SASW SURVEY AND NUMERICAL MODELLING OF ROCK SCOUR AT GATED SPILLWAY (TEXAS, US)

E.F.R. BOLLAERT
AquaVision Engineering Llc., Ecublens, Switzerland

K. H. STOKOE
The University of Texas at Austin, Austin, US

D. MORTENSEN AND H. GERKUS-HARRIS
Freese & Nichols Inc., Austin, US

ABSTRACT

Following a major scour event immediately after dam construction, a scour hole with a depth of 9 m and a length of 61 m formed downstream of a gated spillway. Detailed scour analysis of the spillway included rock coring and strength tests, SASW (Spectral Analysis of Surface Waves) tests to define shear-wave velocity profiles, geologic mapping, and hydraulic loading determined using a computational fluid dynamics (CFD) model. Scour potential itself was modeled using the Comprehensive Scour Model (CSM), a physics-based 2D numerical model for time-dependent scour in fractured rock. This is the first study where SASW tests and 3D CFD modelling were used to generate input parameters to a 2D scour model. The results compared well with expected behavior based on previous physical model experience. Based on the available geologic and hydraulic input data, the CSM soundly reproduced past flood events for calibration as well as future scour potential by 2D longitudinal scour profiles. This allowed the dam owner to evaluate risk and determine if risk reduction measures were necessary. This paper describes how advanced field survey techniques like SASW have been used to determine the different rock layers at the site, allowing detailed 2D numerical modelling to predict scour potential at a gated spillway.

1. INTRODUCTION

1.1 Dam Description

The dam is located in Texas, USA and is used for generating hydroelectric power and for water supply. The present paper deals with a scour potential study performed downstream of the 7-Gate spillway. Due to confidentiality, the dam owner has authorized release of details of the study, but not identification of the specific structure. Figure 1 illustrates both the spillway. Figure 2 shows the main elements of the spillway, i.e. 7 gates, 8 piers, the original concrete apron, the 1938 apron extension and the 1991 repair works. Both the 1938 apron extension and the 1991 repair were the result of flood damage. Also, the current main scour hole (pond) downstream and an area containing big boulders (height of more than 10 feet) along the right-hand side of the basin are marked on Figure 2.

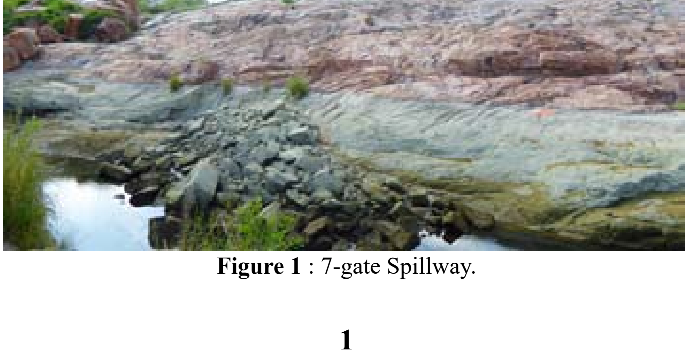

Figure 1 : 7-gate Spillway.

1.2 Scour history

The 7-gate spillway structure was constructed to pass floodwaters via seven (7), 12 m long by 8 m tall radial gates. The reinforced concrete spillway includes a chute section flanked by training walls. The flood of record occurred at the dam shortly after construction was completed in 1938. During the flood all seven gates were operated at various sequences to 5.5 m open with a maximum of five gates opened concurrently and a peak discharge of approximately 2,265 m^3/s. After flood operations were completed, scouring of the rock substrate adjacent to the spillway chute base slab directly downstream of Gates 1 to 3 was identified. The scour section along the right training wall was filled in and an approximately 46 m wide section of the spillway chute base slab were extended downstream approximately 30 m. The 7-gate spillway was not operated again until 1991 when another severe flood occurred in the basin and Gate 7 was opened to 5.5 m. The maximum discharge from the spillway was approximately 510 m^3/s. After flood operations were completed, scouring of the rock substrate adjacent to the spillway slab directly downstream of the opened gate was identified. The scour section was repaired with reinforced concrete.

