The Comprehensive Scour Model: Theory and Feedback from Practice

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ABSTRACT

This paper provides a feedback on theory and applications of the Comprehensive Scour Model (CSM, Bollaert 2002) to case studies and real-life projects over the last 10 years. The CSM has been initiated by Dr Bollaert in 2001 at the Laboratory of Hydraulic Constructions of the EPFL, Switzerland. The model has since then been further developed and completed at AquaVision Engineering by applying it to real-life rock scour problems at high-head dams worldwide. The model is part of the scour prediction methods recommended in the 2006 USSD bulletin on scour of unlined spillways.

The present paper briefly discusses the physics of failure and the corresponding mathematical principles, and illustrates applications by the model for different situations of design of scour mitigation measures or to predict potential future scour formation.

INTRODUCTION

The Comprehensive Scour Model is termed comprehensive in the sense that it incorporates the major physics relevant to scour in an easily understandable manner, i.e. by using mathematics of the physical laws that are both representative for the phenomena in question but at the same time easy to understand.

The model is applicable to any kind of brittle fractured medium, i.e. fractured rock, strong clays, concrete, etc. Typical fields of application are rock scour at spillways and stilling basins, rock scour at bridge piers, concrete fracturing of spillway chutes, uplift of stilling basin concrete linings, uplift of anchored sidewalls and protection slabs, a.s.o.

It uses the basic principles of linear elastic fracture mechanics to express hydraulic crack propagation in the fractured medium of interest. Second, dynamic uplift of the fractured medium due to net uplift forces and impulsions is being simulated. The hydraulic action for each failure mechanism is determined along the scour critical parts of the liquid-solid interface. The scour resistance of the fractured medium is expressed by using its geomechanical characteristics, such as for example the Unconfined Compressive Strength (UCS). Interaction between the progressing scour hole and its influence on the hydraulic action is being accounted for.

PHYSICS OF ROCK SCOUR

Fluvial erosion of rock as it appears in the vicinity of engineering structures mainly occurs following three physical-mechanical processes:

1. rock block removal (pressures in joints or shear flow),
2. rock mass fracturing (suddenly or progressively),
3. rock block abrasion (long term).

Figure 1 summarizes the most pertinent failure mechanisms of fractured rock in the vicinity of hydraulic structures, distinguishing between instantaneous and time-dependent processes. Each of these processes has its own time-scale of occurrence, ranging from instantaneous to long term. While certain short term actions have been rather well described in literature, sound assessment of medium and long term fluvial actions is still in its initial phases of development. Their relevance to scour depends on the characteristics of the turbulent flow and on the shape and the protrusion of the rock blocks. For small-sized rocky material, shear flow is generally predominant, just like for a granular riverbed. For large-sized irregular rock blocks, however, the shape, dimensions and protrusion of the blocks significantly impact the failure process.

In the following, the physics are explained as well as the corresponding computational modules being part of the CSM model.

