Conference Paper, Published Version

Bollaert, Erik F. R.; Petry, B.

Application of a physics based scour prediction model to Tucurui Dam Spillway (Brazil)

Application of a physics based scour prediction model to Tucurui Dam Spillway (Brazil)

E.F.R. Bollaert* and B. Petry**
* AquaVision Engineering Ltd., Lausanne, Switzerland
** Engineering Consultant, Delft, The Netherlands

I. INTRODUCTION

This paper presents the application of a recently developed physics based model for the prediction of scour formation downstream of large dam spillways (Bollaert, 2004). The model is based on experiments with air-water jets impinging on jointed media and related numerical modeling of involved physical phenomena. It predicts ultimate scour depth and time evolution of scour formation in different types of fissured media. It is under continuous development and has been applied to several scour cases worldwide. The model includes several physics based modules for scour prediction that are based on the resistance of the medium to fissure initiation and propagation, as well as on the resistance of individual blocks to sudden ejection.

The real case example of Tucurui Dam Spillway, located on the Tocantins River in northern Brazil is presented here (Petry et al., 2002). The spillway facilities were designed to handle a design discharge of up to 100,000 cms. Model tests using non-cohesive and cohesive representations of the plunge pool were conducted during the design process to verify scouring. They resulted in the forecast of a satisfactory scour behavior. During the operation period, underwater surveys confirmed the occurrence of only minor effects caused by the recorded floods.

The 17 year long period of record (1984-2001) included several large flood occurrences but did not contain an extreme flood event. The numerical model described above was first calibrated based on the prototype observations of the plunge pool resistance and then design discharge conditions were applied to forecast the plunge pool behavior with regard to scour formation under such an extreme event. The Paper presents the details of the modeling process and provides an account of results obtained for the case of such an extreme occurrence.

Tucurui Dam Spillway is located on the Tocantins River in northern Brazil, in operation since 1984 (Petry et al., 2002) The uncommonly large dimensions of the Tucurui Spillway facilities have practically no precedence in the history of large Dams. As illustrated in Figure 1, they are characterized by: an ogee type gate-controlled surface overflow structure topped by 23 radial gates (20.75m high x 20m wide); a compact flip bucket; and a 50m deep plunge pool extending over the entire length of the structure, pre-excavated in the downstream rock. The spillway facilities were designed to handle an un-precedent design discharge of up to 110,000 cms under a gross head of 60 to 70 m. Extensive and detailed Hydraulics Laboratory model tests using different non-cohesive and cohesive representations of the plunge pool area were conducted during the design process to verify scouring. They resulted in the forecast of a satisfactory and controlled scouring behavior for a pre-excavated plunge pool at an elevation of – 40 m a.s.l. At a later stage, during the operation period, underwater surveys were conducted by the Owner to verify such scour behavior. This confirmed the occurrence of only minor effects caused by the actually recorded sequence of flood discharges.

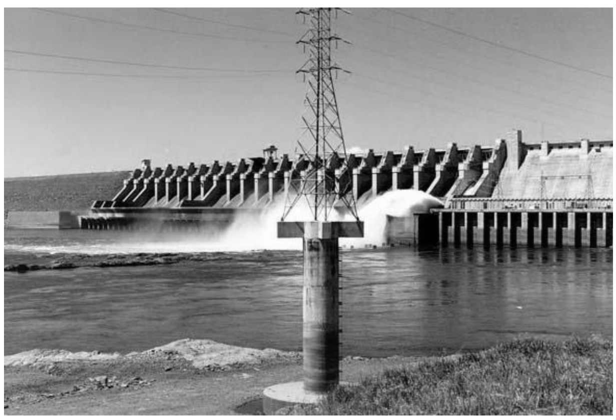

Figure 1. a) Detailed view of flip bucket spillway; b) General view of spillway and plunge pool

II. History of Flooding and Scour Formation at Tucurui Dam

Tucurui Dam has a well-described and amazing flood history. Floods and water levels of the Tocantins River have been recorded since 1969. Initially, extreme flood discharge values were determined based on a statistical analysis of the data of Tucurui and Itupiranga about 175 km upstream of the dam site. Between 1978 and 1980, however, three major flood events occurred, resulting in a redefinition and calibration of the flow rating curve at Tucurui. This allowed defining the 1980 peak flood at 68'400 m^3/s, the largest value ever recorded on the Tocantins River.

More detailed subsequent flood studies showed that the initially assumed extreme flood values for the dam spillway had been significantly underestimated. As a function of meteorological and drainage conditions, the PMF event was increased from 90'000 m^3/s to 110'000 m^3/s, in agreement with the maximum possible capacity of the spillway under construction at that time. This increase, however, would need the construction of an additional auxiliary spillway with 3 additional gates during phase 2 of the power house construction (1994).

Flood records made since dam construction have shown that the PMF value of 110'000 m^3/s is slightly conservative. For example, the maximum peak discharge recorded between 1984 and 2001 was of only 43'400 m^3/s in January 1990. Hence, in conjunction with economic considerations, it has finally been decided to not construct the auxiliary spillway and to keep the initial spillway design.