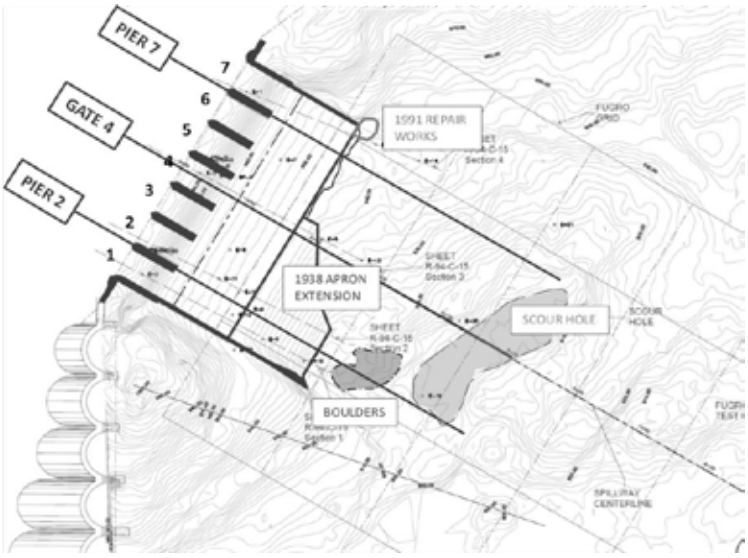

Figure 2 : Plan view of 7-gate Spillway, showing the different gates and piers, as well as the original concrete apron and its 1938 extension.

1.2 Project objectives

The 7-gate spillway structure may be used more frequently in the future. As such, future scour potential under frequent spillway use should be determined.

2. HYDROLOGY

2.1 Past flood events

As shown in Figure 3, the 1938 flood event had a total duration of about 96 h, for a maximum discharge of 2,300 m3/s during 48h. Gates 1-5-6 were used during the first 24h of the event at max. 1,200 m3/s, while gates 3 to 7 took over during the remaining 72 h of the event, because of the observed damage encountered along the concrete apron just downstream of gates 1 and 2. The 1991 flood occurred in December 1991 and lasted for about 48h at a quite constant discharge release of only 520 m^3/s.

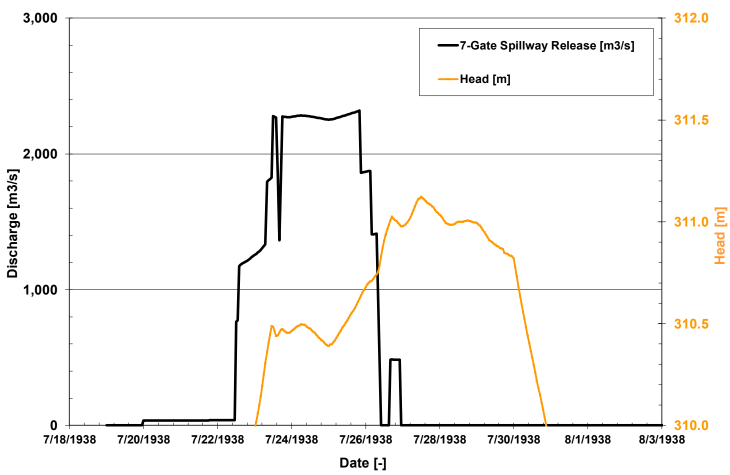

Figure 3 : The 1938 flood event at the 7-gate spillway.

3. GEOLOGY

3.1 Main characteristics and units

3.1.1 *Geologic units*

The principal rock foundation is Valley Spring Gneiss, with a mineral composition similar to granite (pink feldspar and quartz), with coarse or fine-grained granitic intrusions. The unconfined compressive strengths (UCS strengths) of the massive intact gneiss are between 90 and 175 MPa, with an average of 145 MPa (Fugro, 1992). 2018 lab tests indicate values in the flow area between 105 and 295 MPa, with an average of 175 MPa. The bulk density of the gneiss is ~2,600 kg/m^3. Beside the Valley Spring Gneiss, large rectangular or cubical boulders with dimensions ranging from 1-3 m are present. The larger deposits are situated downstream of the concrete apron extension, with a large number of big boulders. Top and bottom of these boulders are formed by near-planar exfoliation fractures and the sides are formed by widely spaced near-vertical fractures. Basalt intrusions are present under the form of discontinuous dikes that are perpendicular to the flow, 0.5-1.0 m wide, more than 15 m long. The basalt forms local shallow depressions.