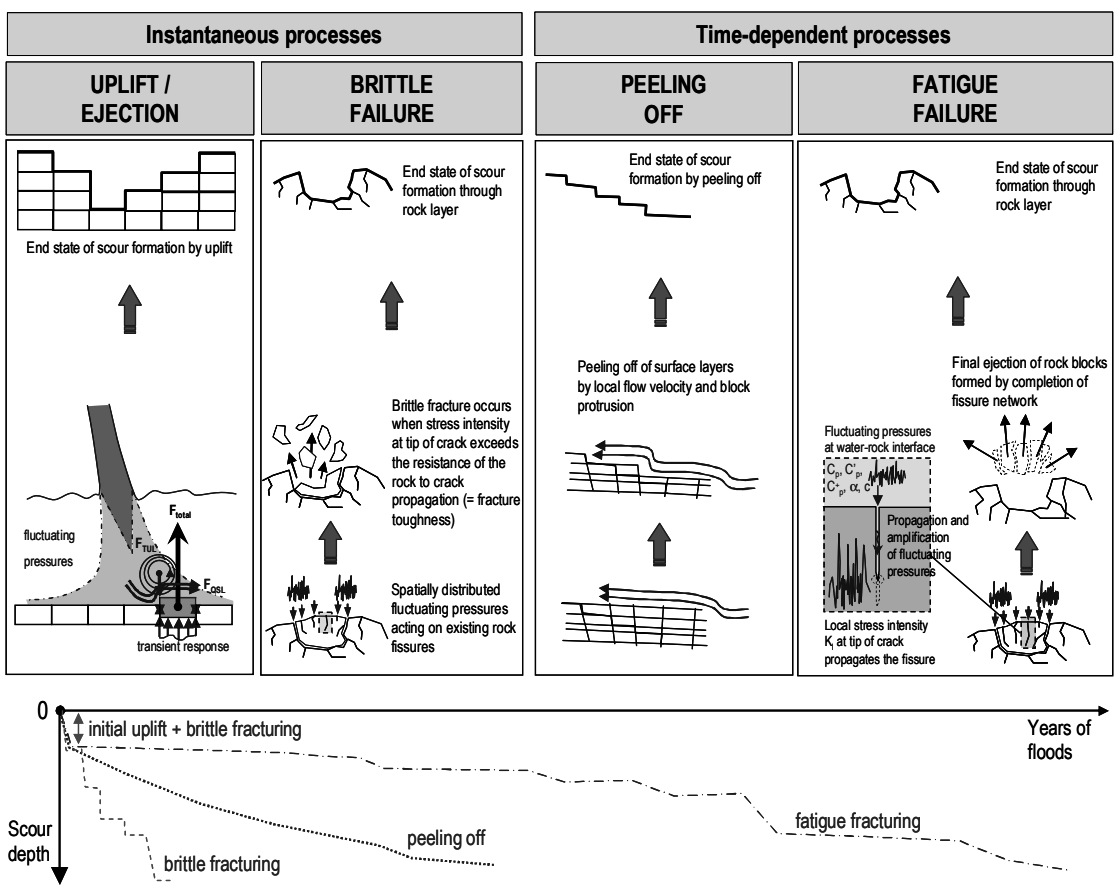

Figure 1. Principle failure mechanisms of fractured rock at hydraulic structures

Rock block removal

Rock may fail by removal of distinct blocks. This may happen by uplift (quasi-vertical ejection), by horizontal displacement, or by a combination of both. Flow turbulence is thereby of utmost importance. Which one of the movements is most plausible depends on the size, dimensions and protrusion of the blocks compared to the surrounding rock mass. These parameters directly define the relevance of the static, quasi-steady and turbulent forces that may lift the block. The Dynamic Uplift (DI) module of the CSM computes uplift of distinct rock blocks.

Rock mass fracturing

Rock may also fail by sudden or progressive hydraulic fracturing, which is mathematically described by the theory of linear elastic fracture mechanics. Brittle fracture occurs when the stress intensity at the edges of closed-end fractures is greater than the in-situ fracture toughness of the rock (Bollaert, 2002). The stresses induced by water pressures are governed by the geometry of the fracture and the support of the surrounding rock. The in-situ fracture toughness of the rock depends on the type of rock, the in-situ stress field and its unconfined compressive strength (UCS).

Second, progressive fracturing of rock occurs when the stress intensities do not exceed the fracture toughness. Prototype-scaled laboratory tests have shown the presence of severe air-water transient pressure waves inside rock joints (Bollaert, 2002; Bollaert & Schleiss, 2005). These will, on the medium or long term, propagate an existing fracture by fatigue, depending on the number and the intensity of pressure pulses. This failure type is time-dependent and takes an end when fracture formation is completed. The Comprehensive Fracture Mechanics (CFM) module of the CSM computes both brittle and fatigue fracturing as a function of duration of flooding.

Rock block peeling off

Peeling off of blocks is a specific combination of both quasi-steady pressure forces and brittle or fatigue fracturing. The phenomenon typically occurs in case of thin near-horizontal rock layers. The destabilizing forces are not due to flow turbulence alone, but are also generated by local flow deviation due to protrusion of the block. This flow deviation generates drag and lift forces on the exposed faces of the block, which are governed by the relative importance of the protrusion of the block and by the local quasi-steady flow velocity in its immediate proximity.

