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Newfoundland and Labrador Hydro
Lower Churchill Project
Pre-Feed Engineering Services

Muskrat Falls Hydroelectric Project

MF1250 – Numerical Modeling of Muskrat Falls Structures

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Newfoundland & Labrador Hydro

LOWER CHURCHILL PROJECT

TECHNICAL REPORT

MF 1250 – Numerical Modeling of Muskrat Falls Structures

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FINAL **May 2008**

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EXECUTIVE SUMMARY

In WTO MF1010 – Review of Variants, a layout and cost review was carried out on the short-listed variants presented in the January 1999 - Final Feasibility Study for Muskrat Falls, in particular Variants 7, 10 and 11. In WTO MF1050 – Spillway Design Review, it was concluded that Variant 10 represents the best alternative since it offers a considerably shorter construction schedule and better alternatives for the spillway and site access.

For this WTO, the proposed variant from the WTO MF1050 study was evaluated by numerical modeling. The various facilities related to this variant were analysed and improved when necessary. These facilities are:

- Diversion channels;
- Powerhouse (approach channel and tailrace channel);
- Spillway (sluices and overflow crest).

Numerical modeling has shown that the proposed approach channel for the diversion facilities should be modified to improve the flow conditions for the left sluices. A curved wall was added, which improves the flow conditions and increases the discharge capacity of the system.

At the power intake, the flow pattern at unit No. 1 (near the spillway) is disturbed by a strong lateral velocity, which can reduce the unit efficiency. Different variants were considered and it appears that a longer and higher retaining wall will be required combined with an increase of excavation of the approach channel. This alternative improves the hydraulic conditions at the power intake considerably, but it should be further optimized to minimize the construction costs.

The analysis of the spillway facilities shows the presence of a vortex upstream of sluice No. 1. This vortex disturbs the flow pattern and probably reduces the discharge capacity of the system.

Final optimization of the layout should be performed by numerical and/or physical modeling after the review of the layout based on an update of the hydraulic conditions at Muskrat Falls and the results of the present report.

1 INTRODUCTION

The previous 1999 study recommended a preliminary layout of structures that became the basis of the 2007 program. This program would identify and study potential design and schedule improvements based on the updated field investigation. It will include the optimization of the structure locations (approach channel, power plant intake, power plant and spillway) and the redesign of the diversion and spillway facilities with advances in technology.

One such advance is the use of 3D modeling, and in particular the use of the Flow-3D hydraulic model (Reference 1) and CATIA software. With these tools the geometry of the river and all facilities structures can be modeled and analysed in the Flow-3D numerical hydraulic model.

The use of 3D modeling has been taking place more and more frequently in recent years, especially in the domain of hydraulic modeling, because the modeling methods can more accurately describe the complexity of the flow phenomena through hydraulic structures.

The aim of the present study was to test and validate, using Flow-3D software, the selected design and to propose any necessary improvements to the selected layout. The selected variant is described in detail in separate reports related to WTO MF1010 and WTO MF1050 (Reference 2).

The numerical hydraulic modeling covers the following facilities:

- Diversion works;
- Power intake, approach channel and tailrace channel;
- Spillway Facilities.

The hydraulic structures were modeled and analyzed in detail for conditions expected to exist either during the diversion phase (construction period) or during the operations phase. The design of the structure layouts was reviewed when necessary and an improved design proposed for the hydraulic facilities.

2 SCOPE OF WORK

The activities performed in the present study were:

- Preparation and calibration of the numerical model based on natural conditions;
- Evaluation of the hydraulic conditions during diversion: The diversion facilities comprise an approach channel, four gated sluiceways and a discharge channel;
- Evaluation of the hydraulic conditions at the power intake including the approach channel and the power plant tailrace channel;
- Evaluation of the hydraulic conditions at the spillway facilities. The facilities comprise four gated sluiceways and an overflow spillway.

3 METHODOLOGY

3.1 SOFTWARE DESCRIPTION

The FLOW-3D model, developed and commercialized by Flow-Science Inc, was used for the present study. This numerical model uses the CFD (Computational Fluid Dynamics) principle to simulate the hydraulic conditions in 3D. The free surface of the water is calculated using VOF (Volume of Fluid) techniques, which allow fixing the water surface, to model the movement of water and to apply the associated boundary conditions to the surface.

FLOW-3D uses the finite difference method to numerically solve the Navier-Stokes and mass conservation equations by an iterative procedure. The value of each dependant variable is associated with each mesh and applied to the centre of the mesh except the velocity, which is applied to the face of the mesh. In order to solve the mass conservation and movement equations, the following procedure is applied at each time step:

- The explicit procedure is used to evaluate the variable's associated velocities in the Navier-Stokes equation at a given time using the initial conditions or variable value at the previous time step.
- Water pressure is evaluated in each mesh and iterations are used to advance a solution through a sequence of steps from a starting state to a final, converging state. The mesh corresponding velocities are then adjusted;
- The VOF method detects the free surface and computes the new configuration;
- The turbulence model "Renormalized Group (RNG) Model" with a Newtonian viscosity is used for the analysis.

Version 9.2.1 of the software was used in the present study.

3.2 MODEL PREPARATION

The 3D model of the study area (bathymetry and topography) and the structures (dam, cofferdams, spillway, power intake, and channels) were prepared with CATIA and the information transferred to Flow-3D.

3.3 MODEL CALIBRATION

The next phase of the study was the calibration of the hydraulic model. The calibration process indicated the size of the mesh required to accurately represent the hydraulic conditions and the roughness coefficients for the study area.

For the present case, the only information available was the water level obtained in natural conditions from three hydrometric stations located upstream and downstream of the site and the calibration was performed considering these conditions.

3.4 EVALUATION OF THE PROPOSED LAYOUT

After the calibration of the model, the proposed layout was analysed with Flow-3D. Considering the dimensions of the study area, the size of the mesh required to accurately represent the geometry of the structures and the different phases of operation, the analysis of the proposed layout was performed in three (3) phases:

- Analysis of the diversion conditions during the construction period;
- Analysis of the power intake with and without the operation of the spillway;
- Analysis of the spillway for the probable maximum flood (PMF) and for the maximum normal operation level.

3.5 LAYOUT OPTIMIZATION

Based on the analysis of the proposed layout, possible problems and constraints were identified and modifications to the layout were proposed and evaluated with the simulation model.

4 MODEL DESCRIPTION

4.1 STUDY AREA

The study area extends about 600 m upstream of the RCC main dam and about 900 m downstream of the dam and covers all the facilities. The limits of the study area were determined to avoid any impact from features upstream and downstream of the boundaries. Figure 4-1 presents the study area and the proposed layout of structures.

4.2 NATURAL GROUND

The natural ground of the study area (topography and bathymetry) was based on the information available in the 1999 report (Reference 3). The 3D model of the natural ground was prepared in CATIA and transferred to Flow-3D. Data from the LiDAR mapping could have improved the natural ground representation, however the impact on the results should be relatively small as there is no major difference between the two (2) sources.

4.3 DESCRIPTION OF THE PROPOSED LAYOUT

The proposed scheme from MF1050 was considered as the proposed layout for the present study. The diversion facilities are based on four (4) submerged radial gates, 12.5 m wide by 14.8 m high with permanent sills at elevation 5.0 m. There is no rubber dam and it is not necessary to have an overhead service bridge above the fixed crest of the north dam.