3.1.2 *Joint sets at the site*

Two joint sets exhibit high-angle fractures, with dominant orientations N75E and N15-20W. These joints are near-vertical, widely spaced, tight, planar, and relatively short. One joint set exhibits low-angle fractures. These joints are nearly horizontal, with a strike that is perpendicular to the flow, and a dip of 5-15° mostly towards downstream, mostly caused by exfoliation, rough and irregular. The joints appear as thin relatively smooth surface layers parallel to the rock surface. The thickness of these layers ranges from less than 0.3 m to several meters.

3.1.3 *Valley Spring Gneiss unit*

The Valley Spring Gneiss Unit appears at the site under the following circumstances:

HFG = Highly Fractured Gneiss: material still consists of rock, UCS strengths at 70-200 MPa, rock break-up generates small pieces with a height-to-depth ratio of 1:5 to 1:10. The thickness of the original surface layer that has been almost completely scoured is between 1 and 3 m.

LFG = Lowly Fractured Gneiss: some joints are present, but the rock is mostly massive and the RQD values are high (75-100 %), UCS strengths at 200-300 MPa, rock break-up generates rather cubic pieces with a height-to-depth ratio of 1:1 to 1:2.

SDRS = Soft Decomposed Rock: completely loose and highly weathered, with no consistency, UCS strengths at 20-30 MPa, rock break-up generates pieces with a variable height-to-depth ratio of 1:1 to 1:10, that are easy to be peeled off by the flowing water.

3.2 Borehole data

Logs from different time periods are available (1930's, 1990's, 2018). The 1930's boreholes make a distinction between granite-good foundation, fractured zone-suitable for foundation, and finally soft decomposed rock and soil, not suitable for foundation. For the 2018 boreholes, detailed logs are available, containing a description of the joints encountered as well as of the RQD and UCS values. The most relevant boreholes are illustrated in Figure 4 (B102 to B108).

3.3 SASW data

Spectral Analysis of Surface Waves (SASW) tests have been performed at the site (Stokoe et al., 2017). The locations of the tests are presented by the white continuous lines in Figure 4. The shear wave velocities are mostly situated in between 600 m/s and 3,400 m/s. Values of 2,400 to 3,400 m/s are considered representative for intact gneiss rock. Values less than 2,400 m/s represent weathered and/or more or less heavily fractured gneiss rock. Especially values less than 2,000 m/s indicate significant presence of fractures, or even loose rock. The plan view shows the main areas of highly fractured or decomposed rock as those surfaces that are not darkened in Figure 4:

- Underneath the initial chute and a part of the chute extension,
- Area of concrete repair in 1992 (downstream of gate 7)
- Near the downstream pond all around,
- Whole area downstream of the chute extension at gates 1-2.

A zone of lowly fractured or even massive rock exists in the center, as shown by the darkened area in Figure 4. This zone corresponds to the flow area of the 1938 flood event. This event has almost entirely scoured the initial rock surface layer over a thickness between 1 and 3 meters. The underlying rock mass (current surface) is clearly more massive and less weathered and fractured. At some locations, pieces of the initial surface layer remain visible. Some are located along the left-hand side of the basin, but the most significant ones are concentrated along the right-hand side, downstream of the chute extension. All of the investigations performed underneath the initial chute show highly fractured rock or decomposed rock, based on both boreholes and SASW data, which are in good agreement.