Second, scour formation in the downstream plunge pool has been described by a series of bathymetric surveys and underwater inspections between 1984 and 2001. The first inspection took place in 1984 and allowed detecting sedimentation along the right hand side of the plunge pool due to 3 years of continuous river flow through the diversion outlets at time of dam construction. The second survey took place in 1985, but had to be restricted to the most left part of the plunge pool, due to turbulent vortices in the basin.

The third survey occurred in 1986, after two years of continuous spillway operation. This showed that all former sediment deposits had been washed away and that, as predicted by the laboratory model, only two areas along the pool bottom were slightly eroded. The maximum observed scour formation was of only 5 m. It was assumed that this erosion is related to removal of partially detached rock blocks during initial spillage. These blocks were fractured and detached by blasting during dam construction and their removal does not represent any risk to the structure.

This satisfactory plunge pool performance, together with a progressive reduction of flood discharges and spillage periods, allowed reporting the fourth bathymetric survey to 1997. This survey showed erosion in the same areas than the ones detected during previous surveys. The maximum scour depth was still situated around – 45m.

Hence, as a conclusion, it may be stated that the pre-excavated plunge pool behaves like expected during dam construction. For a recorded period of 17 years, incorporating 6 flood events of more than 31'000 m^3/s and a maximum value of 43'400 m^3/s, no significant scour formation could be observed. Nevertheless, since the extreme flood event of 68'000 m^3/s during dam construction, no major flood events have been recorded.

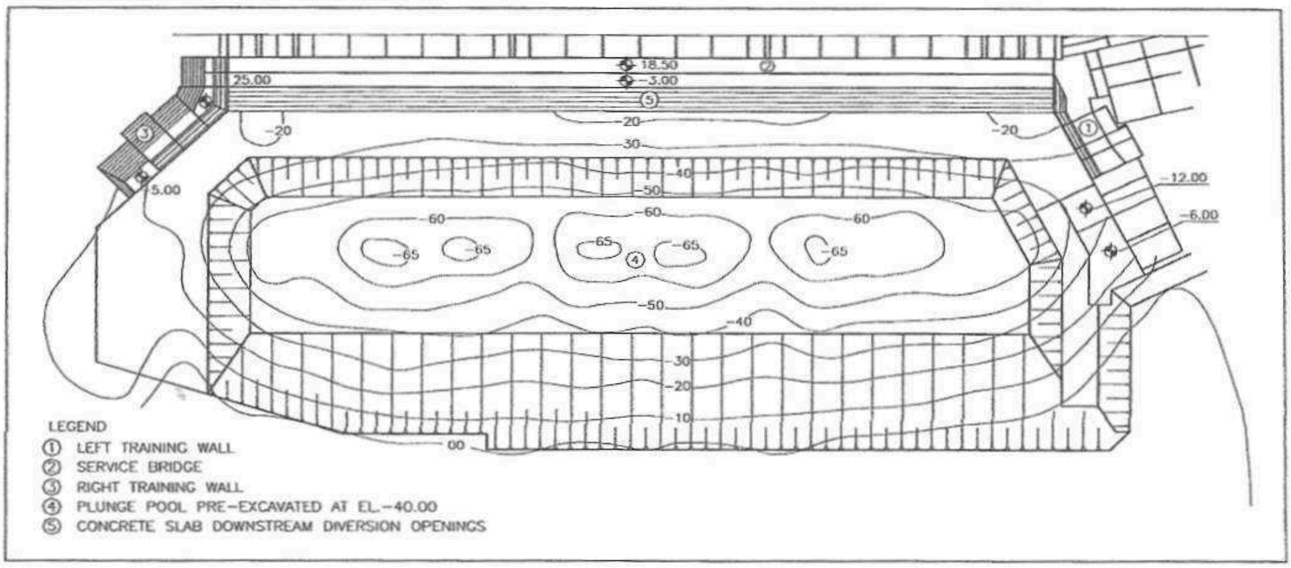

Figure 2. Scour formation in plunge pool following laboratory test with gravel and 100'000 cms (Large Brazilian Spillways, ICOLD, Petry et al. 2002)

III. Detailed Spillway and Plunge Pool Characteristics

The spillway and plunge pool of Tucurui Dam are presented in Figures 2 and 3. The spillway consists of 23 bays with radial gates of 20 m width by 20.75 m height. The spillway crest has a Creager profile. Its detailed design was based on two-and three-dimensional hydraulic model tests at a maximum scale of 1/50. The chute of the spillway ends with a compact flip bucket. The bucket has a radius of 35 m, an elevation of 30 m and an exit angle of 32°, resulting in minimum and maximum jet throw distances of 80 m respectively 130 m.

Plunge pool design has been optimized by means of hydraulic model tests. Tests have been performed for discharges between 15'000 and 100'000 m^3/s and for pool bottom elevations of -15, -25 and -40 m a.s.l. The latter pool depth has been retained because significantly reducing return currents in the modeled basin and avoiding substantial erosion for discharges of up to 50'000 m^3/s. For a design discharge of about 100'000 m^3/s, the model tests predicted scour formation down to –65m, i.e. 25 m of additional scour formation compared to the pre-excavated bottom level of the pool. This result was obtained by using non-cohesive gravel and might thus be considered as slightly conservative.