During the initial construction phase, the river will remain in its normal channel (until Year 3) then it will be diverted through the spillway sluices located in the south shore between the powerhouse and the RCC dam. The spillway has sufficient capacity to release the 1:40 year flood and will have control gates to maintain a high forebay level in winter for frazil ice control.

The power intake was not revised in the 2007 studies; the proposed layout from 1999 was therefore used for the numerical analysis.

4.4 BOUNDARY CONDITIONS

The boundary conditions must be defined to adequately represent the characteristics of the flow pattern and the conditions outside the limits of the model. Table 4-1 summarizes the typical boundary conditions considered for each type of simulation.

Table 4-1: Summary of Boundary Conditions

Boundary Conditions	Diversion Facilities	Power Intake	Spillway Facilities
Upstream	Water velocity + water elevation	Water velocity + water elevation	Water velocity + water elevation
Downstream	Water elevation with stagnation pressure	Water velocity	Water elevation with stagnation pressure

4.5 SUBDIVISION OF THE STUDY AREA

For simulation purposes, the study area was divided into three (3) blocks:

- the first block covers the upstream part of the river;
- the second block covers the structures area and;
- the third block covers the downstream part of the river.

The size of the meshes of the upstream and downstream blocks are 8 m x 8 m x 4 m, which makes it possible to represent the general flow conditions in these sectors without significantly increasing the calculation time. The size of the meshes for the second block, which includes the dam, cofferdams, sluiceway, overflow spillway and the powerhouse, are subject to mesh refinement to improve the representation of the hydraulic conditions. Typically, the size of the meshes in the second block is 4 m at the beginning of the simulation and is reduced progressively to reach 1 m or even 0.5 m.

Figure 4-2 presents the blocks and meshes considered for the simulation of the Muskrat Falls system.

4.6 MODEL CALIBRATION IN NATURAL CONDITIONS

For the calibration of the numerical model, the only information available was the water levels available from four (4) hydrometric stations near the dam site. The hydrometric stations are presented in Table 4-2 and their locations are shown on Figure 4-3. Two (2) of the hydrometric stations are located downstream of the site (Stations 03OE004 and 03OE007), one (1) is located upstream of Muskrat Falls (Station 03OE001) and the fourth one is located between the two (2) falls (station 03OE005); the discharge was determined based on the water level measured at station 03OE001. For simulation purposes, the downstream water level for a specific discharge was given as an input to the model and the water levels estimated at two (2) upstream stations were compared to the observed discharge. Figure 4-4 illustrates the simulation results in natural conditions.

A discharge of 1,880 m^3/s[1] was first considered for the model calibration; the water levels at the two (2) upstream stations were respectively 17.05 m and 9.68 m. However, the water level calculated with Flow-3D was found to be about 1 m below the observed water level when the roughness coefficient for the river remained in its normal range. Considering the fact that the hydrometric stations are located near the falls, a major change in the roughness coefficient of the river would be required to obtain the observed water level. Most probably, the bathymetric data at the control section of both waterfalls does not reflect the real conditions. Other discharge conditions were reviewed but the same problem occurred.

It might be possible to modify the geometry of the control section of the waterfalls, but it would have no real impact on the present study. For simulation purposes, the river roughness coefficient considered at the Gull Island site was retained for the Muskrat Falls site study.

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1 The value of 1,880 m^3/s was chosen after a review of the data at the hydrometric stations during the open surface period (to avoid ice effect).

Table 4-2: Muskrat Falls - Hydrometric Stations (*)

Station	Name	Latitude	Longitude	Operation
03OE001	Churchill River above Upper Muskrat Falls	53º14’52” N	60º47’21” W	1953-Present
03OE004	Churchill River below Lower Muskrat Falls	53º14’46” N	60º42’38” W	1979-1980
03OE005	Churchill River between Lower and Upper Muskrat Falls	53º14’39” N	60º46’24” W	1978-1994
03OE007	Churchill River at foot of Lower Muskrat Falls	53º14’57” N	60º46’08” W	1980-1995