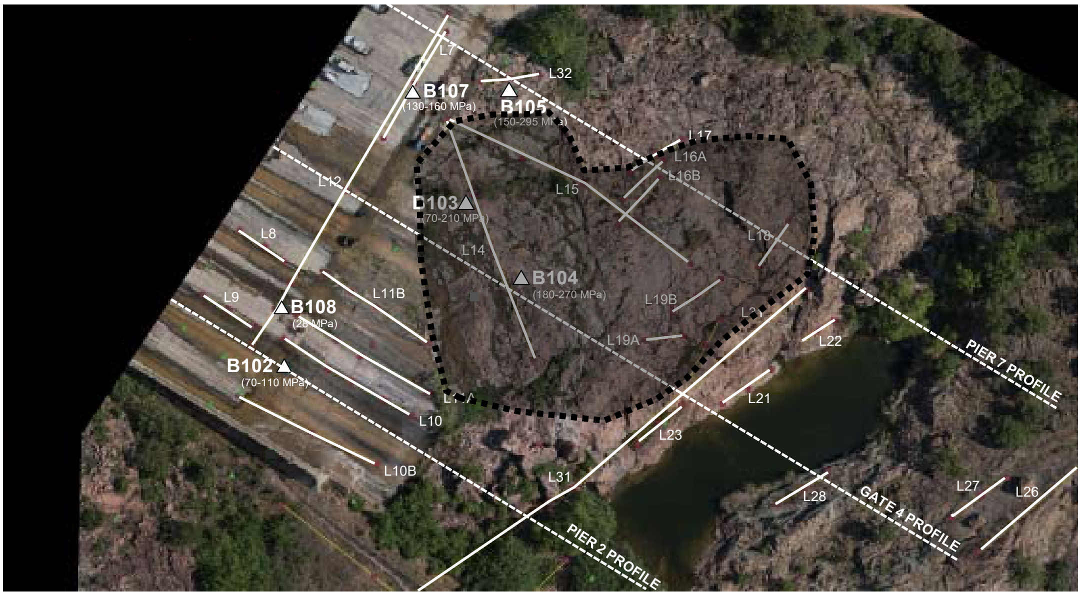

Figure 4 : Plan view of SASW test locations in rock mass downstream of 7-gate spillway.

3.4 Principal vertical geologic profiles

Based on the available geological data, three vertical geological profiles have been set up:

- PIER 2: through the axis of pier 2, in between gates 1 and 2 (right-hand side of the spillway)
- GATE 4: through the axis of gate 4, located in the center of the spillway
- PIER 7: through the axis of pier 7, in between gates 6 and 7 (left-hand side of the spillway)

An example is presented in Figure 5, valid downstream of pier 2 in 1938.

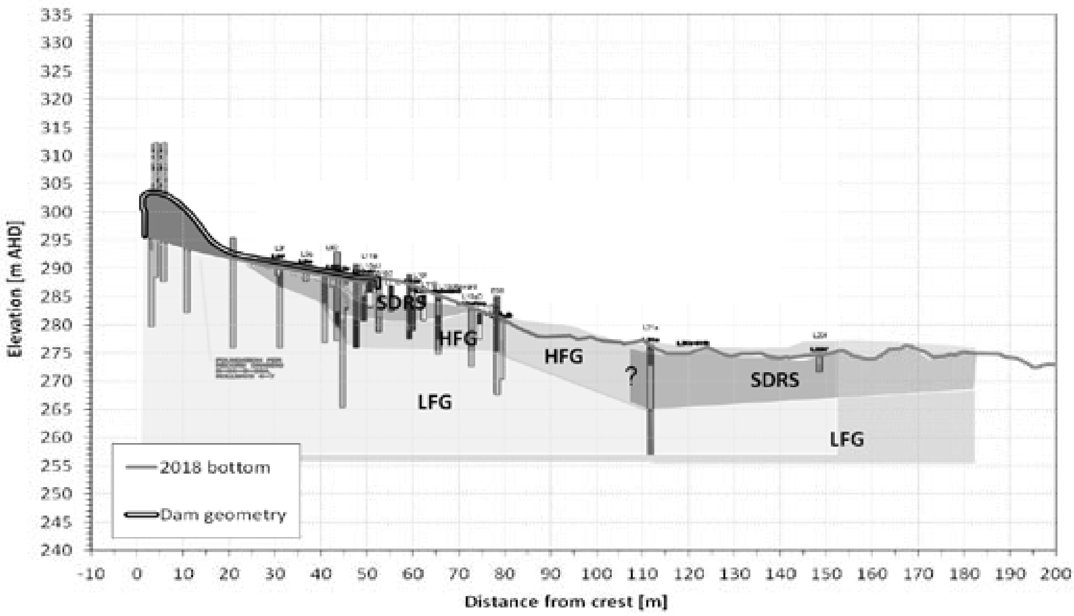

Figure 5 : Vertical geological profile downstream of pier 2, showing different boreholes and layers of lowly and highly fractured gneiss (LFG and HFG) and of soft decomposed rock (SDRS).

