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HYDRODYNAMIC MODELLING OF INTAKES FOR RUN-OF-RIVER HYDROELECTRIC PROJECTS

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ABSTRACT: Run-of-river, diversion-type hydroelectric projects on steep mountainous streams create changes in environmental conditions, including modified hydraulic and sediment transport conditions in the headpond upstream of the flow diversion intake and in the diversion section of the stream between the intake and the tailrace. These types of changes can lead to environmental and operational challenges that require careful consideration at the project design stage in order to achieve a successful, sustainable project. Hydrodynamic modelling including two dimensional and Computational Fluid Dynamics (CFD) modelling were used to investigate the flow and sediment transport patterns through the headponds and intake structures of one existing and two proposed run-of-river hydroelectric projects located in coastal British Columbia. The intake structures consist of a concrete weir, a Coanda screen or an inflatable rubber weir, an intake facility with conveyance to the penstock entrance, and sluicing facilities for flushing sediment from the headpond. This paper reviews the relevant environmental, engineering and modelling issues for each project and discusses how results from these numerical modelling tools were used in fine-tuning the intake designs.

Keywords: run-of-river; sediment transport, CFD modelling, intake design

1. INTRODUCTION

Run-of-river, diversion-type hydroelectric projects, use a portion of the natural stream flow in combination with natural elevation differences to generate energy. The Pacific Coastal Region of British Columbia, Canada, receives annual precipitation typically in excess of 2000 mm, and the high mountains often contribute snowmelt and glacial melt late into the summer, which creates favourable conditions for producing energy during high demand periods. As of 2014, there were 56 independent run-of-river projects in British Columbia supplying electricity to BC Hydro, with another 25 anticipated to reach operation by 2018 (Clean Energy BC, 2015). These facilities range in installed capacity from <1 MW to over 100 MW, with nearly two-thirds being less than 10 MW and only 15% being greater than 50 MW. The run–of-river projects are being developed by independent power producers (IPPs) and are considered a significant contributor to the Province's goals of attaining energy self-sufficiency. These developments are considered green and renewable as they do not store or alter the timing of water flow, but rather divert a portion of the flow through a penstock or tunnel and return the water back to the creek further downstream unaltered. The intake headponds associated with these projects are used to maintain sufficient water depth for proper intake function. These headponds are much smaller than conventional storage reservoirs, and water levels in the headponds are typically kept at approximately constant levels.

Run-of-river projects have a much smaller environmental footprint compared to traditional hydroelectric projects with large reservoirs. They generally include a small diversion weir, an intake facility, a sluice channel, an in-stream flow requirement (IFR) bypass for ecological flows, a penstock, and a powerhouse. A portion of the stream flow is diverted at the weir through a penstock to the turbines to be returned to the natural stream channel through a tailrace downstream of the powerhouse. Flow conditions and flow patterns change with the construction of these projects, both upstream of the weir in the headpond and through the diversion reach, and even downstream of the tailrace during flow ramping events. The effects of these flow changes on the natural habitat in the diversion reach are, however, often the largest environmental concern. Reduction of natural water flow causes changes to sediment transport and water temperature, depth and flow velocity, and as such can change the quality and quantity of the habitat for fish and other organisms.

Flow patterns through the intake, headlosses, vortex activity, and the amount of sediment passing downstream to the penstock and turbines affect the operation and maintenance of the power generating equipment in the powerhouse. Numerical and physical hydraulic model studies are used to refine the design of the intakes with the aim to improve the hydraulic conditions through the intake, decrease head losses, improve sediment exclusion, decrease vortex activity, and improve sediment sluicing to the diversion reach. The choice of a modelling tool often depends on the type of question asked, the available input data, the required outcome, and the time and budget available to complete the study.

This paper discusses numerical modelling studies that included two-dimensional hydrodynamic modelling and Computational Fluid Dynamics (CFD) models, which were used to investigate the flow and sediment transport patterns through headponds and intake structures of one existing and two proposed run-of-river hydroelectric projects located in coastal British Columbia.

The incipient motion of sediment particles was estimated either using the Hjulström diagram (Hjulström 1935) in combination with the modelled flow velocities, or the modelled bed shear stress in combination with the Shields equation (Shields 1936). The Hjulström diagram relates the velocities to sediment particle sizes to determine if channel flow would erode, transport, or deposit sediment. The mobile sediment size using the Shields relationship is given by Equation 1:

[1] $$d_s = \frac{\tau}{\tau_0 g(\rho_s - \rho_w)},$$

where d_s is the sediment particle size that would have a potential to get mobilized (m); τ is bed shear stress caused by the given flow conditions (Pa); τ_0 is the non-dimensional critical shear stress equivalent to 0.06; ρ_s is the sediment density (kg/m^3); and ρ_w is the water density (kg/m^3). The non-dimensional critical shear stress of 0.06, selected for evaluating the sediment sizes at incipient motion, is typical for stream channels with gravel/cobble/boulder substrate material (Buffington and Montgomery 1997; Martin 2003). Local bed topography, however, plays an additional role in particle mobility, and particles hiding in wakes of larger boulders or imbricated within the bed may remain immobile. Non-dimensional critical shear stresses encountered in complex channel morphologies were shown to be as high as 0.1 (Church et al. 1998). Consequently, the modelled results represent average conditions for the modelled reaches, whereas these conditions may differ locally depending on bed topography.

