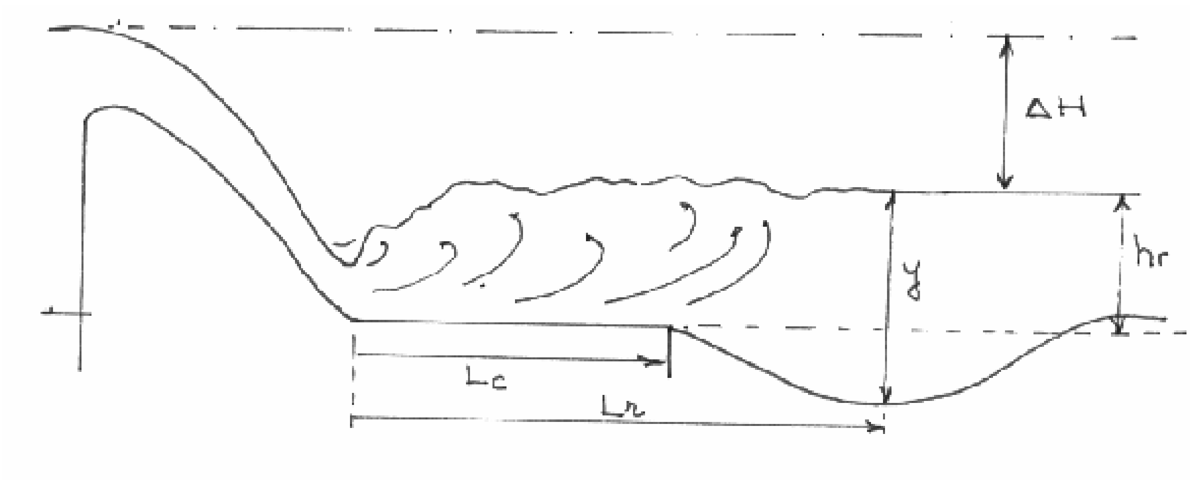

Figure 4: Local scour downstream hydraulic jump stilling basin

As the local scour downstream ski-jump spillways, it is possible to estimate the maximum depth of scour "y" (Figure Nº 4) by means of:

$$y/\Delta H = F\ (F_1,\ h_r/\Delta H,\ d_s/h_r,\ L_c/L_r),\quad [4]$$

where L_r is the length of the jump and y L_c is the length of the stilling basin.

In a first analysis, the case of a very short concrete bed is considered (L_c = 0). Empiric expression were proposed for this configuration by Schoklitsch in 1935 and Veronese in 1937, both without dimensional accord. Jaeger proposed an adaptation of Veronese's expression:

$$y/\Delta H = 10.62\ (q/(g\ \Delta H^3)^{0.5}\ )^{0.5}\ (h_r/d_{90})^{0.33}\ ,\qquad [6]$$

On the other hand , Eggenber y Müller adapted Schoklitsch's formula:

$$y/\Delta H = 45.39\ (q/(g\ \Delta H^3)^{\ 0.5})^{0.6}\ (\Delta H/d_{90})^{0.4}\ .\qquad [7]$$

There are more up to date equations in the same direction, as the proposed by Kotoulas:

$$y = 1{,}9\ g^{-0{,}35}\ \Delta H^{0{,}35}\ q^{0{,}7}\ d_{95}^{\ -0{,}4}\ ,\qquad [8]$$

where d_{95} is the maximum diameter of the rock protection.

Altinbilek and Basmaci (8) proposed for the flow downstream a sluice gate with contracted depth h_c the equation:

$$y/h_c = (tg\Phi/(d_s/h_c))\ (F_c/(s-1)^{0.5})\ , \qquad [9]$$

where Φ is the angle of the bed material and F_c the Froude Number calculated with the velocity and h_c.