The corresponding pressures may develop brittle or fatigue fracturing of the joint between the block and the underlying rock. In case the exposed block is detached or almost detached, no further fracturing is needed to uplift the block by pressure fluctuations entering laterally into the joint. The Quasi-Steady Impulsion (QSI) module computes peeling off of distinct rock block layers.

Rock mass/block abrasion

Finally, rock scour by abrasion occurs if the fluid interacting with the rock is abrasive enough to cause scour in a layer-by-layer fashion. The process is enhanced by surface weathering of the exposed rock and, because of its lengthy time scale, often neglected compared to the other failure mechanisms. No module actually exists in the CSM for rock abrasion.

THE COMPREHENSIVE SCOUR MODEL (CSM)

A physics based scour prediction model has been developed (Bollaert, 2002, 2004; Bollaert & Schleiss, 2005). The model uses physical phenomena that have been simplified to allow their application for practice. It is based on experimental and numerical investigations of dynamic water pressures in rock joints (Bollaert, 2002).

The model computes failure of fractured rock following each of the aforementioned mechanisms. The structure consists of 3 modules: the falling jet, the plunge pool and the rock mass. The latter implements the failure mechanisms.

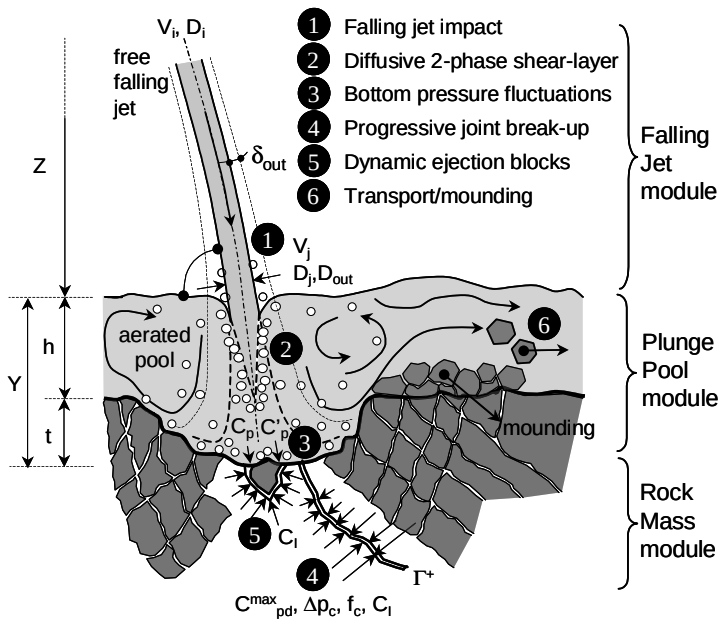

Figure 2. Main phenomena responsible for break-up of rock.

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 2). Three main 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. 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.

Plunge Pool Module

This module describes the 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. The water depth Y and jet diameter at impact D_j determine the ratio Y/D_j, which is directly related to jet diffusion. The most relevant pressures 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.

Rock Mass Module

The pressures at the bottom are used for determination of pressures inside rock joints. The main parameters are: the maximum dynamic pressure coefficient $C^{max}_{p,}$ the characteristic amplitude Δp_c and frequency f_c of pressure cycles and the 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}. The product of C'_{pa} times Γ^{+} results in a maximum pressure, written as (Bollaert, 2002):

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

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.

Second, the resistance of the rock has to be determined. The cyclic character of pressures in joints makes it possible to describe joint propagation by fatigue stresses occurring at their tip. This can be described by Linear Elastic Fracture Mechanics. Joint propagation distinguishes between brittle and time-dependent propagation. The former happens for a stress intensity equal to or higher than the fracture toughness of the rock. The latter is occurring in the opposite case. Joints may then be propagated by fatigue. Failure by fatigue depends on the frequency and the amplitude of the load cycles. Stresses are characterized by K_I as follows:

$$K_I = P_{max} \cdot F \cdot \sqrt{\pi \cdot L_f}$$

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. 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. 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. Hence, the in-situ fracture toughness $K_{I,ins}$ is based on a linear regression of available literature data and written as:

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

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 as follows:

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

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. A calibration for granite (Cahora-Bassa Dam; Bollaert, 2002) resulted in C_r = 1E-8 for m_r = 10.