The choice of a flip bucket together with a pre-excavated plunge pool has been influenced by the very high unitary discharges (up to 228 m^2/s at maximum capacity) and by the valley's geomorphology. Also, pre-excavation of the rock allowed to re-utilize this material for the concrete of the spillway.

IV. Comprehensive Scour Model (CSM, Bollaert 2004)

A new and physics based scour prediction model has been developed at the Laboratory of Hydraulic Constructions of the Swiss Federal Institute of Technology in Lausanne, Switzerland (Bollaert, 2002 and 2004; Bollaert & Schleiss, 2005). The model uses physical laws and phenomena that have been simplified to allow its application to practical engineering projects. It is based on experimental and numerical investigations of dynamic water pressures in rock joints (Bollaert, 2002).

The model comprises two methods that describe failure of fractured rock. The first one, the Comprehensive Fracture Mechanics (CFM) method, determines the ultimate scour depth by expressing instantaneous or time-dependent joint propagation due to water pressures jacking inside the joint. The second one, the Dynamic Impulsion (DI) method, describes the ejection of rock blocks from their mass due to sudden uplift pressures.

The structure of the Comprehensive Scour Model consists of three modules: the falling jet, the plunge pool and the rock mass. The latter module implements the two aforementioned failure criteria. More details can be found in Bollaert (2004) or Bollaert & Schleiss (2005).

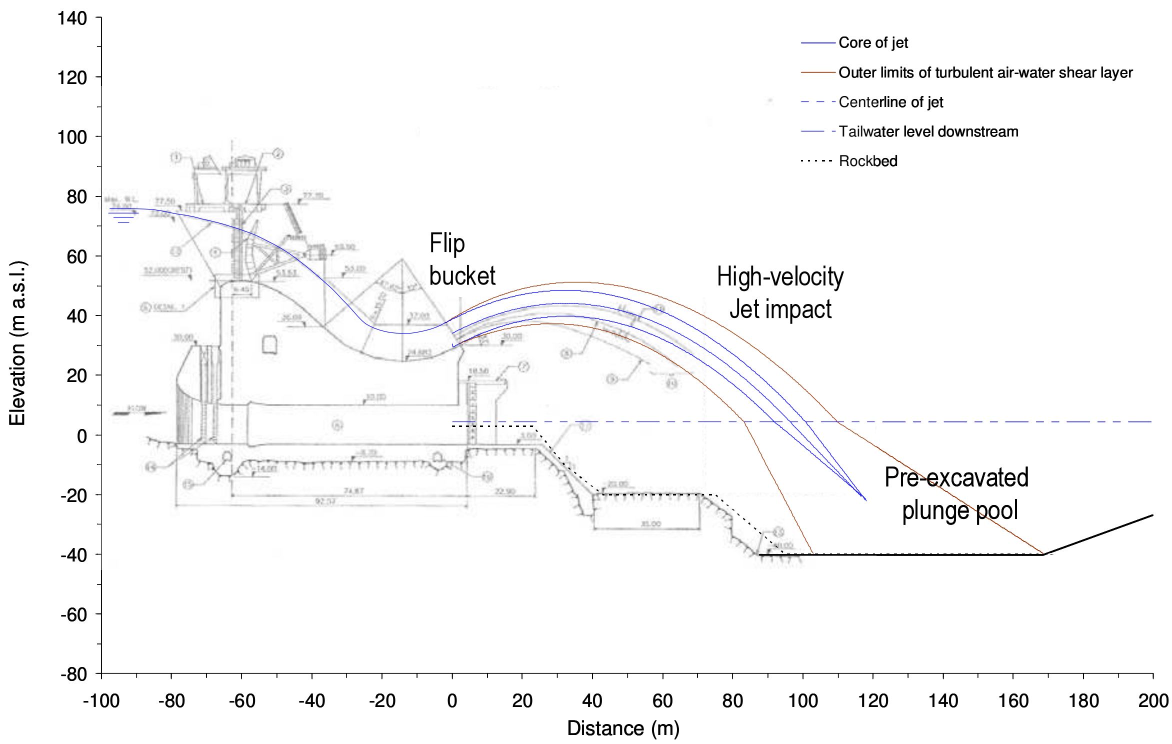

Figure 3. Cross-sectional view of spillway and pre-excavated plunge pool.

A. *Falling jet Module*

This module describes how the hydraulic and geometric characteristics of the jet are transformed from dam issuance down to the plunge pool (Figure 3). Three parameters characterize the jet at issuance: the velocity V_i, the diameter (or width) D_i and the initial turbulence intensity Tu, defined as the ratio of velocity fluctuations to the mean velocity.