4. QUASI-3D NUMERICAL MODELLING OF SCOUR POTENTIAL

4.1 CFD modelling of flow velocities and dynamic water pressures

Freese and Nichols, Inc. performed a detailed 3D CFD numerical modelling of the dynamic pressures (average values and RMS fluctuations) and the velocity field (average values) for the 1938 flood event and for different future gate operating scenarios. The computed data has been used as input to the Comprehensive Scour Model (Bollaert, 2004). Figure 6 shows an example of 3D computed dynamic pressures along the gate 4 geologic profile during the 1938 flood event (gates 1-5-6 open). Figure 7 shows the RMS and average dynamic pressures computed by CFD for all gates fully open.

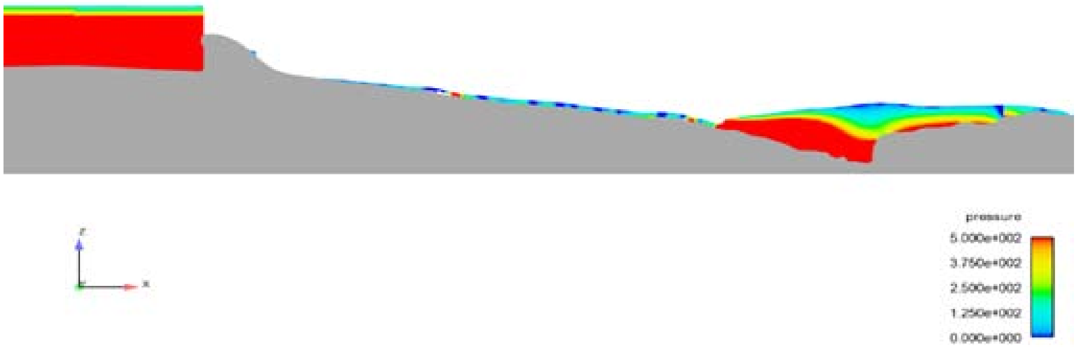

Figure 6 : 3D dynamic pressure field computed downstream of gate 4 during the 1938 flood event.

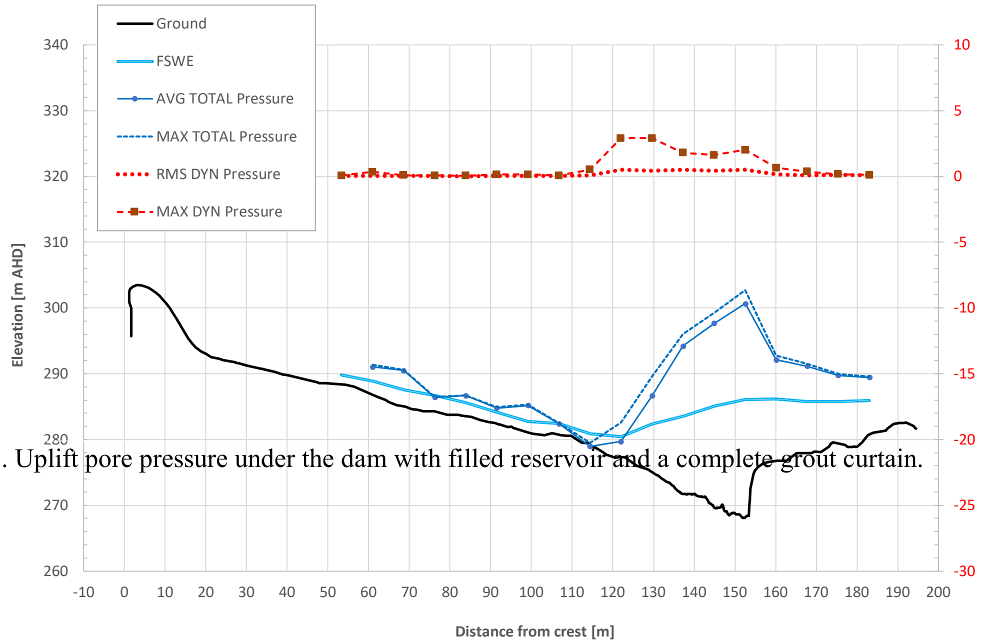

Figure 2. Uplift pore pressure under the dam with filled reservoir and a complete grout curtain.

Figure 7 : 3D dynamic pressure field downstream of gate 4 during a future flood with all gates fully open.