Franke (9) for the same configuration proposed:

$$y/\Delta H = 1.21\ Z^{*0.3}\ (d_{90}/\Delta H)^{-0.4}\ . \qquad [10']$$

The experience of China in the subject (10) can be summarized in the following formula:

$$y = 1{,}25\ (q^2/g)^{1/3}\ \Delta H^{0{,}26}\ d_{95}^{-0{,}22}\ h_r^{-0.04}\ . \qquad [11]$$

Taking into account the case of hydraulic jump stilling basins with length not less than the jump length, there are in the literature other empirical equations. One of these expressions was proposed long time ago by Schoklitsch (11):

$$y = 4.5\ \alpha\ ß\ (\Sigma b/B)^{1/4}\ z^{1/6}\ H^{1/2}\ q^{1/3} + 2.15\ a\ , \quad [12]$$

where, Σb is the total free wide of the spillway, B is the river wide at the downstream section, z is the time period (in hours) of the flow, q is the specific discharge, α is a coefficient which depends of the stilling basin shape (for example α = 0.36 for a horizontal floor and $0.17 < \alpha < 0.25$ for a stilling basin with a Rehbock end sill), ß is other coefficient which take into account the flow dissymmetry and "a" is the difference between the level of the top of the end sill and the level of the river bottom.

Mostly of the literature on the subject includes the influence of the riverbed material size. This is the case of the equation proposed by Catakli et Al (12) for a stilling basin with length $L_c = 5\ h_2$.

$$y = K\ q^{0.6}\ (\Delta H + h_r)^{0.2}\ d_{90}^{-0.1}\ , \qquad [13]$$

where d_{90} is expressed in mm, K is a shape factor (K = 1.62 when the basin has not end sill and K = 1.42 when it has end sill with height of 10% to 12% of the hydraulic jump sequent depth.

As it is considered by Breusers and Raudkivi (13) there are not general expressions wit capacity for maximum local scour prediction downstream stilling basins. They mentioned two equations for estimated calculations: one proposed by Dietz and other by Blench. Dietz ,in 1969, proposed the following formula:

$$(y - h_r)/h_r = (U_{max} - U_c)/\ U_c\ . \qquad [15]$$

where U_{max} is the mean velocity of the flow downstream the stilling basin and U_c is the critical velocity of sediment drag, which is function of the grain diameter, for example by means of the Neill equation:

$$U_c^2 = 2\ g\ (s - 1)\ d\ (h_r/d)^{1/3}.$$

On the other hand Blench, in 1957, based in the "regime theory" proposed an equation for estimation of the maximum depth of scour downstream hydraulic jump stilling basins. This equation is

$$y_{max} = (0,75 \text{ a } 1,25)\ y_{2r} + h_r \qquad [16]$$

where y_{2r} is el the "regime depth" for two-dimensional flow: $y_{2r} = 1,34\ (q^2/f)$, and "f" is the "sediment factor" , which can be calculated as: $f = 1,76\ [d(mm)]^{1/2}$.

Briefly, among the different factors capable to generate a severe local scour downstream a hydraulic jump stilling basin (14) two of them can be specially mentioned: an uncalculated decrease of the tailwater level and an eventual deficiency of symmetry of the spillway discharge distribution.

LOCAL SCOUR FOR THE MAXIMUM MEASURED DISCHARGE

As it was mentioned above, the maximum flow discharge through the spillway of the Río Hondo Dam was Q = 1,400 m^3/s. For that flow condition the reservoir water level was + 274.8 and the tailwater level was +250.8 m (tailwater depth h_r = 3,8 m) over a riverbed with a main bottom level + 247 m. Therefore, the total head was ΔH = 24 m y and the specific discharge over the end sill was q = 7,67 m^2/s. The measured maximum depth of scour for long time of operation was y = 5,53 m, reaching a topographic level + 245.5 m.