The jet trajectory is based on ballistics and air drag and will not be further outlined. The jet module computes the longitudinal location of impact, the total trajectory length L and the velocity and diameter at impact V_j and D_j. The turbulence intensity is presented in the next paragraph and defines the spread of the jet δ_{out} (Ervine et al., 1997). Superposition of the outer spread to the initial jet diameter D_i results in the outer jet diameter D_{out}, which is used to determine the extent of the zone at the water-rock interface where severe pressure damage may occur. Relevant mathematical expressions are written:

$$Tu = u'/U \tag{1}$$

$$\frac{\delta_{out}}{X} = 0.38 \cdot Tu \tag{2}$$

$$D_j = D_i \cdot \sqrt{\frac{V_i}{V_j}} \tag{3}$$

$$V_j = \sqrt{V_i^2 + 2gZ} \tag{4}$$

$$D_{out} = D_i + 2 \cdot \delta_{out} \cdot L \tag{5}$$

in which δ_{out} is the half angle of outer spread, X the longitudinal distance from the point of issuance and Z the vertical fall distance of the jet. Typical outer angles of jet spread are 3-4 % for roughly turbulent jets (Ervine & Falvey, 1987). The corresponding inner angles of jet spread are 0.5 - 1 %.

The angle of the jet at impact is neglected, which is reasonable for impingement angles that are close to the vertical (70-90°). For smaller impingement angles, it is proposed to redefine the water depth Y as the exact trajectory length of the jet through the water cushion, and not as the vertical difference between water level and pool bottom.

B. *Plunge Pool Module*

This module describes the hydraulic and geometric characteristics of the jet when traversing the plunge pool and defines the water pressures at the water-rock interface. The plunge pool water depth Y is essential. For near-vertically impacting jets, it is defined as the difference between the water level and the bedrock level at the point of impact. The water depth increases with discharge and scour formation. Initially, Y equals the tailwater depth t (Figure 4). During scour formation, Y has to be increased with the depth of the formed scour h. Prototype observations indicate possible mounding at the downstream end of the pool. This mounding results from detached rock blocks that are swept away and that deposit immediately downstream. This can raise the tailwater level. The effect is not directly described in the model but can easily be added to the computations by appropriate modification of the water depth during scour.

The water depth Y and jet diameter at impact D_j determine the ratio of water depth to jet diameter at impact Y/D_j. This ratio is directly related to jet diffusion. Precaution should be taken when applying this parameter. Significant differences may exist in practice due to the appearance of vortices or other surface disturbing effects, which can change the effective water depth in the pool. Again, engineering judgment is required on a case-by-case basis.

Dynamic pressures acting at the water-rock interface can be generated by core jet impact, appearing for small water depths Y, or by developed jet impact (shear layer), appearing for Y/D_j higher than 4 to 6 (for plunging jets) (Figure 4). The most relevant pressure characteristics are the mean dynamic pressure coefficient C_{pa} and the root-mean-square (rms) coefficient of the fluctuating dynamic pressures C'_{pa}, both measured directly under the centerline of the jet. These coefficients correspond to the ratio of pressure head (in [m]) to incoming kinetic energy of the jet ($V^2/2g$) and are defined as follows:

$$C'_{pa} = 0.00022 \cdot \left(\frac{Y}{D_j}\right)^3 + 0.0079 \cdot \left(\frac{Y}{D_j}\right)^2 + 0.0716 \cdot \left(\frac{Y}{D_j}\right) + \eta \tag{6}$$

$$C_{pa} = 38.4 \cdot (1 - \alpha_i) \cdot \left(\frac{D_j}{Y}\right)^2 \quad \text{for } Y/D_j > 4\text{-}6 \tag{7}$$

$$C_{pa} = 0.85 \quad \text{for } Y/D_j < 4\text{-}6 \tag{8}$$

$$\alpha_i = \frac{\beta}{1+\beta} \tag{9}$$

Eqs. (7)-(9) are based on Ervine et al. (1997). The air concentration at jet impact α_i is defined as a function of the volumetric air-to-water ratio β. Plausible prototype values for β are 1-2. For a given α_i, mean and fluctuating dynamic pressures are defined as a function of Y, D_j and Tu. Similar expressions are proposed at locations radially outwards from the jet's centerline and can be found in Bollaert (2002). Tu is assumed representative for low-frequency fluctuations, which define the stability of the jet during its fall. Hence, Tu can be related to the rms values of the pressure fluctuations at the pool bottom. This is essential because these fluctuations generate peak pressures inside underlying rock joints.