4.2 Quasi-3D numerical modelling of scour potential

AquaVision Engineering Llc. performed a quasi-3D numerical modelling of the future scour potential by using the Comprehensive Scour Model (CSM, Bollaert 2004).

4.2.1 *Introduction*

The CSM estimates temporal scour formation generated by turbulent high-velocity flows on any type of fractured material. The model is considered to be one of the most pertinent but unfortunately also one of the most data-demanding models available to practicing engineers to study scour potential downstream of high-head dams. The model is recommended by the US Society of Dams (USSD) in their 2006 bulletin on "Erosion of Unlined Spillways".

The CSM consists of three main parts: the turbulent flow, the plunge pool and the fractured rock (see Fig. 8 for a falling jet). The parts accounting for the turbulent flow and the plunge pool define the hydrodynamic loading that is exerted by the flow upon the rock mass.

For this project, the turbulent flow and plunge pool modules are being directly quantified by the results from the Flow-3D numerical modelling of the flow environment. The part accounting for the rock mass has a twofold objective. First, it transforms the hydrodynamic loading at the water-rock interface into a critical stress inside the rock mass. Secondly, it defines the basic geo-mechanical characteristics of the rock mass, relevant for the determination of its resistance.

4.2.2 *Scour Mechanisms*

The CSM estimates the ultimate depth of scour and the time evolution of scour in partially or totally fractured rock. The model is physics-based and comprises a comprehensive assessment of three major physical processes that are responsible for rock mass destruction by turbulent flow impingement:

1. Comprehensive Fracture Mechanics (CFM) module: hydrodynamic fracturing of existing rock joints by applying turbulent pressure fluctuations inside the joints and using fracture mechanics theory. As such, existing rock joints can be further fractured following a number of pressure cycles that are exerted by the turbulent flow. By defining the relationship between speed of fracturing and number of pressure cycles, this module adds a time phenomenon to the scour formation. This module needs the main turbulent flow parameters (water depth, pressure fluctuations, average pressure) to be determined along the water-rock interface.
2. Dynamic Uplift (DI) module: ejection of distinct rock blocks from the surrounding rock mass (once the joint network is formed). This module uses net uplift impulsions generated by flow turbulence (pressure fluctuations) on single blocks. The impulsions are determined based on the local standard deviations (RMS values) of pressure fluctuations.
3. Quasi-Steady Impulsion (QSI) module: peeling off of protruding rock blocks, generated by flow velocities close to the bottom (i.e. wall jets or equivalent kind of flow structure). These velocities are being deviated locally by protruding blocks, generating a dynamic pressure underneath the block and a suction pressure over the surface of the block. This module is based on average flow velocities, defined by a 2D turbulent flow model.

The CFM module is by far the most relevant break-up module in formations with a high UCS strength, and where the rock mass may be considered as not yet fully broken up into distinct rock blocks, such as is the case at the current dam.

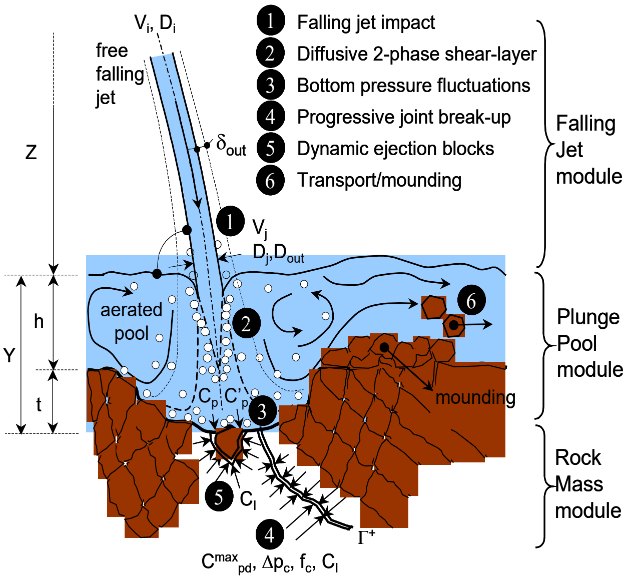

Figure 8 : Sketch of physical processes that generate scour in fractured rock.

4.2.3 *Calibration based on 1938 flood event*

The CSM has been calibrated based on the 1938 flood event, for which significant damage was observed in the area repaired with a concrete apron at gates 1 and 2 and further downstream. Figure 9 illustrates a comparison between the measured and computed scoured bottom downstream of gate 4.