2. PROJECT #1: EXISTING PROJECT WITH HIGH SEDIMENT TRANSPORT RATES

The first project discussed in this paper is an existing run-of-river project with an 88 MW installed capacity and a Coanda screen type intake structure. It is located on a creek that is subject to high sediment transport rates, which cause various management issues for the facility. Two-dimensional hydrodynamic modelling using River2D was undertaken to evaluate the flow conditions and sediment mobility within the headpond, and to examine mitigation options that would improve flow patterns and increase velocities to promote sediment flushing during sluicing operations. Two types of mitigation options were examined. The first option considered installing an additional sluice gate at the south end of the diversion weir, referred to as the south sluice gate. The second option considered headpond shaping with a sediment deflection/training groyne at the upstream end of the Coanda screen, which would deflect the flows toward the existing north sluice gate, and toward the newly proposed south sluice gate. Two sediment deflection groyne configurations were evaluated: an approximately 30 m long sediment deflection groyne, and a shorter approximately 15 m long sediment deflection groyne extending into the headpond.

2.1. Hydrodynamic Modelling and Discussion

A base case model was set up to evaluate current conditions in the headpond and through the north sluice gate during normal sluicing activities (Figure 1). The model was built in River2D to represent the diversion weir and the headpond geometry, using the “as-built” drawings based on survey data. The base case model geometry was then altered directly in River2D to represent the proposed mitigation options. All of the model runs were completed for the same steady state flow conditions to provide the same base of comparison for the different model simulations. A relatively high inflow of 33 m^3/s was selected, which is associated with the maximum design flow of the existing sluice gate, and therefore represents the maximum flow that sluicing activities could pass without overtopping the Coanda weir with the headpond drawn down. Flows of this magnitude typically occur over several days in the snowmelt freshet period each year (the mean monthly flow in June is 27.4 m^3/s).

Selected model results are presented on the following figures showing velocity contours combined with velocity vectors that are indicative of the velocity magnitudes and flow patterns. The same figures are then included separately to show the velocity contours and the corresponding largest mobile sediment sizes, as indicated on the figure’s legends.

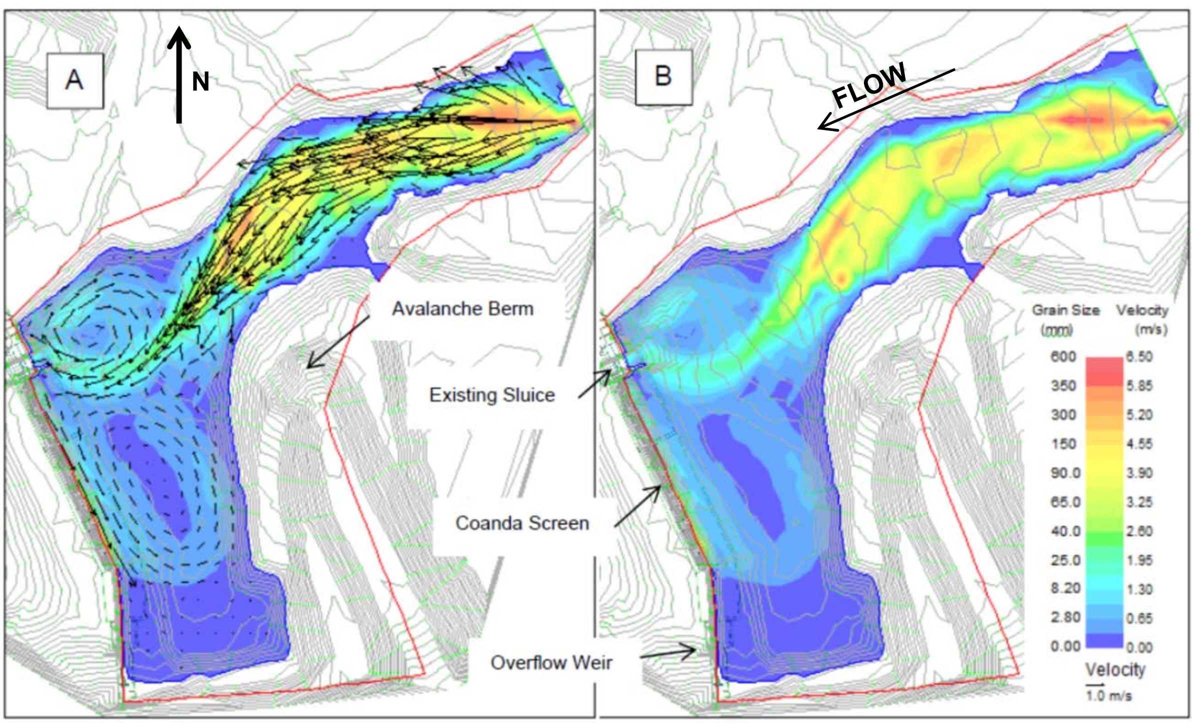

Figure 1. Baseline model – flow through the existing north sluice gate: A) Flow paths and velocities; and B) Mobile sediment grain sizes.