Following eq. (6), the rms values of the pressure fluctuations at the pool bottom (C'_{pa}) depend on Y/D_j and Tu. The parameter η of eq. (6) represents the degree of jet stability: η is equal to 0 for compact jets and goes up to 0.15 for highly turbulent and unstable jets. Compact jets

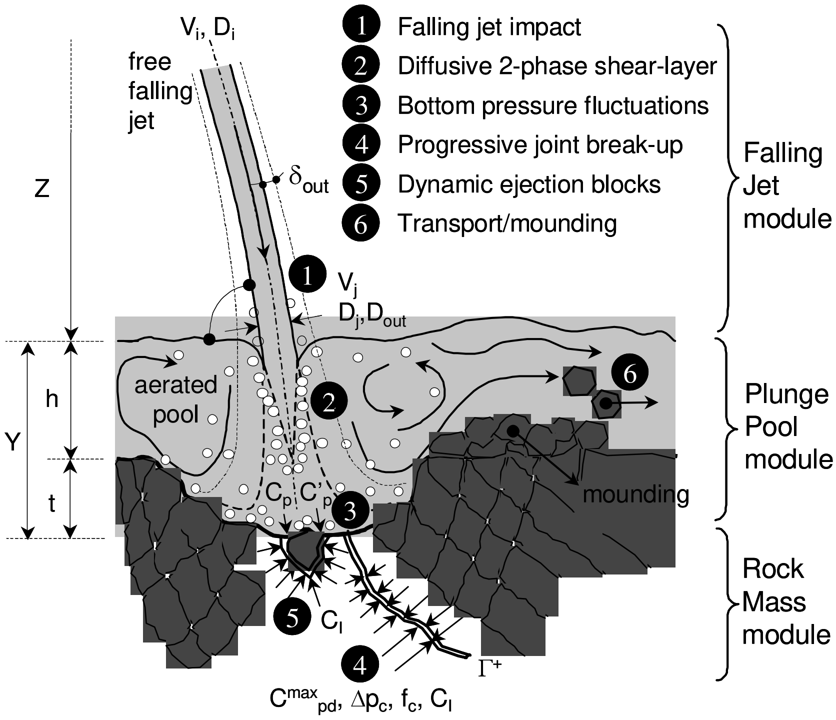

Figure 4. Main parameters of scour and its physical phenomena (Bollaert, 2004)

(Tu < 1%) are smooth-like during their fall, without any instability. Highly turbulent jets have a Tu > 5%. In between, for 1% < Tu < 5%, η has to be chosen between 0 and 0.15 as a function of jet stability effects.

Generally, Tu is unknown. Under such circumstances, estimation can be made based on the type of outlet structure (Bollaert et al., 2002). As such, a free overfall jet has an estimated Tu of 1-3 %, a ski jump 3-5 %, and intermediate or bottom outlets 5-8 %. However, Tu may largely depend on the outlet geometry, the flow pattern upstream, etc. These aspects should be accounted for by appropriate engineering judgment.

C. *Rock Mass Module*

The pressures defined at the bottom of the pool are used for determination of the transient pressures inside open-end or closed-end rock joints. The parameters are:

1. maximum dynamic pressure coefficient C^{max}_p
2. characteristic amplitude of pressure cycles Δp_c
3. characteristic frequency of pressure cycles f_c
4. maximum dynamic impulsion coefficient C^{max}_I

The first parameter is relevant to brittle propagation of closed-end rock joints. The second and third parameters express time-dependent propagation of closed-end rock joints. The fourth parameter is used to define dynamic uplift of rock blocks formed by open-end rock joints.The maximum dynamic pressure C^{max}_p is obtained through multiplication of the rms pressure C'_{pa} with an amplification factor Γ^+, and by superposition with the mean dynamic pressure C_{pa}. Γ^+ expresses the ratio of the peak value inside the rock joint to the rms value of pressures at the pool bottom and has been determined based on prototype-scaled experiments (Bollaert, 2004).

The product of C'_{pa} times Γ^+ results in a maximum pressure, written as (Bollaert, 2002):

$$P_{max}[Pa] = \gamma \cdot C_p^{max} \cdot \frac{V_j^2}{2g} = \gamma \cdot \left(C_{pa} + \Gamma^+ \cdot C'_{pa}\right) \cdot \frac{V_j^2}{2g} \quad (10)$$

The main uncertainty of eq. (10) lies in the Γ^+ factor. The characteristic amplitude of the pressure cycles, Δp_c, is determined by the maximum and minimum pressures of the cycles. The characteristic frequency of pressure cycles f_c follows the assumption of a perfect resonator system and depends on the air concentration in the joint α_i and on the length of the joint L_f.

Beside the dynamic pressure inside rock joints, the resistance of the rock also has to be determined. The cyclic character of the pressures generated by the impact of a high-velocity jet makes it possible to describe joint propagation by *fatigue* stresses occurring at the tip of the joint. This can be described by Linear Elastic Fracture Mechanics (LEFM).

A simplified methodology is used (Bollaert, 2004). It is called the *Comprehensive Fracture Mechanics (CFM) method* and applicable to any type of partially jointed rock. Pure tensile pressure loading inside rock joints is described by a stress intensity factor K_I, which represents the amplitude of the stresses that are generated by the water pressures at the tip of the joint. The corresponding resistance of the rock mass against joint propagation is expressed by its fracture toughness K_{Ic}.