Verification as ski-jump condition

As the stilling basin is not capable to have the hydraulic jump into its dimensions, because the tailwater level is not enough, the flow is accelerated again and the contact with the river bottom is rather similar than a ski-jump spillway. It can be considered as a first approximation the use of this hypothesis for a maximum local scour depth. This final depth of scour can be estimated by the INCYTH expression [2], where K = 2.5, giving the value y = 9.06 m, reaching the topographic level + 241.76 m.

There are several objections for the direct application of this expression, even for ski-jump usual cases. The difference between experimental data and the use of this formula for the Río Hondo Dam local scour is obviously due to these objections Firstly, the equation is valid for equilibrium scour depth, and the time required to attain this depth in the nature is extremely large. Secondly, and more essential objection in this case, between the reservoir water level and the tailwater level there are very important energy losses (baffle piers action, end sill action and friction losses) and they were not considered in the calculation. Finally, the equation [2] don't take into account the river bottom material size.

Verification as hydraulic jump stilling basin condition

With this hypothesis of configuration the hydraulic jump length don't exceeds the stilling basin length and the flow downstream the structure must be in fluvial regime. The riverbed material is granular and composed by silt and fine sand.

The Schoklitsch empirical equation [12] is applied to the Río Hondo maximum discharge measured. With $B = 182.5$ m, $z = 1$, $q_d = 7.67$ m^2/s, $\alpha = 0.17$, $ß = 1$ and $a = 1$ m, the maximum depth of scour calculated is $y = 9.24$ m, and minimum topographic level + 241.56 m.

For the knowledge of the sediment characteristics on the river bottom downstream the stilling basin some representative soil data were obtained. The collection was near the surface (between 0.5 and 1 m depth) at sections located between 500 m and 750 m downstream the end sill.

By means of this sediment information, the representative diameter selected was $d = (d_{15} \,.\, d_{85})^{1/2} = 1.51$ mm, and a bed armoring diameter $d_{85} = 19$ mm.

With this basic condition and the hydrodynamic data obtained for the maximum reorded flood it is possible the application of Catakli's formula [13], giving in this case $y = 6.93$ m, reaching the topographic level + 243.87 m, more than 1.5 down the experimental data (+ 245.5 m).

Applying the methodology proposed by Dietz [15], the critical velocity is $V_c = 1.89$ m/s. The maximum depth of scour calculated was $y = 8.49$ m, and this value implies a topographic level + 242.31 m. Breusers y Raudkivi (28) considered that values predicted by Dietz can be extremely large.

By means of the Blench equation the maximum depth of scour is (as a mean value) $y = 5.85$ m. As the tailwater level is +250,82 m , the erosin can reach the level +244.97m. Even considering that the hydraulic jump is contained in the stilling basin the calculated scour is more than the measured in the prototype.

The calculations of the maximum depth of scour downstream the Río hondo Dam stilling basin by the different equations previously mentioned are shown in Table 2.

TABLE 2

MAXIMUM DEPTH OF LOCAL SCOUR ESTIMATIONS DOWNSTREAM THE RÍO HONDO DAM										
Basic data			Estimation as a ski-jump spillway		Maximum depth of scour estimation downstream hydraulic jump stilling basins				Maximum depth of scour without stilling basin	
Hreservoir	Hdownstream	Q (m3/s)	yAuthors	yINCYTH	yBlench	ySchoklitsch	yDietz	yCatakli	yKotoulas	yWang
(m)	(m)	(m3/s)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)
273.41	249.58	500	3.60	5.41	2.94	7.18	4.87	5.27	9.70	12.85
273.59	249.83	600	3.72	5.92	3.50	7.48	5.23	5.88	11.01	14.43
273.77	250.08	700	3.89	6.39	4.04	7.75	5.79	6.45	12.26	15.91
273.93	250.32	800	4.09	6.83	4.55	7.99	6.17	6.98	13.44	17.31
274.09	250.46	900	4.35	7.24	4.96	8.23	6.53	7.49	14.60	18.68
274.24	250.55	1000	4.65	7.64	5.30	8.45	6.99	7.99	15.73	20.02
274.53	250.70	1200	5.23	8.38	5.94	8.86	7.73	8.92	17.91	22.57
274.80	250.82	1400	5.80	9.06	6.51	9.24	8.49	9.79	19.99	25.00
275.00	250.89	1550	6.23	9.55	6.91	9.50	9.08	10.42	21.51	26.76
275.00	251.09	1800	6.83	10.27	7.64	9.84	9.95	11.38	23.82	29.41
275.00	251.24	2000	7.29	10.81	8.19	10.08	10.67	12.11	25.58	31.43
275.00	251.50	2248	7.75	11.43	8.94	10.35	11.38	12.97	27.66	33.77