The results of this 2D numerical modelling study indicate that the addition of a second sluice gate on the south end of the diversion weir can increase velocities in the southern portion of the headpond, enhancing the sediment mobility in that area. When both the north and the south sluice gates are open, the velocities somewhat increase in the southern region of the headpond compared to the base case model. A promotion of even higher velocities in the southern region of the headpond is present when the north sluice gate is closed and only the south sluice gate is operating, hence resulting in greater flow conveyance with subsequent sediment mobilization towards the south sluice gate (Figure 2).

For the second mitigation option of headpond shaping, the effects of two sediment deflection groyne configurations were simulated. The results indicate that the larger deflection groyne is less effective than the shorter deflection groyne for increasing the velocities in the southern part of the headpond (Figure 3). Further optimization of the deflection groyne geometry would need to be undertaken to determine the most effective layout for improving flushing activities.

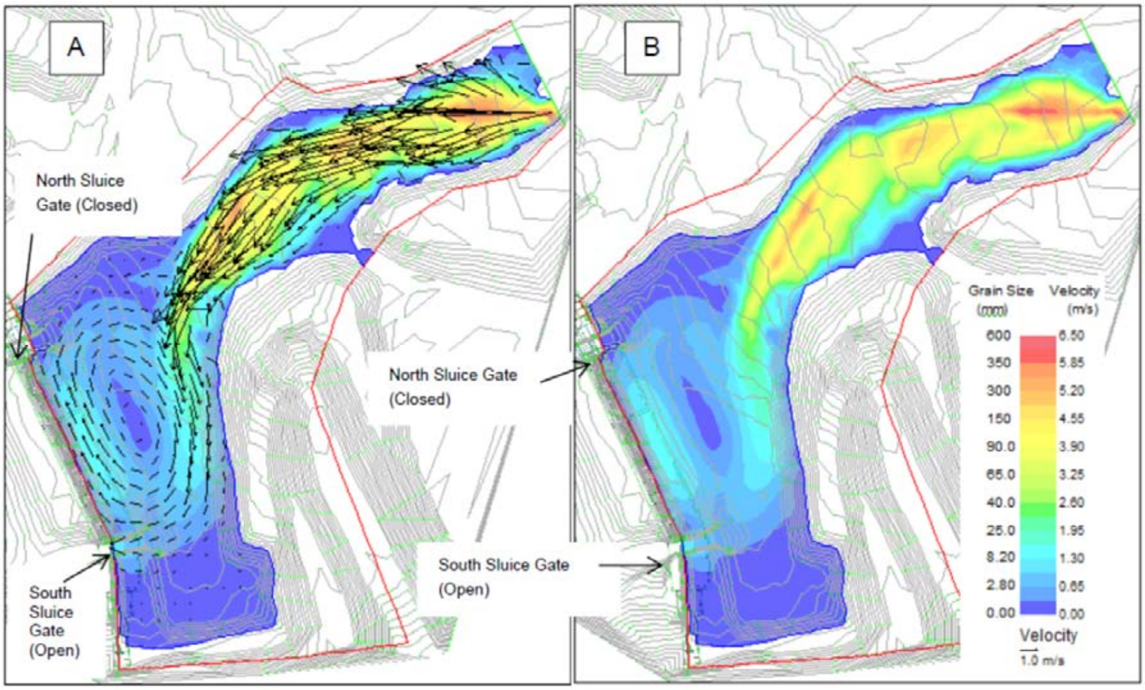

Figure 2. Additional sluice gate – flow only through the south sluice gate: A) Flow paths and velocities; and B) Mobile sediment grain sizes.