(*) From Environment Canada

Figure 4-1: Muskrat Falls - Study Area and Proposed Layout of Structures

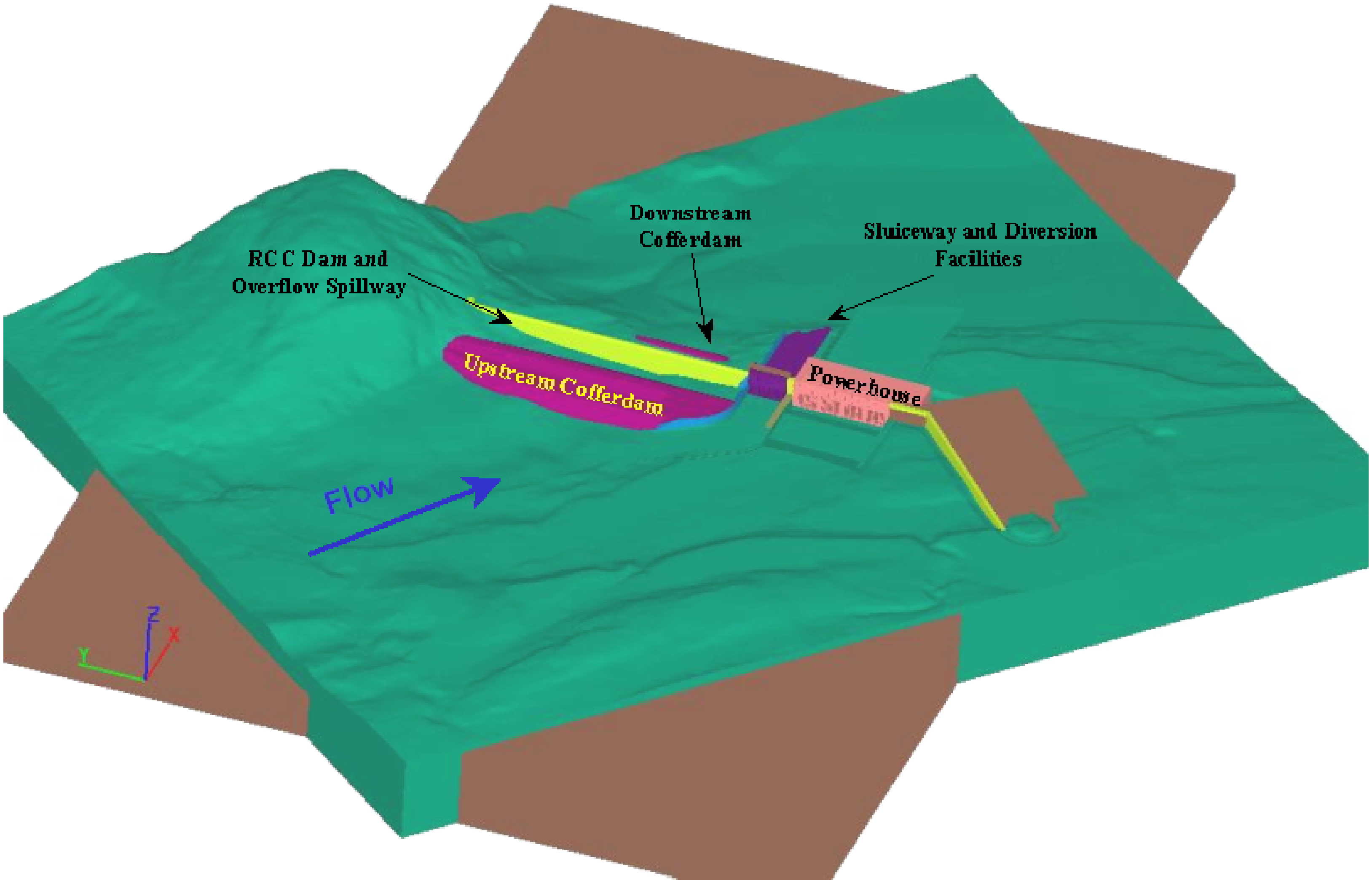


Figure 4-2: Muskrat Falls - General View of the Mesh Blocks

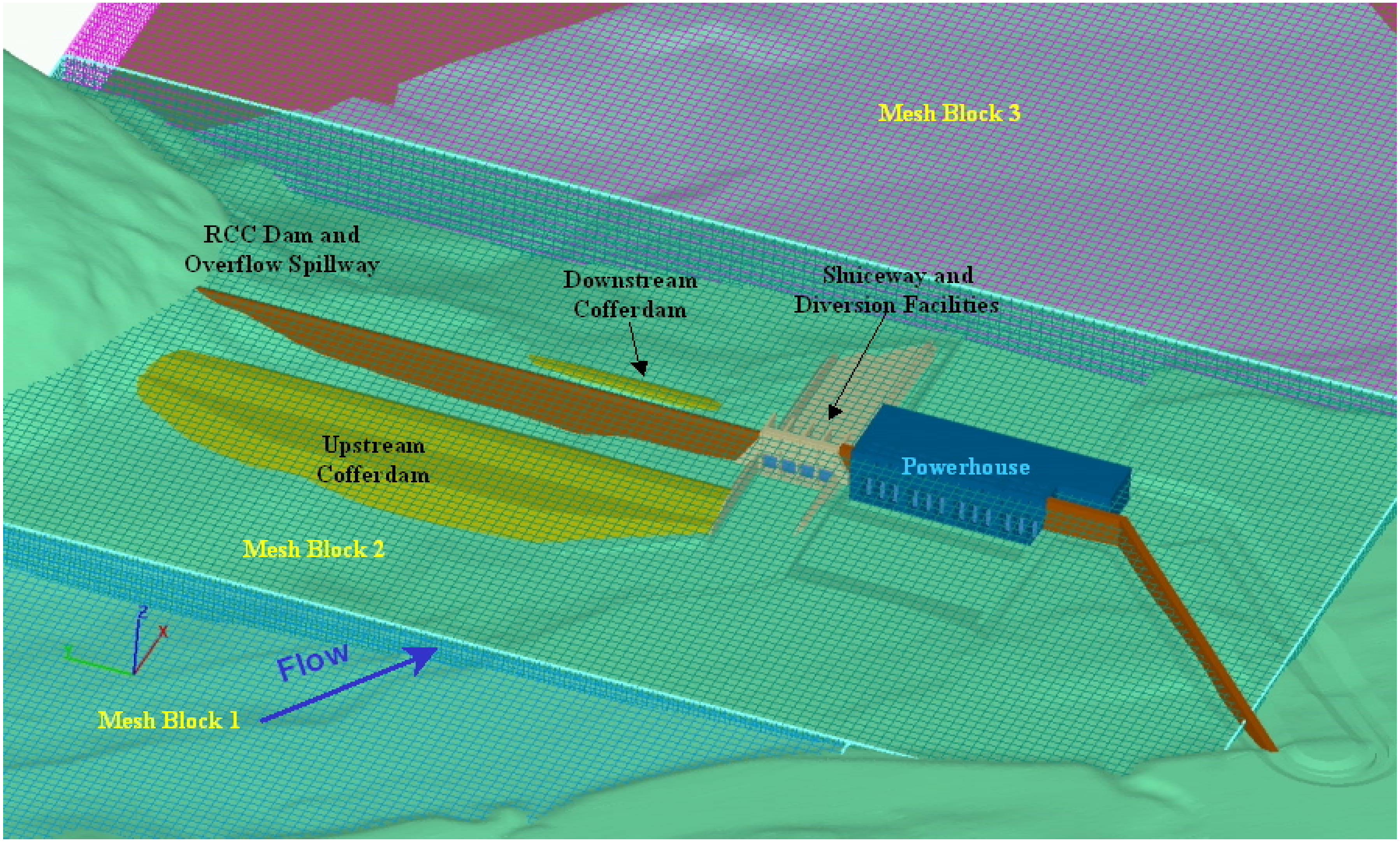


Figure 4-3: Muskrat Falls - Location of Hydrometric Stations

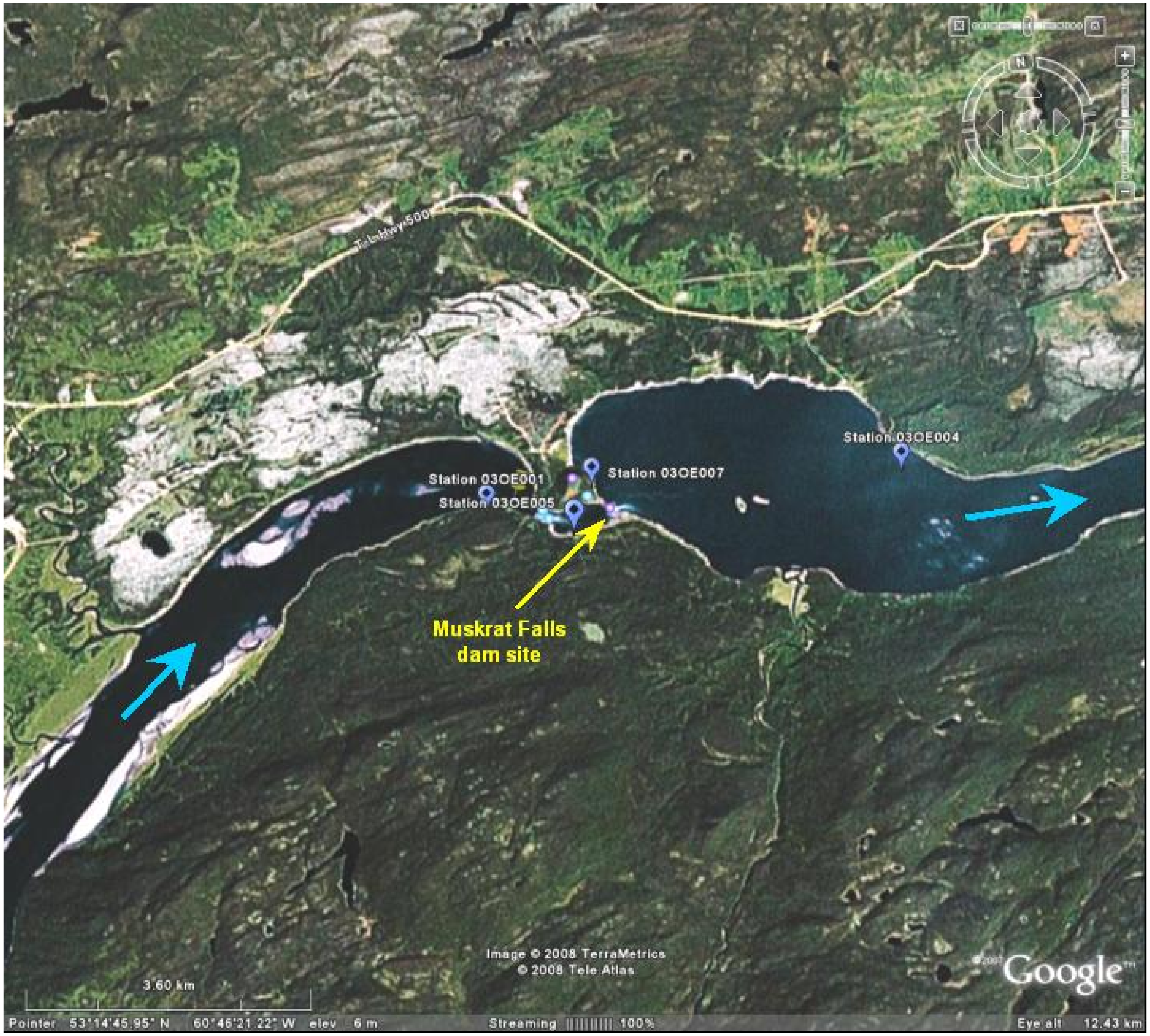
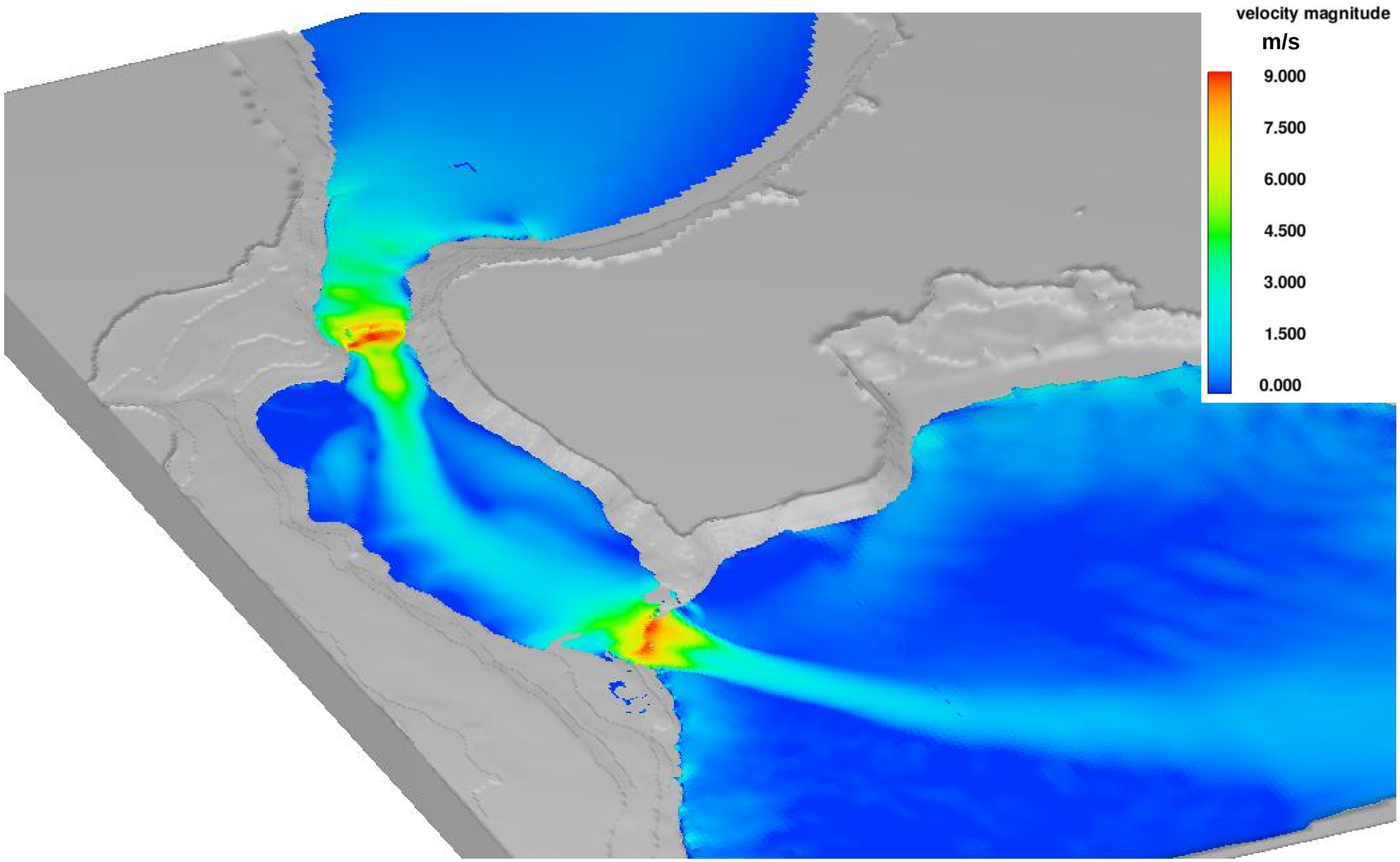


Figure 4-4: Muskrat Falls - Model Calibration - Simulation Results



5 DIVERSION FACILITIES

5.1 DIVERSION FACILITIES CHARACTERISTICS

The proposed diversion scheme is based on diverting the river through the spillway facilities (Reference 2). The construction of the spillway will be performed over a two (2) year period and the water will be diverted only during the third year. It must be noted that the scheme allows the possibility of controlling the water level for the third year to maintain a water level of 24 m in winter, in order to create upstream conditions favourable for the creation of an ice cover, thereby minimizing frazil ice formation during this period.

The main characteristics of the diversion facilities are (Reference 2):

- Design flow (1:40 year flood): 5 300 m³/s [2]
- Number of sluices: 4
- Type of gates: Submerged radial
- Width: 12.5 m
- Height: 14.8 m
- Sill elevation: 5.0 m
- Forebay level for design flow: 21.7 m
- Upstream level in winter for frazil control: 24.0 m

Appendix A presents a drawing of the proposed variant from the 2007 study.

[2] The design flood discharge for the diversion is based on the 1999 study and modified operation rules at Churchill Falls to reduce the peak discharge. A larger value for the flood was determined in the 2007 studies and the larger value will be used in any future modeling.