Joint propagation distinguishes between brittle (or instantaneous) joint propagation and time-dependent joint propagation. The former happens for a stress intensity factor that is equal to or higher than the fracture toughness of the material. The latter is occurring when the maximum possible water pressure results in a stress intensity that is inferior to the material's resistance. Joints may then be propagated by fatigue. Failure by fatigue depends on the frequency and the amplitude of the load cycles. The fracture mechanics implementation of the hydrodynamic loading consists of a transformation of the water pressures in the joints into stresses in the rock. These stresses are characterized by K_I as follows:

$$K_I = P_{\max} \cdot F \cdot \sqrt{\pi \cdot L_f} \qquad (11)$$

in which K_I is in MPa√m and P_{max} in MPa. The boundary correction factor F depends on the type of crack and on its persistency, i.e. its degree of cracking defined as a/B or b/W in Figure 5. This figure presents two basic configurations for partially jointed rock. The choice of the most relevant geometry depends on the type and the degree of jointing of the rock.

The first crack is of semi-elliptical shape and partially sustained by the surrounding rock mass in two horizontal directions. Corresponding stress intensity factors should be used in case of low to moderately jointed rock. The second crack is single-edge notched and of two-dimensional nature. Support from the surrounding rock mass is only exerted perpendicular to the plane of the notch and, as a result, stress intensity factors will be substantially higher. Thus, it is appropriate for significantly to highly jointed rock.

For practice, F values of 0.5 or higher are considered to correspond to completely broken-up rock, i.e. the DI method becomes more applicable than the CFM method. For values of 0.1 or less, a tensile strength approach is more plausible. However, most of the values in practice can be considered between 0.20 and 0.40, depending on the type and number of joint sets, the degree of weathering, joint interdistances, etc.

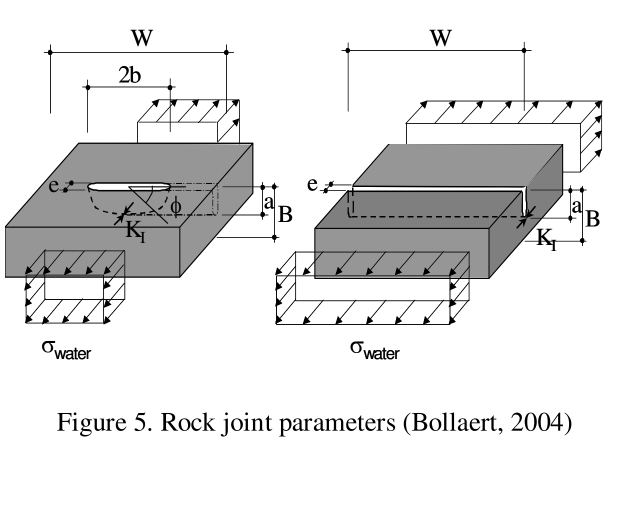

Figure 5. Rock joint parameters (Bollaert, 2004)

The fracture toughness K_{Ic} has been related to the mineralogical type of rock and to the unconfined compressive strength UCS. Furthermore, corrections are made to account for the loading rate and the in-situ stress field. The corrected fracture toughness is defined as the in-situ fracture toughness $K_{I,ins}$ and is based on a linear regression of available literature data.

$$K_{I\,ins,\,UCS} = (0.008\text{-}0.010)\cdot UCS + (0.054\cdot\sigma_c) + 0.42 \qquad (12)$$

in which σ_c represents the confinement horizontal in-situ stress and T, UCS and σ_c are in MPa. Instantaneous joint propagation will occur if $K_I \geq K_{I,ins}$. If this is not the case, joint propagation is expressed by an equation as originally proposed to describe fatigue growth in metals:

$$\frac{dL_f}{dN} = C_r \cdot (\Delta K_I / K_{Ic})^{m_r} \qquad (13)$$

in which N is the number of pressure cycles. C_r and m_r are material parameters that are determined by fatigue tests and ΔK_I is the difference of maximum and minimum stress intensity factors. To implement time-dependent joint propagation into the model, m_r and C_r have to be known. They represent the vulnerability of rock to fatigue and can be derived from available literature data on quasi-steady break-up by water pressures in joints (Atkinson, 1987). A first-hand calibration for granite (Cahora-Bassa Dam; Bollaert, 2002) resulted in C_r = 1E-8 for m_r = 10.

The fourth dynamic parameter is the maximum dynamic impulsion C^{max}_I in an open-end rock joint (underneath single block), obtained by time integration of net forces on the block:

$$I = \int_0^{\Delta tpulse} (F_u - F_o - G_b - F_{sh}) \cdot dt = m \cdot V_{\Delta tpulse} \qquad (14)$$

in which F_u and F_o are the forces under and over the block, G_b is the immerged weight of the block and F_{sh} represents the shear and interlocking forces. The shape of a block and the type of rock define the immerged weight. Shear and interlocking forces depend on the joint pattern and the in-situ stresses. As a first approach, they can be neglected. The pressure field over the block is governed by jet diffusion. The pressure field under the block corresponds to transient pressure waves. The first step is to define the maximum net impulsion I^{max}. I^{max} is defined as the product of a net force and a time period. The corresponding pressure is made non-dimensional by the jet's kinetic energy $V^2/2g$. This results in a net uplift pressure coefficient C_{up}. The time period is non-dimensionalized by the travel period that is characteristic for pressure waves inside rock joints, i.e. $T = 2\cdot L_f/c$. This results in a time coefficient T_{up}. Hence, the non-dimensional impulsion coefficient C_I is defined by the product $C_{up} \cdot T_{up} = V^2 \cdot L/g \cdot c$ [m·s]. The maximum net impulsion I^{max} is obtained by multiplication of C_I by $V^2 \cdot L/g \cdot c$. Prototype-scaled analysis of uplift pressures resulted in the following expression for C_I:

$$C_I = 0.0035 \cdot \left(\frac{Y}{D_j} \right)^2 - 0.119 \cdot \left(\frac{Y}{D_j} \right) + 1.22 \qquad (15)$$