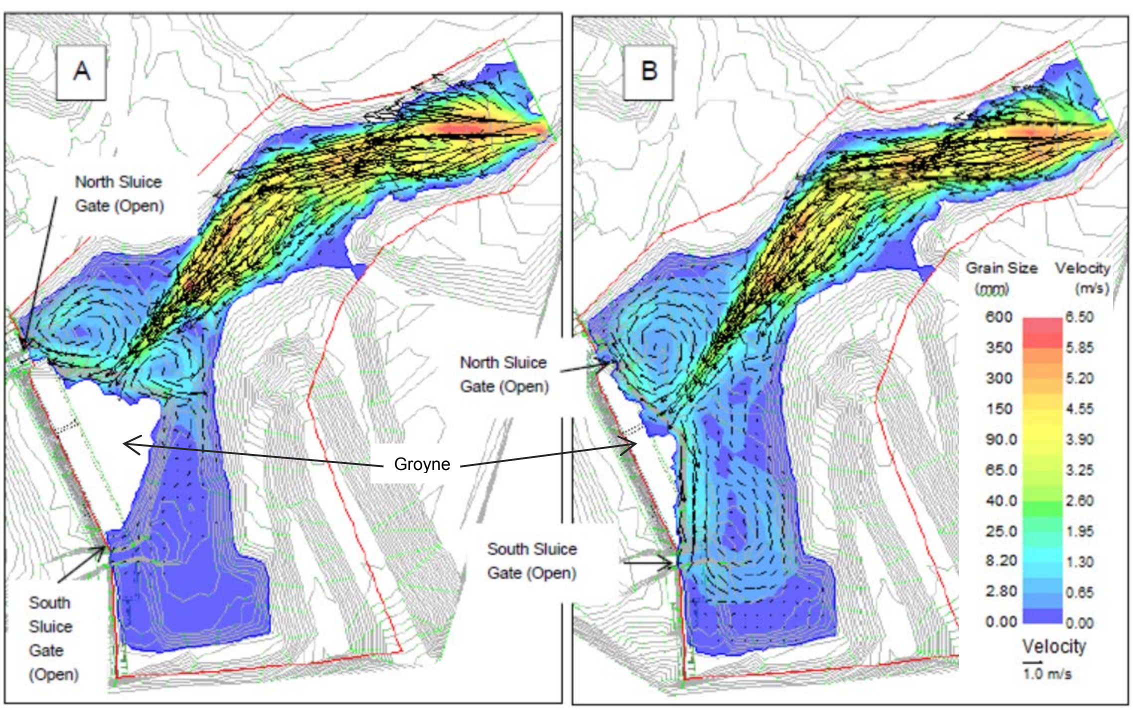

Figure 3. Sediment deflection groyne – flow paths and velocities through both the north and south sluice gates: A) 30 m long groyne and B) 15 m long groyne.

2.2. Summary for Project #1

Opening a new sluice gate on the south side of the Coanda screen would enhance the flows and increase the sediment flushing in that part of the headpond. The addition of a sediment deflection groyne in the headpond upstream of the Coanda screen would complement the efficiency of the newly proposed south sluice gate and serve two major functions: (a) prevent sediment from settling immediately upstream of the diversion weir during normal operations; and (b) promote higher velocities along the sediment deflection groyne during sluicing activities and increase sediment mobility. The region of high velocities along the sediment deflection groyne during sluicing activities would occur in the same region where sediment would be deposited during normal operations. The deflection groyne would, therefore, not only facilitate desirable sediment deposition during operations, but would also improve flushing activities.

3. PROJECT #2: PROPOSED PROJECT WITH LOW SEDIMENT TRANSPORT RATES

The second project discussed in this paper is a proposed 25 MW run-of-river project with a Coanda screen type intake structure. It is located on a creek that has a low bedload transport rate and the interruption of bed material replenishment in the reaches downstream of the intake represents an environmental concern. The headpond was estimated to take many years, even decades, to infill and start passing bedload sediment over the Coanda screen. The ability to use the sluiceway to pass the bedload that would accumulate in the headpond during normal operations is important for maintaining downstream fish spawning habitat. Field studies for this project determined that the appropriate range of bed material sediment sizes for fish spawning are from 10 mm to 100 mm.

The intake structure will feature two sluice facilities located on either side of the Coanda screen. The sluice facilities will pass flows during spillway construction and will facilitate sediment flushing through the headpond during operations. The sluice facility on the south side of the Coanda screen will be located in the deepest section of the headpond (former stream channel) and will be the primary sediment sluice gate to promote sediment flushing to the downstream reaches. The sluice facility on the north side of the Coanda screen will act as the intake bypass during winter low flows and as a secondary sediment sluice gate.

Two-dimensional modelling with FLO-2D was used to investigate the natural flow patterns and sediment transport characteristics of the creek before project construction. CFD modelling using ANSYS Fluent was used for post-construction simulations to evaluate which operating conditions allowed the target spawning size sediment to be flushed through the headpond to the downstream reaches. Two flow conditions were selected for modelling both natural and post-construction conditions: the design flow for this project, which is equivalent to the average monthly freshet flow of 12 m^3/s, and a high annual daily flow of 24 m^3/s. The design flow was used to assess sediment flushing behaviour under normal targeted flow conditions for the project, while the high annual daily flow was used to assess the potential for sediment flushing.

3.1. Hydrodynamic Modelling and Discussion

FLO-2D modelling of the baseline sediment mobility in the natural stream was conducted to assess the potential for mobilization and deposition of a representative 50 mm sediment particle in the vicinity of the proposed intake structure. This sediment size is approximately in the middle of the observed range of spawning sediment sizes for the project. It was found that sediment would be mobilized with flows of both 12 m^3/s and 24 m^3/s, with the higher flow event resulting in more scour, indicating higher mobility of the 50 mm grain size.