The fourth parameter is the maximum dynamic impulsion C^{max}_I in an open-end joint (underneath single block), obtained by time integration of net forces on the block (pressures under and over block, immerged weight of block and eventually shear and interlocking forces). More details can be found in Bollaert (2004).

APPLICATION TO TUCURUI DAM

Tucurui Dam Spillway is located on the Tocantins River in northern Brazil. The spillway is characterized by an ogee type gate-controlled structure topped by 23 radial gates (20.75m high x 20m wide), a compact flip bucket and a 50m deep plunge pool (Figure 3). The design discharge is 110,000 cms under a gross head of 60 to 70 m. Hydraulics laboratory model tests resulted in the forecast of a satisfactory scouring behavior for a pre-excavated plunge pool at an elevation of – 40 m a.s.l.

Scour formation in the downstream plunge pool has been described by a series of bathymetric surveys since 1984. These showed that, as predicted by the laboratory tests, the maximum observed scour depth 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.

Hence, 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.

The CSM 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 (Bollaert & Petry, 2006).

Comprehensive Fracture Mechanics (CFM)

By using conservative parametric assumptions regarding rock resistance to scour, scour formation down to a plunge pool bottom level of about – 65 m (= 25 m of additional scour depth) 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 conservative parametric conditions, the rock is considered almost completely broken up into distinct blocks and thus quite similar to gravel under laboratory conditions.