5.2 ANALYSIS OF THE PROPOSED LAYOUT

As mentioned previously, the river will remain in its normal channel for the initial phase of construction. In the third year, the river will be diverted through the spillway sluices located in the south shore. The spillway will have the capacity to release the design flow and will have control gates to maintain the forebay level in winter at elevation 24 m for frazil ice control.

Figures 5-1 to 5-3 illustrate the hydraulic conditions at the diversion facilities for the design flow (upstream water level of 21.7 m). At the downstream outlet (Figure 5-1), the conditions appear to be normal and the energy dissipation downstream of the channel occurs without problem. However, the hydraulic conditions in the approach channel and near the inlet (Figures 5-2 and 5-3) could be improved. The velocity on the north side of the channel near the wall between the sluices and the cofferdam is much lower than the average. It appears that the angle at the upstream end of the retaining wall generates a zone of low velocity, which reduces the capacity of sluice No. 1.

Figure 5-4 illustrates the hydraulic conditions at elevation 11 m and confirms the previous conclusions. This figure provides a good indication of the flow direction near the left wall and indicates that this sector is not contributing to the flow going through sluice No. 1. Figure 5-5 shows a cross section of the four (4) sluices. The flow conditions of sluices No. 3 and No. 4 are good, but the lateral component of the velocity is more significant in sluices No. 1 and No. 2. It also shows that the water level is lower in sluice No. 1 than in the other sluices, which explains the reduction of capacity.