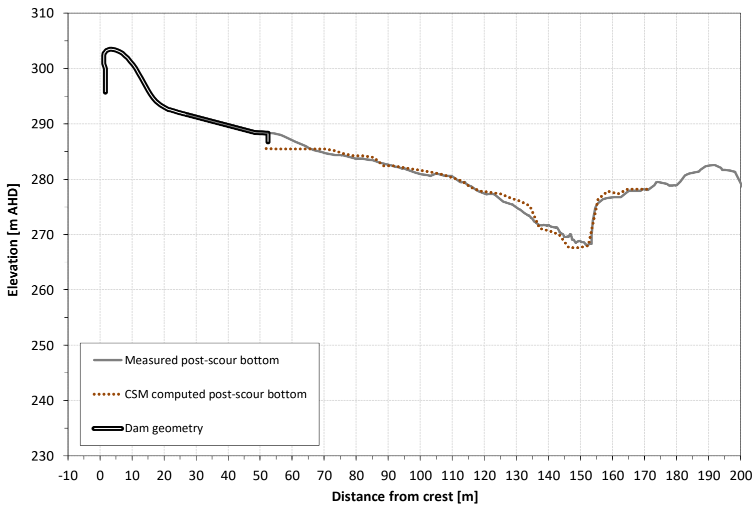

Figure 9 : Calibration of CSM based on the 1938 flood event (downstream of gate 4).

4.2.4 *Future scour potential*

The CSM has then been used to define the future scour potential for different gate opening scenarios, amongst which all gates fully open. Again, the dynamic pressures have been provided based on 3D CFD modelling.

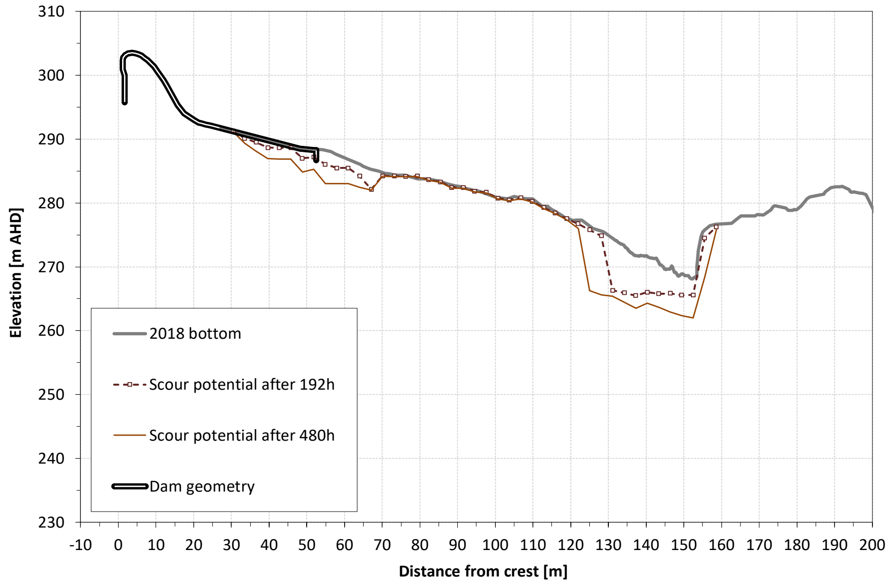

Figure 10 : Scour potential for all gates fully open downstream of gate 4.