Proposal of an unconventional methodology of calculation

Table 2 summarizes the maximum depth of scour estimated for several hydrodynamic conditions (presented in Table 1) downstream Río Hondo Dam stilling basin, for application of the equations of different authors, based on the ski-jump configuration and scour downstream stilling basins hypothesis.

Even if it is usual to have a large dispersion between the results obtained by means of equations of different authors, developed for diverse hydrodynamic and river bottom conditions, Table 2 shows that the maximum depth of scour predicted as ski-jump configuration expressions and predicted with equations for erosion downstream stilling basins is in all cases bigger than the data measured in prototype.

The authors propose an unconventional methodology, based in the following idea: even if the structure is a stilling basin with baffle piers for forced energy dissipation, the critical flow over the end sill generates a new hydrodynamic condition, and the accelerated flow acts on the river bottom as a submerged jet, such as the hydrodynamic configuration of a ski-jump spillway, but with very low hydraulic head.

The proposed methodology of maximum depth of scour estimation is based in the equation [3], considered that the "ski-jump floor" level is given by the top of the end sill level, and the total head (Hups – Hdown) must to be calculated as:

$$\Delta H = h_d + V_d^2/(2\ g).$$

This equation takes into account the friction energy losses along the flow through the spillway and the stilling basin and the local energy losses due to the baffle piers and end sill.

By means of the authors proposed methodology the maximum depth of scour estimated for the maximum recorded flood was y = 5.26 m (as it is showed in Table 2), very closed with the measured value in prototype.

This methodology is proposed for maximum depth of scour downstream stilling basins when the sediment discontinuity generates an important decreas of the tailwater level and a critical flow is produced at the end of the structure. This abnormal situation is frequent in small dams in the northwest region of Argentina.

CONCLUSIONS

Due to the progressive decrease of the river bottom downstream Río Hondo Dam produced by the reservoir sedimentation, during more than forty years, the tailwater level is now below the sequent depth and the hydraulic jump cannot be contained in the stilling basin.

From discharge of 500 m^3/s to the maximum design discharge of 2248 m^3/s, the Froude Number over the end sill is practically Fd = 1, and the end sill acts as a control section. The flow downstream the stilling basin is accelerated with the obvious effect of the increasing the local scour. The structure acts as a ski-jump but with strong energy losses along the spillway and stilling basin (due to baffle piers, friction losses and end sill).

The authors propose an unconventional methodology, based in the following idea: even if the structure is a stilling basin with baffle piers for forced energy dissipation, the critical flow over the end sill generates a new hydrodynamic condition, and the accelerated flow acts on the river bottom as a submerged jet, such as the hydrodynamic configuration of a ski-jump spillway, but with very low hydraulic head.

Taking into account this study, the authors proposed two actions that can be developed in different stages.

First of all, the bed protection by means of two layers of graduated stones over a synthetic filter.

The second action can be delayed in time. The goal is to avoid the bed degradation downstream the dam, by means of the bed control with small sills constructed with local elements, increasing gradually the tailwater level to allow the hydraulic jump location into the stilling basin.

As it is showed, a mixed protection solution (structural and non structural) was proposed. It needs the execution of diverse works and a continuous bed river behavior monitoring.

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