Failure of a block is expressed by the displacement it undergoes due to the net impulsion C_I. This is obtained by transformation of $V_{\Delta tpulse}$ in eq. (14) into a net uplift displacement h_{up}. The net uplift displacement that is necessary to eject a rock block from its matrix is difficult to define. It depends on the protrusion and the degree of interlocking of the blocks. A first-hand calibration on Cahora-Bassa Dam (Bollaert, 2002) resulted in a critical net uplift displacement of 0.20.

V. Application of CSM model to Tucurui Dam

The Comprehensive Scour Model has been applied to Tucurui Dam spillway and plunge pool. The model has first of all been calibrated based on the assumption that, for flood events of up to 50'000 m^3/s, no significant scour forms at the plunge pool bottom.

Second, the model has been applied to a fictitious design event with a discharge of 110'000 m^3/s. The different steps performed are explained hereafter for each module of the scour model.

A. *Falling Jet at Tucurui Dam*

The jet at Tucurui Dam is issuing from a chute with flip bucket. As such, numerical computations have first been performed of the air-water flow characteristics along the chute. The water surface line for a 110'000 m^3/s is presented in Figure 3.

The flow velocity at the lip of the flip bucket equals 27.8 m/s, for a total flow depth of about 8 m. The upstream water level is 75.3 m a.s.l. and the downstream plunge pool water level is at 4.4 m a.s.l. The jet is thus of rectangular shape with a width of 20m and an issuance thickness of 8m.

Second, the jet characteristics at impact in the tailwater depth have been defined based on ballistics accounting for air drag. As such, the jet velocity at impact has been computed at 37.9 m/s for an inner jet core diameter of about 6.9 m. Its turbulence intensity has been estimated at 8 % and its air concentration at impact at 40 %, corresponding to very turbulent air-water jets.

B. *Plunge Pool Diffusion at Tucurui Dam*

The diffusion of the jet through the pool water depth is presented in Figure 3. Significant spread of the jet is computed, with an outer jet diameter at impact in the pool of about 20m. The jet core vanishes before impacting the plunge pool bottom, corresponding to fully developed jet conditions.

Very little information is available regarding the type and quality of the rock mass at Turcurui Dam. As a first-hand approach, and based on the initial model calibration for lower discharges, the following table of values has been used for the computations:

Table 1. Rock mass characteristics used for the numerical computations at Tucurui Dam

	Property	Symbol	CONSERV	AVERAGE	BENEF	Unity
	Unconfined Compressive Strength	UCS	50	75	100	MPa
	Density rock	γ_r	2600	2700	2800	kg/m3
	Ratio horizontal/vertical stresses	K_0	2-3	2-3	2-3	-
	Typical maximum joint length	L	1	1	1	m
	Vertical persistence of joint	P	0.25	0.25	0.25	-
	Form of rock joint	-	single-edge	elliptical	circular	-
	Tightness of joints	-	tight	tight	tight	-
Rock mass	Total number of joint sets	N_j	3+	3	2+	-
	Typical rock block length	l_b	1	1	1	m
	Typical rock block width	b_b	1	1	1	m
	Typical rock block height	z_b	0.5	0.75	1	m
	Joint wave celerity	c	150	125	100	m/s
	Fatigue sensibility	m	8	9	10	-
	Fatigue coefficient	C	1.00E-07	1.00E-07	1.00E-07	-

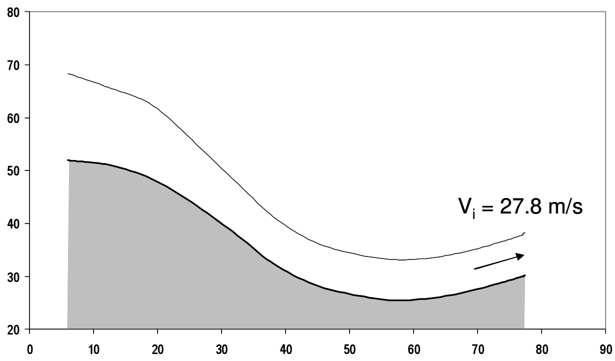

Figure 6. Numerical computation of flow characteristics along chute and flip bucket

Distinction has been made between conservative, average and beneficial assumptions on the different geomechanic parameters. A UCS strength between 50 and 100 MPa has been assumed.