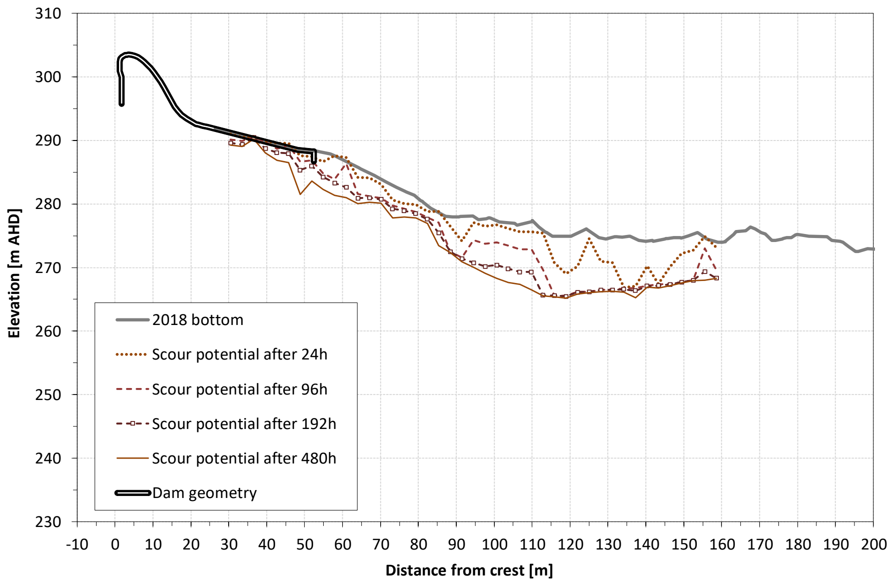

Figure 11 : Scour potential for all gates fully open downstream of pier 2.

Figure 10 shows that, downstream of gate 4, scour will form rapidly in the weak rock layer at the downstream pond (soft decomposed rock). Also, scour will be retarded in the underlying better-quality rock, but will start in this layer after less than 20 days of flood. Finally, scour will form in the highly fractured gneiss layer close to the chute end, on the order of a meter in 4 days, and down to an elevation of 284 m asl in 20 days of flood.

The risk for undermining and progressive destabilization of the original apron exists, but most probably not within one flood event (All Gates, max. 96h). For such an event, scour potential just downstream of the apron remains limited to about 1-2 m. It is believed that the current foundations of the apron are able to resist and maintain the apron in place and operational. Based on the performed scour computations, scour will continue during subsequent flood events, and reach 3-4 m after 192 h, and 7-8 m after about 480h of All Gates functioning. For these scour depths, destabilization of the downstream part of the apron will progressively generate scour towards upstream, by increased turbulence and concrete protrusion of parts of the apron. This phenomenon will stop along the interface between the highly and lowly fractured gneiss rock. Destabilization of the downstream part of the apron is likely to occur at gate 4, but would need more than one flood. Similar to Pier 2, scour will take an end based on the weak rock underneath. As the exact extension of this layer towards the spillway is not precisely known, it must be considered that the spillway itself might be in danger.

Figure 11 illustrates that scour potential downstream of pier 2 is significant and will destabilize the 1938 apron extension, generating regressive scour by subsequent displacement and protrusion of local concrete elements, generating on their turn additional local turbulence from downstream to upstream by perturbation of the main flow conditions. Hence, local flow perturbations actually present (boulders etc.) are triggers for scour initiation and start of scour regression towards the dam. It is believed that the 1938 apron extension will be scoured within 72h. Once the original apron is reached, local flow turbulence conditions will be at least as severe as presented in Figure 11. Local perturbations by displaced and protruding concrete elements will intensify the turbulence and thus the scour potential at that location.

Hence, the downstream part of the original apron will also be undermined and destabilized during the same event. The scour will take an end based on the positioning of the weak rock layer underneath the apron (soft and decomposed rock). As the exact extension of this layer towards the spillway is not precisely known, it must be considered that the spillway itself might be in danger.

5. CONCLUSIONS

This paper presents a novel approach for assessment of scour potential downstream of a gated spillway, based on a sound combination of a 3D CFD modelling of dynamic pressures and flow velocities and a quasi-3D comprehensive numerical modelling of the fracturing phase of the in-situ rock mass. Based on both borehole information and extensive SASW surveys performed all over the exposed rock surface, as well as on a sound calibration of the numerical approach by using the 1938 flood event and its observed damage, future scour potential could be determined as a function of duration of flooding for different gate opening scenarios. This has allowed the dam owner to understand scour risk for various gate operating scenarios and determine a series of scour mitigation measures that can be implemented at site.

REFERENCES

Bollaert (2004). A comprehensive model to evaluate scour formation in plunge pools. Int. Journal of Hydropower & Dams, 2004, 2004(1), pp. 94-101.

Stokoe, II, K.H., Hwang, S., Joh, S.-H. (2017). "Spectral-Analysis-of-Surface-Waves (SASW) Testing to Evaluate Vs Profiles at Geotechnical and Geological Sites", 16th World Conference on Earthquate Engineering, Santiago, Chile.