CFD modelling was conducted for three different conditions, all based on the initial headpond morphology (i.e. before sediment infilling): for normal operating conditions at a design flow of 12 m^3/s; for operating conditions during the high annual daily flow with 12 m^3/s passing through the intake and 12 m^3/s passing through the south sluice gate; and for the high annual daily flow of 24 m^3/s all passing through the south sluice gate. The results provide a preliminary overview of the expected sediment mobility in the headpond.

During normal operations at design flow, sediment entrained from the upstream channel would deposit within the headpond and result in most of the headpond becoming gradually infilled; albeit, this may take a very long time. Similarly, sluicing activities during high flows while operating the facility to its design capacity with the headpond kept full, have a limited potential for scouring the sediment. Some scour is predicted only in the near region just upstream of the sluice gate, but other areas of the headpond would still be depositional. To mobilize the sediments from farther upstream in the headpond, the operations would have to be ceased, the south sluice gate fully opened, and the headpond drawn down to create flows that resemble stream-like conditions somewhat similar to those prior to project construction.

With the headpond drawn down during sluicing activities in high flow events, the sediment mobility through the sediment sluice channel greatly improves. Higher velocities ranging between 1.0 m/s and 2.0 m/s are predicted along the headpond thalweg (Figure 4). CFD modelling indicates that backwatering would occur upstream of the sluice gate as the narrow gate opening forms a restriction to flow. The backwater would extend for about 30 m creating a zone of lower velocities and shear stresses. The simulated bed shear stresses shown on Figure 5 indicate that during this flow scenario there would be a zone of increased shear stresses through the headpond thalweg (former stream channel) with 30 mm to 60 mm grain size potentially being mobilized.

3.2. Summary for Project #2

In general, the sluice channel is expected to facilitate sediment transport through the headpond during flushing events with the headpond lowered. It has been shown that there is a region of backwatering during these events that still creates a zone of lower velocities, which limits the size of sediment that would move in that area. It is expected that the bed slope and the velocities would change and enhance the predicted sediment transport as the headpond infills with sediment during normal operations, or as the sluicing activities progress and the sediment starts moving into these slower areas and the energy slope changes.

Based on this study and after comparing the pre- and post-project conditions, it appears that the sediment sluice channel planned for the project would be effective in passing the spawning size gravel similar to existing conditions in the natural channel. The project is currently under construction. Some intake details have changed during the detailed design; however, the south sluice gate and the sluice channel have been kept within the thalweg of the former stream channel, as modelled in this CFD study. The true effectiveness of the sluice gate for passing spawning size sediment downstream of the intake remains to be tested upon project completion.