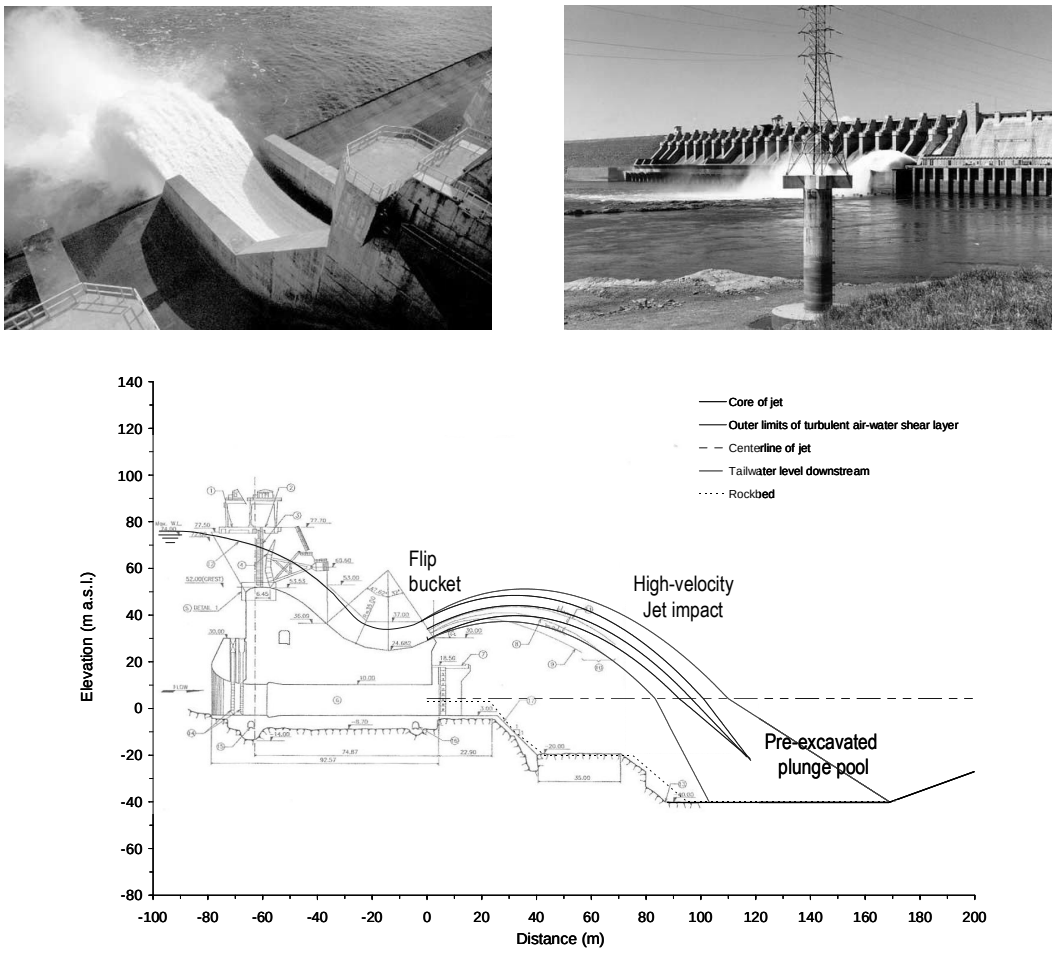

Figure 3. Photos and longitudinal section of spillway at Tucurui Dam.

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, 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.

Dynamic Impulsion (DI) results

Based on the DI model, scour becomes more important than for the CFM 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, this ratio has been taken equal to 0.5. This means that only flat and completely detached rock blocks would be present at the plunge pool bottom, which is obviously not the case.

APPLICATION TO KARAHNJUKAR DAM

Landsvirkjun, the National Power Company in Iceland, has completed in 2008 the 690 MW HEP Kárahnjúkar project in eastern Iceland. The main dam is a 200 m high CFRD dam. The bottom outlet of Kárahnjúkar Dam is 5.2 m wide, 6 m high and is concrete lined (Figure 4).

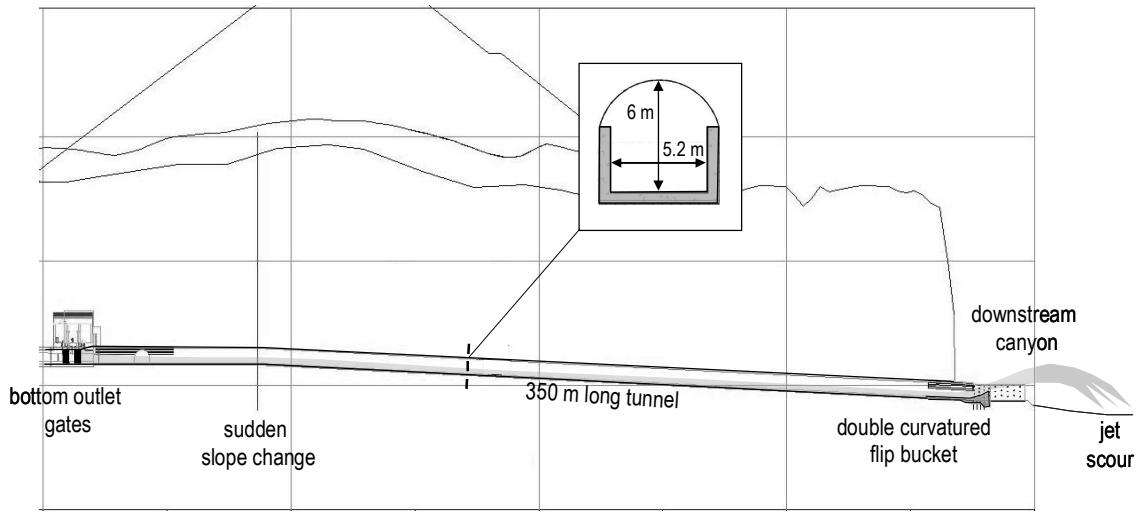

Figure 4. Longitudinal profile of bottom outlet and flip bucket.