Figure 5-6 presents the discharge through each sluice and shows the reduction of capacity of sluice No. 1 compared to the others; the total discharge of the sluices is 5,180 m^3/s instead of 5,300 m^3/s. In this case, the upstream water level will be higher than the expected value (21.7 m) to release the design flow, but it will remain lower than the upstream water level for frazil control.

5.3 LAYOUT OPTIMIZATION

Considering the hydraulic conditions observed for the proposed layout, modifications of the approach channel were considered. Figure 5-7 presents the plan view of the hydraulic conditions at elevation 11 m for an approach channel with a curve of 75 m of radius to replace the angle at the upstream end of the left wall. This figure shows a better velocity distribution along the left wall, even if the water velocity is slightly lower along the wall than the other side.

A cross section through the four (4) sluices (Figure 5-8) shows much better flow distribution through the four (4) sluices. The water level appears to be almost the same in each sluice and the lateral velocity distribution is more uniform along the spillway.

Figure 5-9 presents the discharge through each sluice and shows a slight reduction of capacity in sluice No. 1 compared to the others, but the distribution is much better than that corresponding to the initially proposed layout. The total discharge of the sluices is as expected (5,295 m^3/s compared to 5,300 m^3/s). On the right side of the channel, the velocity in the curve is slightly lower than elsewhere. The excavation in the curve could probably be slightly reduced without having impact on the diversion capacity.

Considering the improvement on the flow distribution, the revised approach channel was integrated to the proposed layout for the study of the power intake facilities and the spillway facilities.

Figure 5-1: Muskrat Falls - Diversion Facilities - Proposed Layout Design Flow (5,300 m^3/s) - 3D View from Downstream

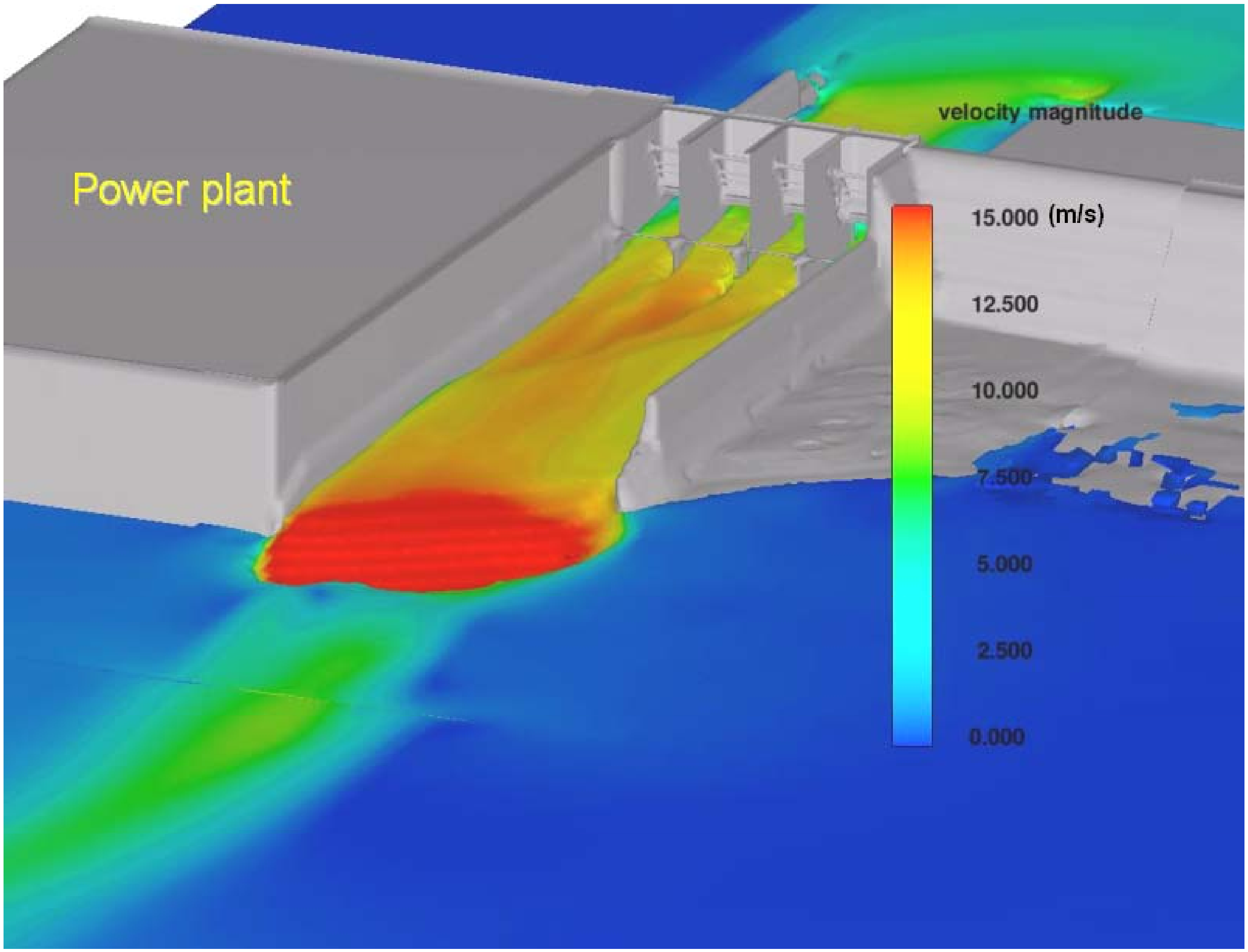


Figure 5-2: Muskrat Falls - Diversion Facilities - Proposed Layout Design Flow (5,300 m^3/s) - 3D View from Upstream