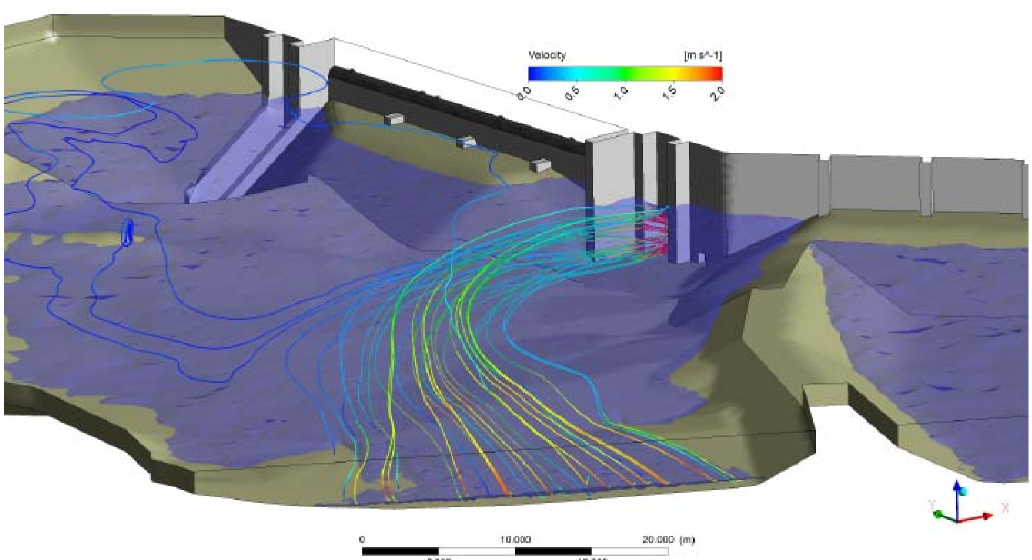

Figure 4: Streamlines through the headpond for sluicing activities

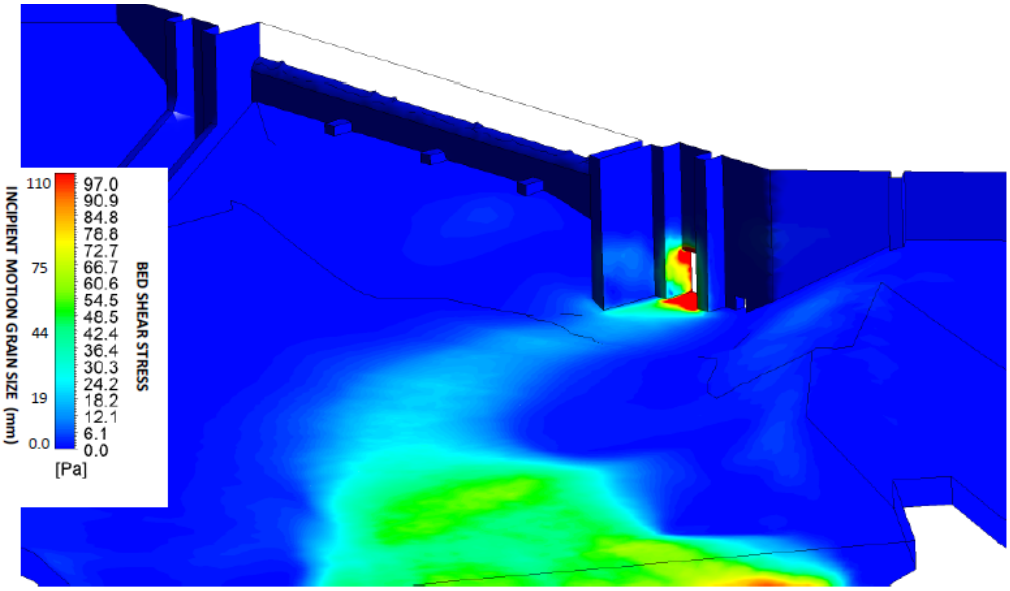

Figure 5: Shear stresses through the headpond for sluicing activities

4. PROJECT #3: EVALUATION OF AN INTAKE ORIENTATION FOR A PROPOSED PROJECT

The third project discussed in this paper is a proposed 77 MW run-of-river project with a rubber weir and an intake structure with three openings. It is located a short distance downstream of a tributary that is prone to landslides and debris floods. To protect the intake, a sediment deflection berm is proposed to be constructed within the headpond between the tributary and the intake. CFD modelling using ANSYS Fluent was undertaken to evaluate the flow patterns through the headpond and the intake openings, and to determine the best orientation of the intake to achieve the most uniform flow distribution through the intake openings, with and without the sediment deflection berm in place.

Four variations of the intake and headpond geometry were investigated for the project design flow of 56.5 m^3/s. For all four scenarios, there was no flow over the rubber weir with all flow passing through the three intake openings. The locations of the sediment deflection berm and the two tested intake orientations are indicated on Figure 6. The four modelled scenarios considered are as follows:

- Preliminary Intake design (Intake A - *green*) without the berm
- Preliminary Intake A with the berm
- Modified Intake design (Intake B - *maroon*) without the berm
- Modified Intake B with the berm

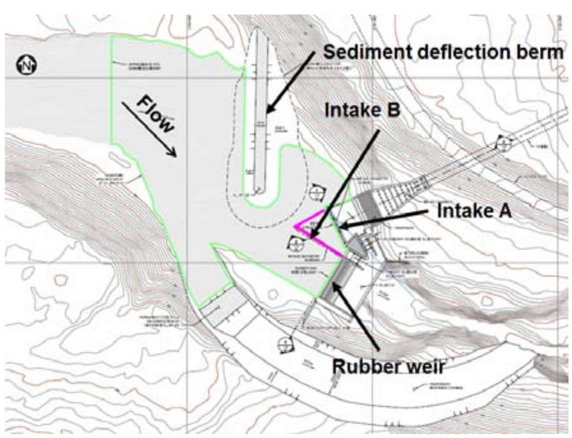

Figure 6: CFD model setup with the key intake elements

4.1. Hydrodynamic Modelling and Discussion

Figures 7 (A) and (B) show the velocity distribution through the headpond and the intake openings for Intake A without and with the sediment deflection berm in place, respectively. For Intake A without the sediment deflection berm, the bulk of the flow appears to be confined to the existing river channel closer to the left (north) headpond bank, which represents the deepest sections of the headpond, with lower flow velocities higher up the existing river banks (Figure 7A). In this case, the orientation of the intake openings is at an acute angle to the river flow, which results in a non-uniform flow distribution through the intake openings. The flow velocities within each opening range from less than 0.3 m/s to over 1.1 m/s. The addition of the berm redirects the flow over a shallow bank south of the berm creating a high velocity region in that area (Figure 7B). The flow and the streamline direction is more perpendicular to the intake openings, resulting in more uniform flow velocities of approximately 1 m/s through all three openings, and with only a very small area of the openings remaining less effective.