C. *Comprehensive Fracture Mechanics (CFM) Results*

When applying the Comprehensive Fracture Mechanics (CFM) model under the centerline of the jet impacting in the plunge pool, Table 2 shows that beneficial parametric assumptions result in no scour formation.

For the most conservative parametric assumptions, scour formation down to a plunge pool bottom level of about – 65 m for a flood duration of 2 months has been computed. This is very close to the results obtained on the laboratory model using gravel. This is not so surprising given the fact that, for the single-edge rock joints that are assumed under conservative parametric conditions, the rock is considered almost completely broken up into distinct blocks and thus quite similar to gravel under laboratory conditions.

Second, on the long term (= after 80 months of design flood), maximum scour elevations between -47 m and -74 m have been computed. For only 8 months of design flood, which seems much more plausible during the lifetime of the dam, the corresponding plunge pool scour elevations are between -41 m and -67 m. In other words, even during very long periods of design discharge at Tucurui Dam, potential scour formation would still remain within controllable limits. This is especially true given the fact that, based on Figure 3, for such high floods, the jet impacts at about 130 m from the toe of the dam and thus no risk is apparent for the dam.

D. *Dynamic Impulsion (DI) Results*

Based on the Dynamic Impulsion model, the scour computed under the centerline of the jet becomes more important than for the Comprehensive Fracture Mechanics model, with scour elevations at – 63 m for beneficial parametric assumptions and down to – 94 m for conservative parametric assumptions. While the former value is again very close to the laboratory results, the latter seems much more pessimistic regarding future scour formation during the design flood event.

Nevertheless, it has to be kept in mind that the DI model results largely depend on the assumed ratio of rock block height to side length. Under conservative assumptions at Table 2, this ratio has been taken equal to 0.5. This means that only flat and completely detached rock blocks.

Table 2. Scour elevations as a function of time duration of flood following a 100'000 m^3/s flood event at Tucurui Dam (CSM model)

SCOUR ELEVATION COMPUTATIONS									
TIME				CFM			DI		
Hours	Days	Months	Years	BENEF	AVER	CONS	CONS	AVER	BENEF
96	4	0.1	0.01	-40.4	-53.6	-61.6	-93.9	-78.9	-63.2
192	8	0.3	0.02	-40.4	-53.6	-62.6	-93.9	-78.9	-63.2
720	30	1.0	0.08	-40.4	-54.3	-64.1	-93.9	-78.9	-63.2
1500	62.5	2.1	0.17	-40.4	-54.9	-65.0	-93.9	-78.9	-63.2
5760	240	8.0	0.67	-41.3	-56.4	-67.2	-93.9	-78.9	-63.2
57600	2400	80.0	6.67	-47.5	-59.2	-74.3	-93.9	-78.9	-63.2

VI. CONCLUSIONS

This Paper presents a new and physics based scour prediction model applicable to high-head hydraulic structures. The basics of the model are briefly outlined. The model has been applied to the Tucurui Dam spillway and plunge pool in Brazil, for which almost no scour has been observed in-situ after 17 years of moderate floods events. The model has first been calibrated based on these moderate flood events.

Then, the design flood of 110'000 m^3/s has been tested. Depending on the applied rock failure criteria and the assumptions made on the rock mass quality, scour elevations are situated between – 40 m and – 94 m.

However, when only applying the most plausible parametric combinations, scour would extend down to about – 55 m for partially broken up rock and down to -80 m in case of completely broken up rock at the dam site.

Considering the fact that all detached rock blocks have been washed out of the plunge pool immediately after dam construction, it may be stated that the ultimate scour depth following the design flood event would be situated around – 55 m. This is reasonably close to the laboratory model test result of – 65 m by using gravel to represent the rock mass.

References

[1] Atkinson, B.K. (1987). Fracture Mechanics of Rock. Academic Press Inc., London.

[2] Bollaert, E.F.R. (2002). Transient water pressures in joints and formation of scour due to high-velocity jet impact. Communication 13, Laboratory of Hydraulic Constructions, Lausanne, Switzerland.

[3] Bollaert, E.F.R. (2004). A comprehensive model to evaluate scour formation in plunge pools. Int. J. Hydropower & Dams, 2004, 2004(1), pp. 94-101.

[4] Bollaert, E.F.R. and Schleiss, A.J. (2005), A physically-based model for evaluation of rock scour due to high-velocity jet impact, Journal of Hydraulic Engineering, March 2005.

[5] Ervine, D.A. and Falvey, H.R. (1987). “Behavior of turbulent jets in the atmosphere and in plunge pools.” Proceedings of the Institution of Civil Engineers. Part 2, Vol. 83, 295-314.

[6] Ervine, D.A., Falvey, H.T., Withers, W. 1997. “Pressure fluctuations on plunge pool floors”. Journal of Hydraulic Research, Vol. 35, 2, 257-279.

[7] Petry, B.et al. (2002), Tucurui Spillway on the Tocantins River-Performance during 17 years of Operation, in "Large Brazilian Spillways--An Overview of Brazilian Practice and Experience in Designing and Building Spillways for Large Dams", International Commission on Large Dams/Brazilian Committee on Dams, 2002.