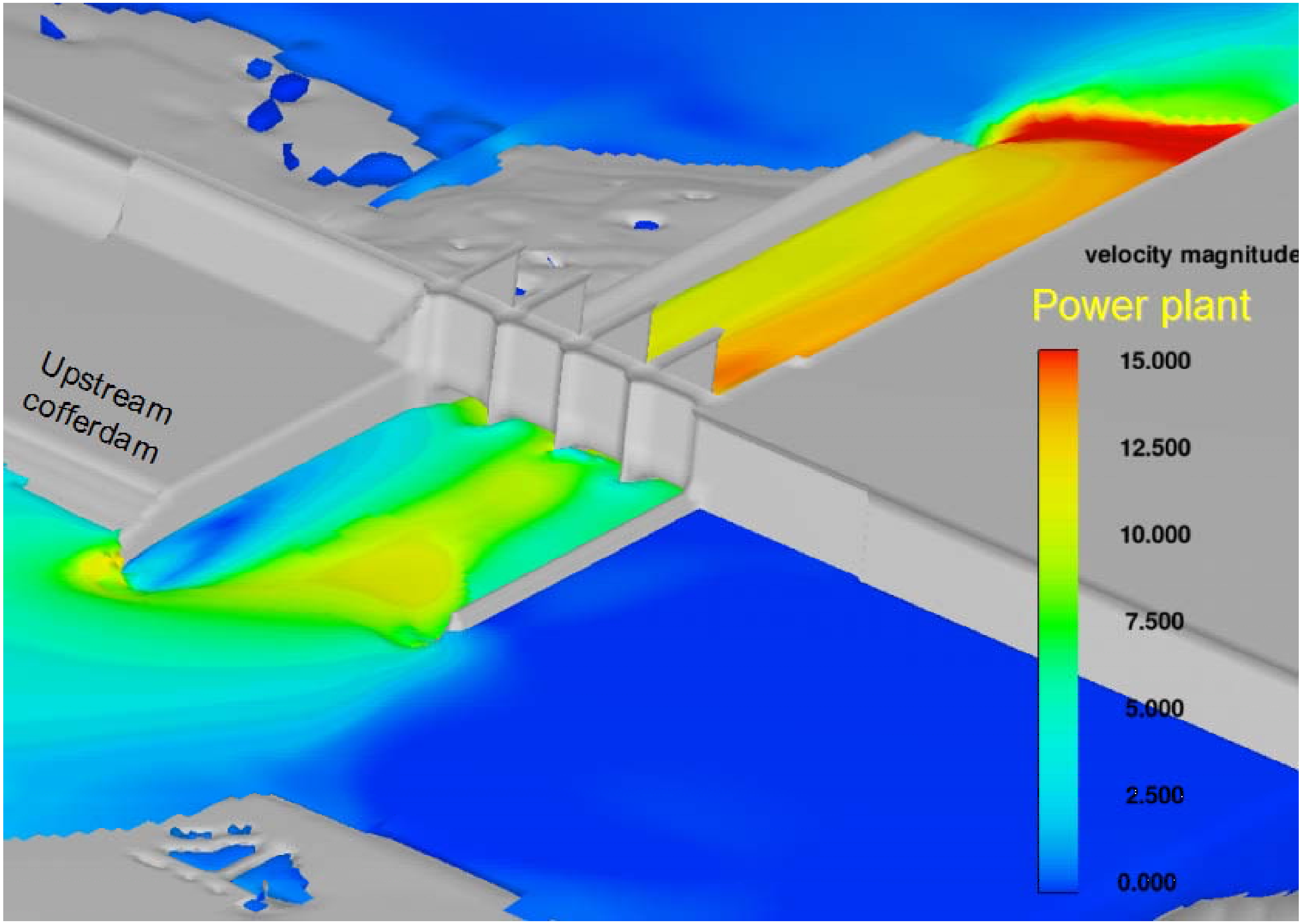


Figure 5-3: Muskrat Falls - Diversion Facilities - Proposed Layout Design Flow (5,300 m^3/s) - Plan View

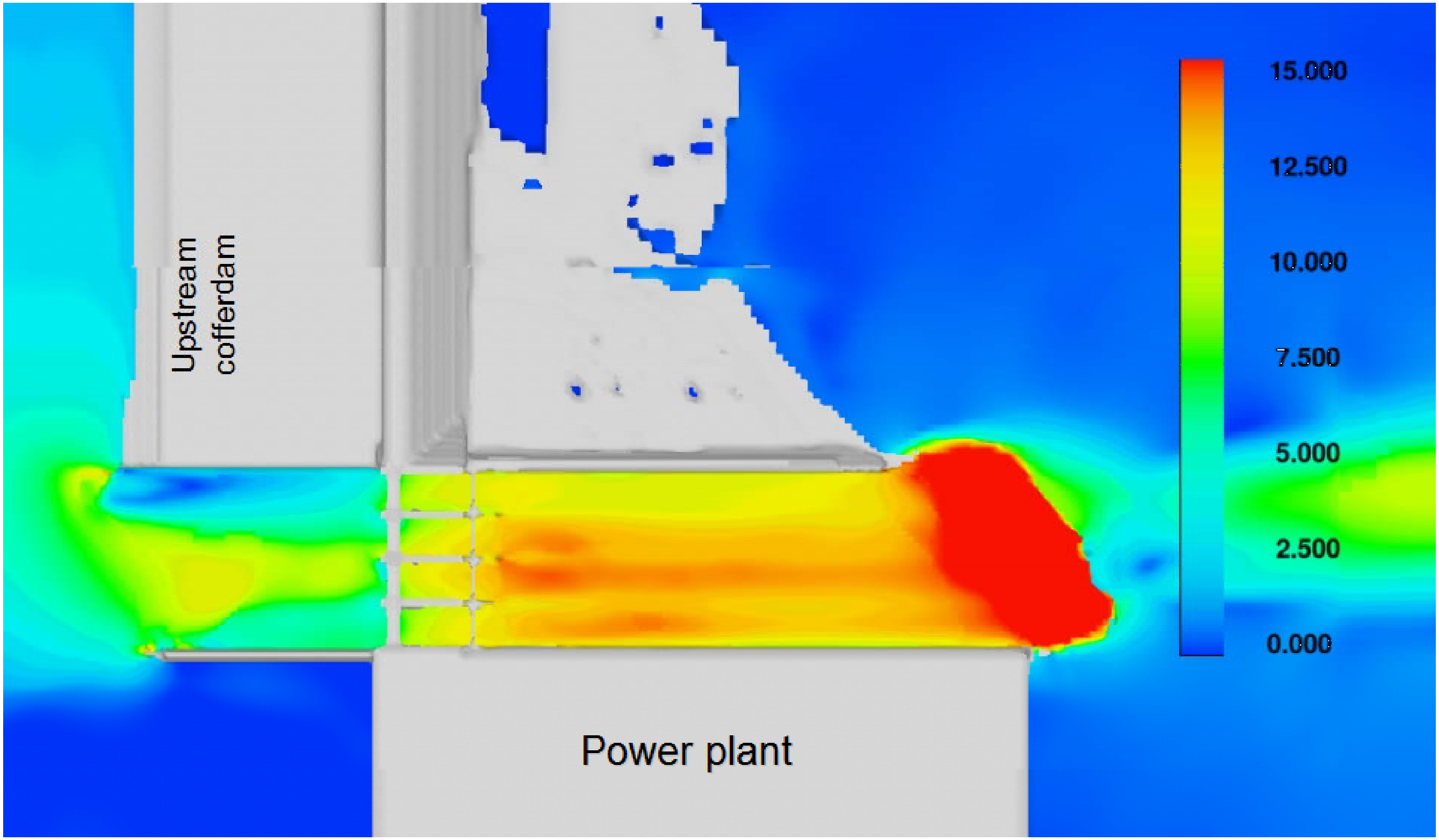


Figure 5-4: Muskrat Falls - Diversion Facilities - Proposed Layout Design Flow (5,300 m^3/s) - Plan View at Elevation 11 m