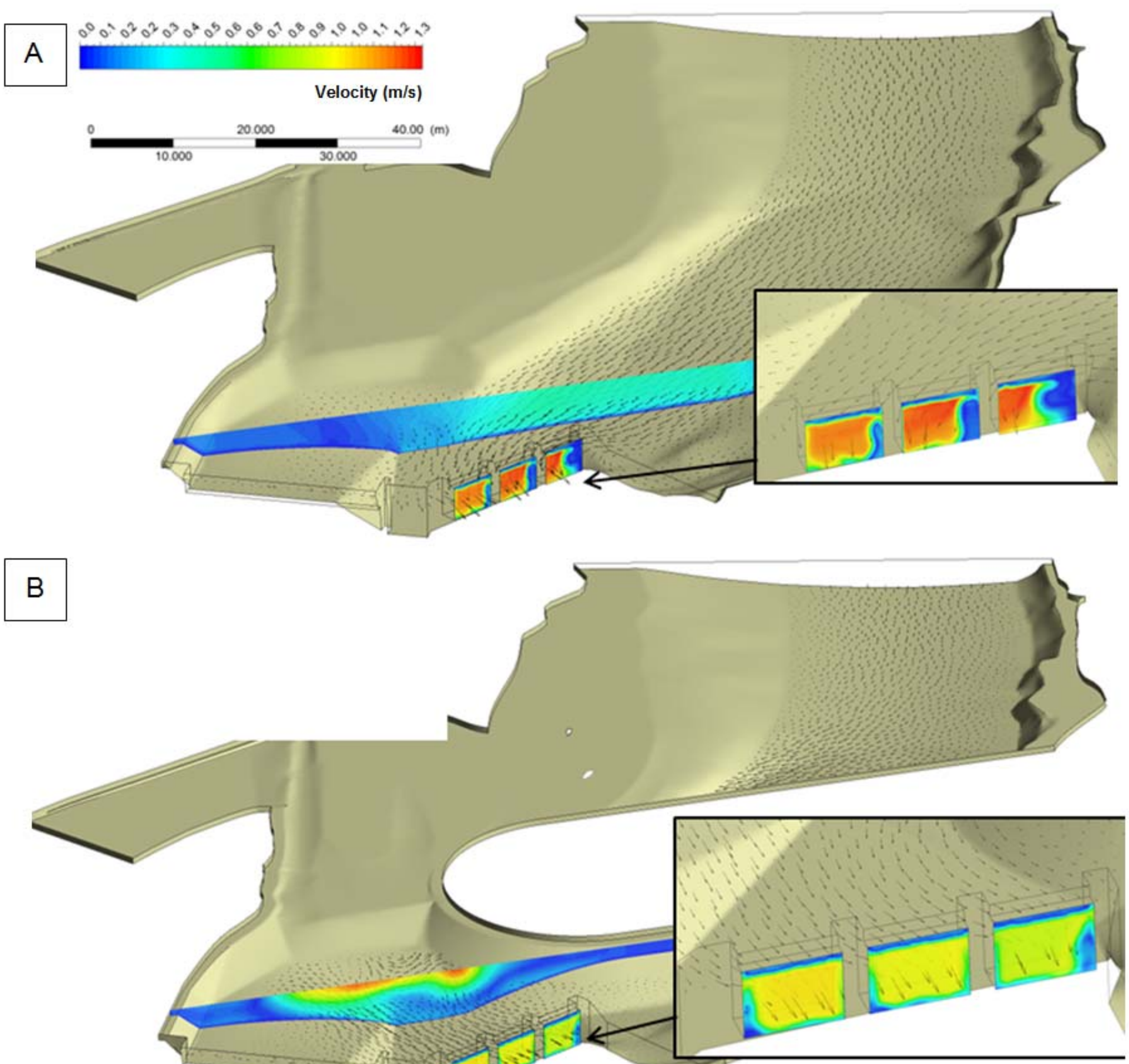

Figure 7: Velocity distribution for Intake A: (A) without a sediment deflection berm, and (B) with a sediment deflection berm

Figures 8 (A) and (B) show the velocity distribution through the headpond and the intake openings for Intake B without and with the sediment deflection berm in place, respectively. The alteration of the angle of the intake openings necessitates an extension of the western wall of the intake structure. This wall extension prevents the flow from directly impinging on the intake openings, deflecting the flow around the wall. The flow then appears to enter the intake openings from a more direct angle. Results for Intake B without the berm indicate that the flow is not confined as much to the original streambed and the deepest section of the headpond closer to the openings. The flow distribution through the intake openings appears to be somewhat more uniform than in the case of Intake A (Figure 8A). As a result, the resulting peak velocity magnitudes for all openings are lower than for Intake A. The addition of the berm results in similar flow patterns as for Intake A with a berm, including the region of high velocities around the berm, the improved flow uniformity through the intake openings, and the highly non-uniform flow between the berm and intake.