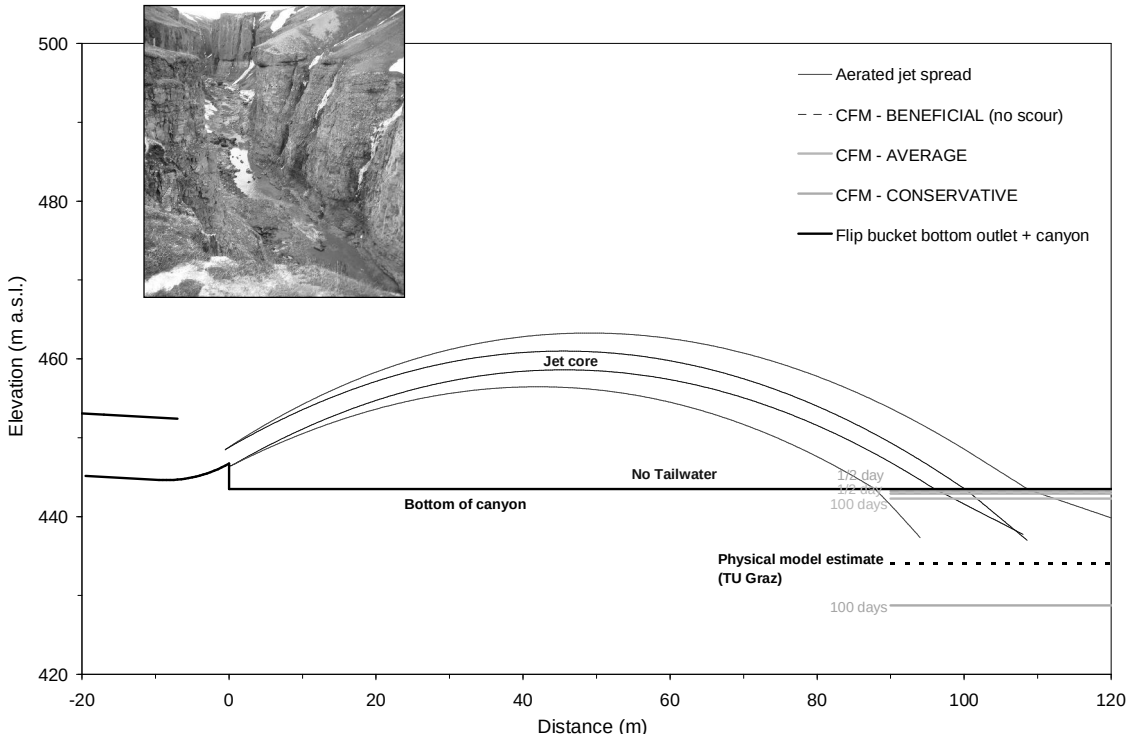

Figure 5. Scour formation in canyon as predicted by CFM module.

The first 50 m are near horizontal, followed by a slope change down to 5 % for the remaining 300 m downstream. The invert and side walls are concrete lined up to a height of 3.5 m. The tunnel ends with a double curvatured flip bucket that projects the water jet with an angle between 21 and 28° into the downstream canyon.

Numerical computations have been performed of potential scour formation of the canyon following bottom outlet operation. Both downstream tailwater level and duration of discharge have been accounted for. The results show that scour formation in the canyon riverbed will remain quite limited (Figure 5). Scour may occur under the form of uplift and displacement of loose blocks that are already present at the riverbed. Subsequent fracturing and block formation of the in-situ rock mass will take considerable time to occur and will most probably not result in excessive scour formation. Comparison has been made with hydraulic model tests of scour formation and showed very good agreement.

APPLICATION TO FOLSOM DAM

The DI and CFM modules have been applied to the lined stilling basin of Folsom Dam, a concrete gravity dam with a height of about 100 m situated near Sacramento, California. Due to a significant increase of the initial PMF estimates, the outlet works of the dam were initially proposed to be increased. This would have resulted in a significant increase of turbulent pressure fluctuations impacting the concrete lining of the downstream stilling basin.

Hence, at first, a concrete lining stability study has been performed, pointing out the need for significant additional steel anchors to keep the slabs in place. Following this, a rock scour study has been performed of the fractured rock mass underneath the concrete lining, to check for scour formation and extent under extreme conditions and following potential lining failure. In the following, examples are provided of results that were generated for the PMF event (Bollaert et al., 2006).

Figure 6 presents a plan and perspective view of the final 3D shape of the scour hole through the rocky foundation of the stilling basin. One can easily detect the areas of impact of the jets issuing from the outlets. The model predicts 6-9 m of scour formation within the first 12-24 h of a PMF flood, while subsequent scour deepening would need far more time to occur. No scour forms at the toe of the dam.