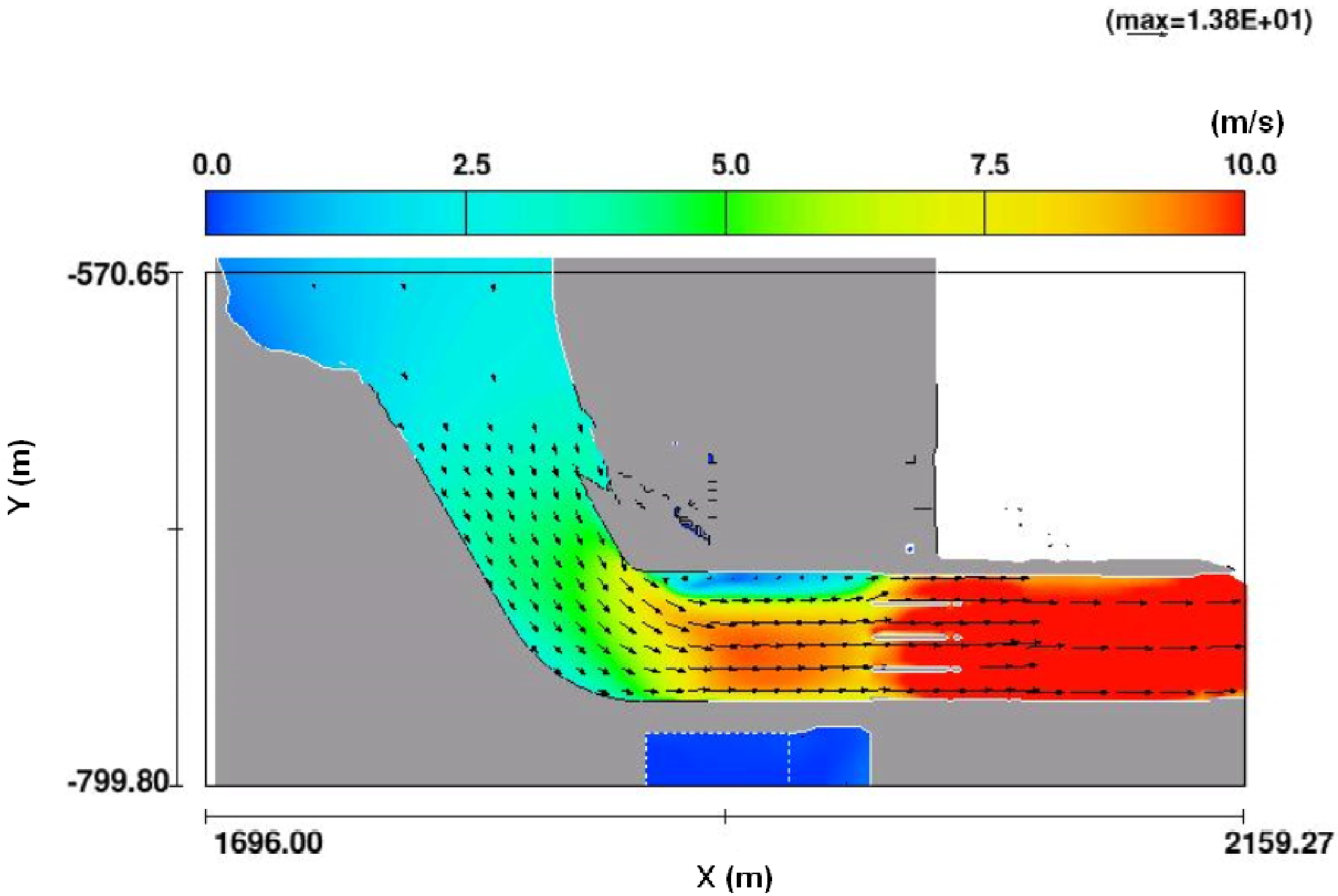


Figure 5-5: Muskrat Falls - Diversion Facilities - Proposed Layout Design Flow (5,300 m^3/s) - Cross Section View of the Four Sluices (looking upstream)

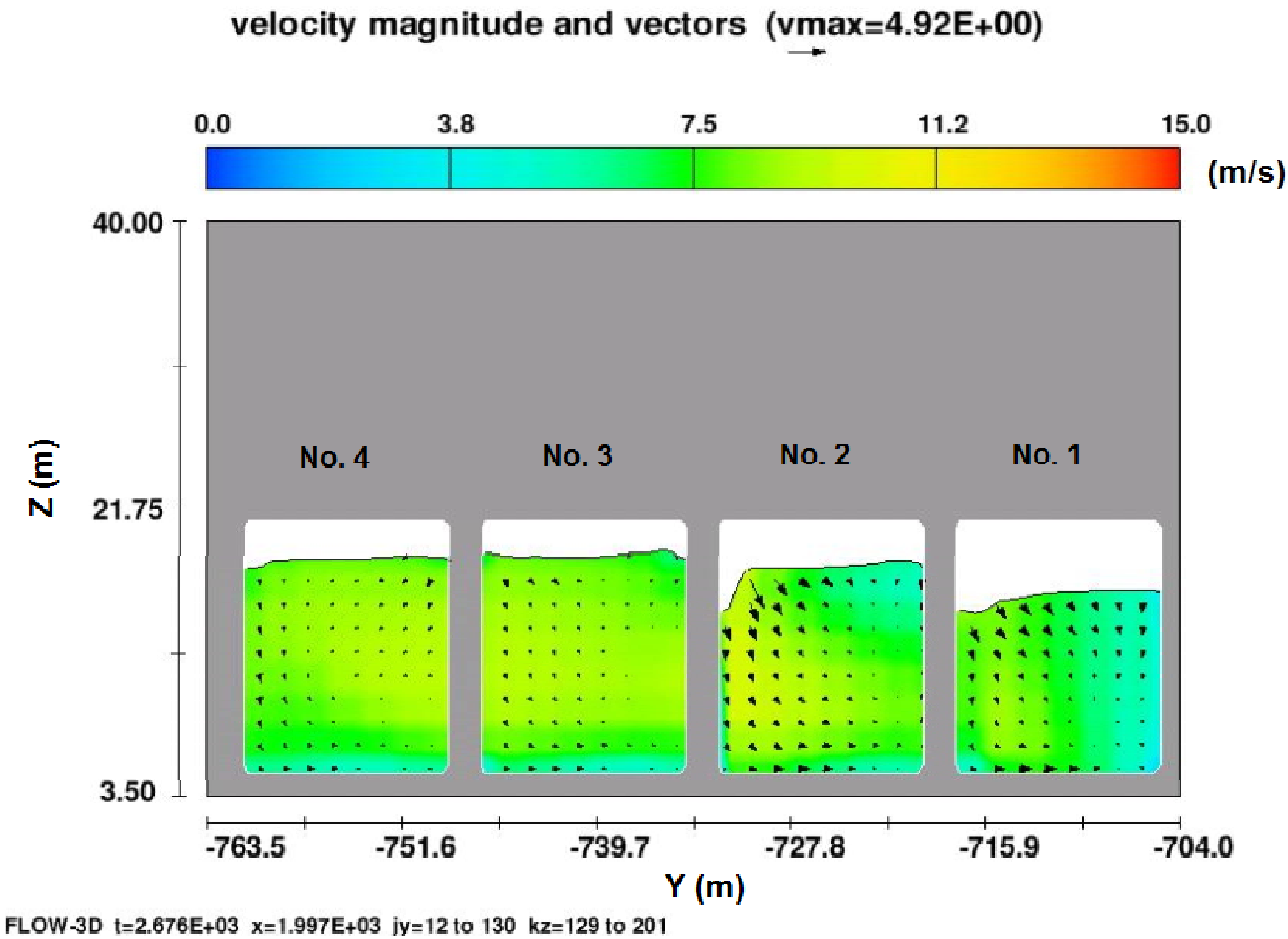


Figure 5-6: Muskrat Falls - Diversion Facilities - Proposed Layout Discharge Through the Spillway Sluices