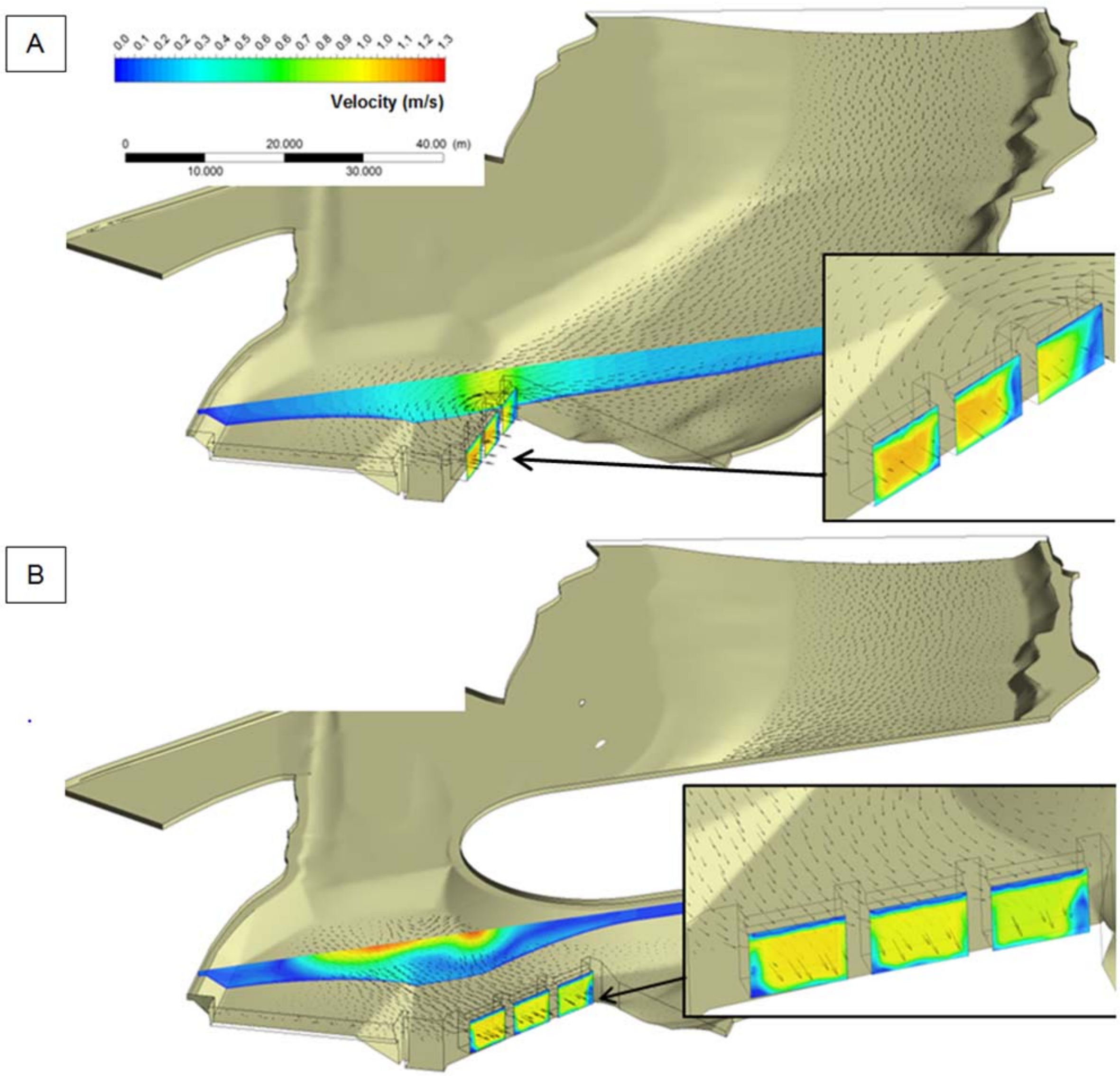

Figure 8: Velocity distribution for Intake B: (A) without a sediment deflection berm, and (B) with a sediment deflection berm

4.2. Summary for Project #3

Based on the modelling results, it is apparent that both the intake orientation and the placement of the sediment deflection berm are likely to affect the flow uniformity through the intake openings. Without the berm in place, Intake B resulted in somewhat improved flow uniformity compared to Intake A. The placement of the berm modifies the flow direction through the headpond and results in better flow uniformity when combined with Intake A.

5. CONCLUSIONS

Hydrodynamic modelling is often being used for run-of-river hydroelectric projects to develop a better understanding of large scale hydraulics of intake structures and associated headponds, and to support and fine-tune the design of various intake components. It is a cost effective way to investigate how various design options alter the flow patterns and modify various parameters that are of importance for the modelled project. In this paper, results of hydrodynamic modelling for one existing and two proposed run-of-river projects were reviewed. Various design options were tested to determine the impact of these options on the functioning of intake structures and further inform the decisions related to design improvements.

6. ACKNOWLEDGEMENTS

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