CONCLUSIONS

Based on the physics of rock failure mechanisms and a vast series of near-prototype scaled laboratory tests on water pressures in artificially generated rock joints, a numerical scour prediction model has been developed in 2001. The model predicts scour formation in any type of fractured medium by computing fracture propagation, dynamic uplift and peeling off of blocks.

During the last 10 years, the model has been widely used for scour mitigation at high-head dams and stilling basins. Within this framework, and based on recorded floods and related scour formation, the numerical model could be calibrated and be used to predict potential future scour formation with time. Hence, feedback from experience has shown that the model provides significant insight into the local scour mechanisms and is able to assist the engineer when designing scour mitigation measures.

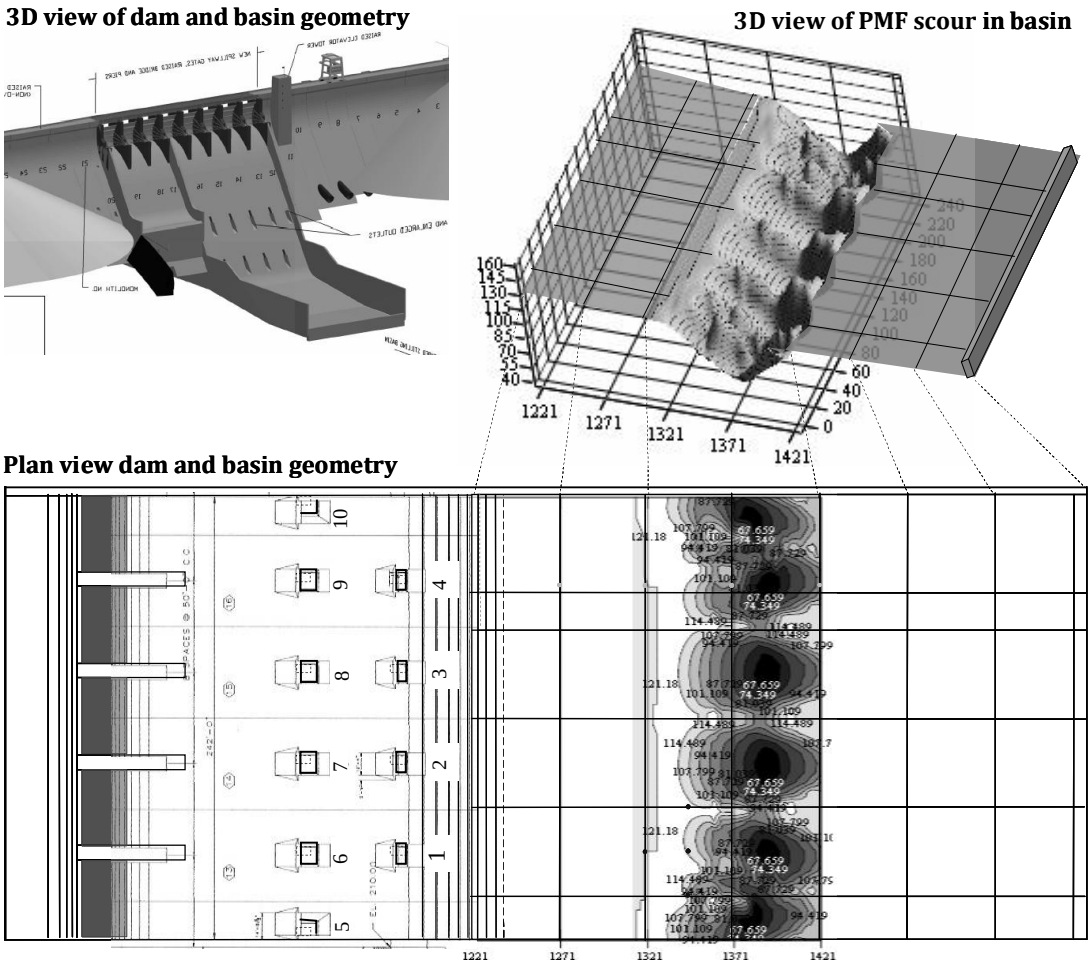

Figure 6. Plan view and perspective view of scour contours in stilling basin due to upper tiers functioning.

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