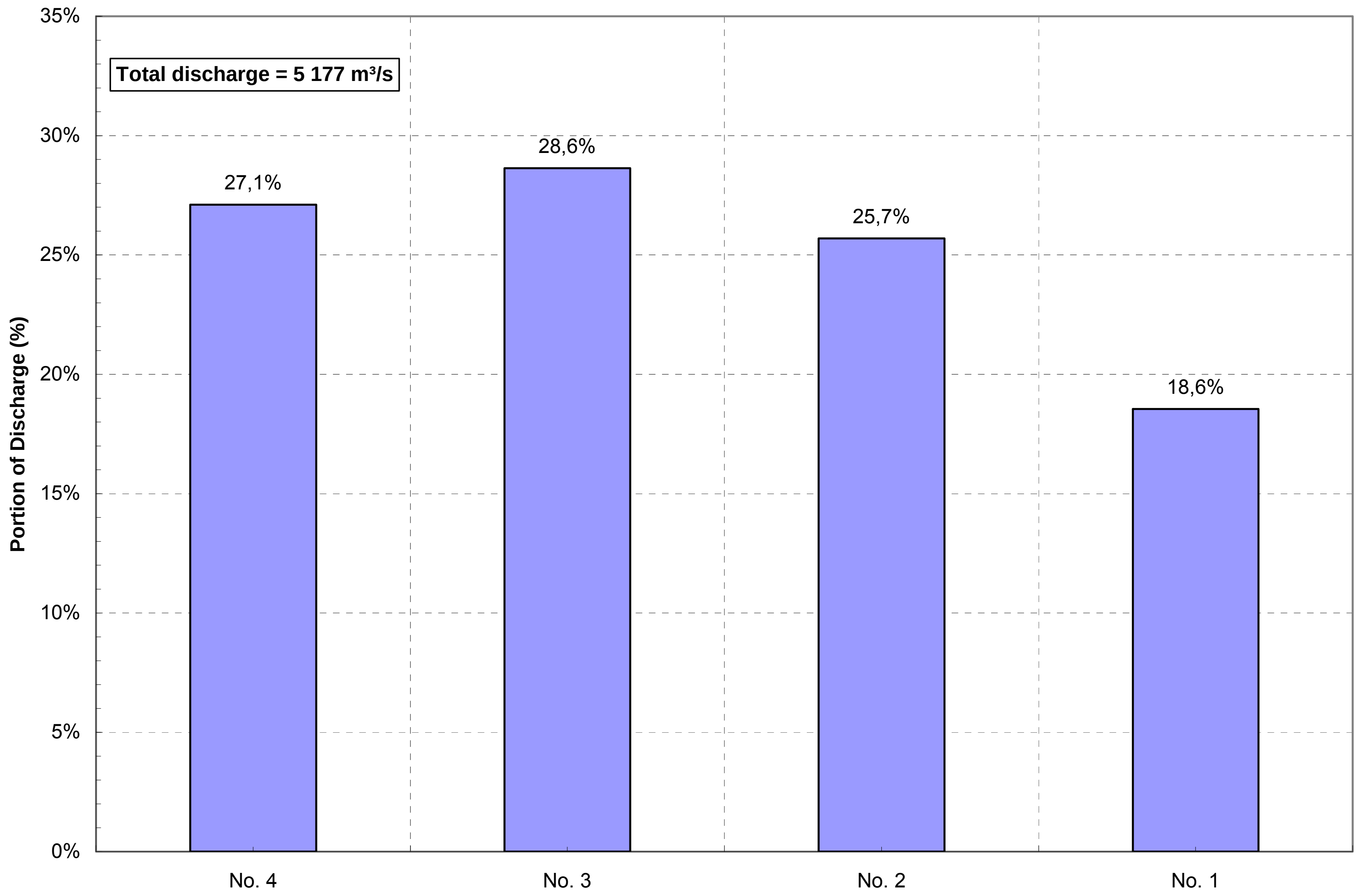


Figure 5-7: Muskrat Falls - Diversion Facilities - Proposed Layout with Revised Approach Channel Design Flow (5,300 m^3/s) - Plan View at Elevation 11 m

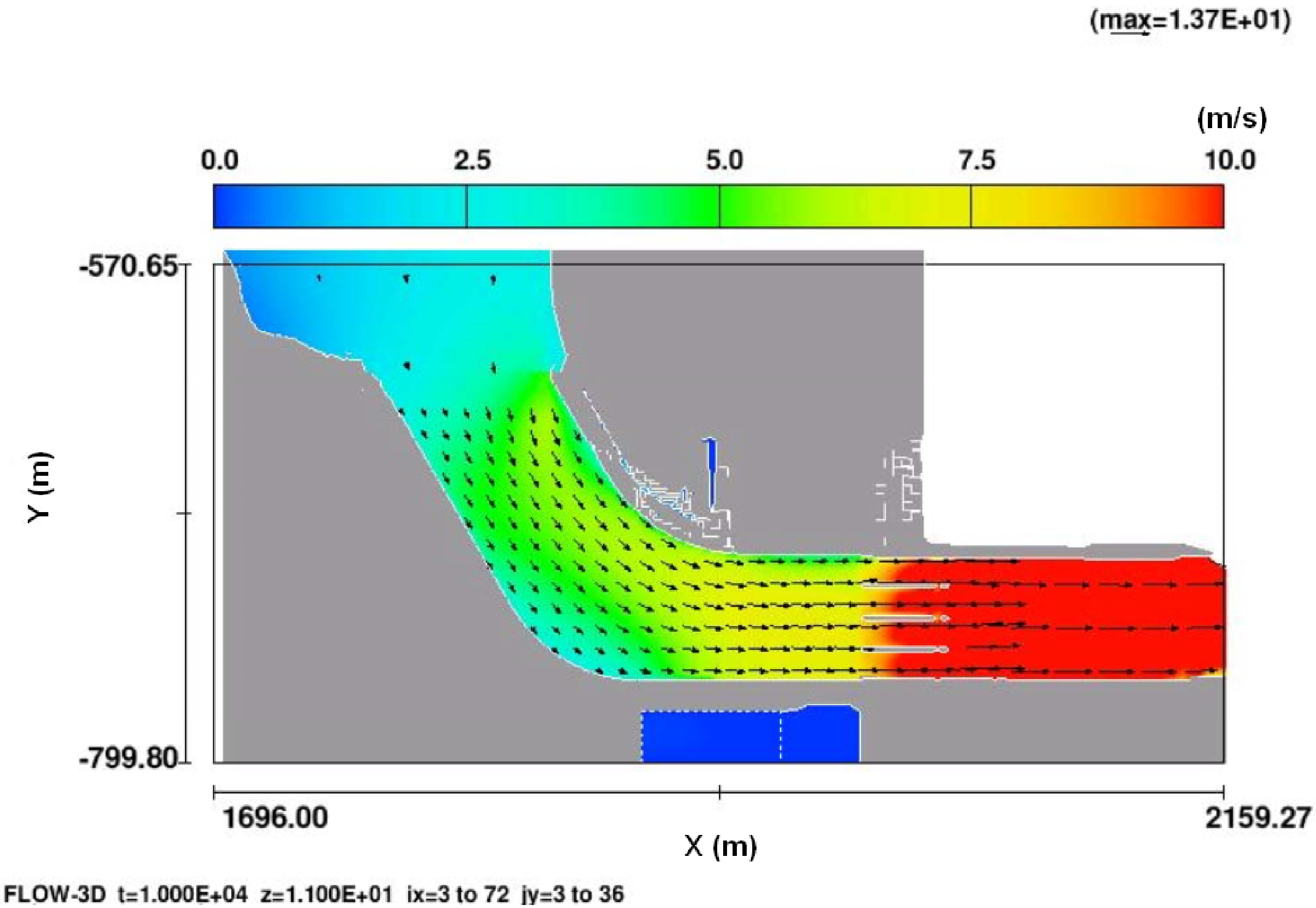


Figure 5-8: Muskrat Falls - Diversion Facilities - Proposed Layout with Revised Approach Channel Design Flow (5,300 m^3/s) - Cross Section View of the Four Sluices (looking upstream)

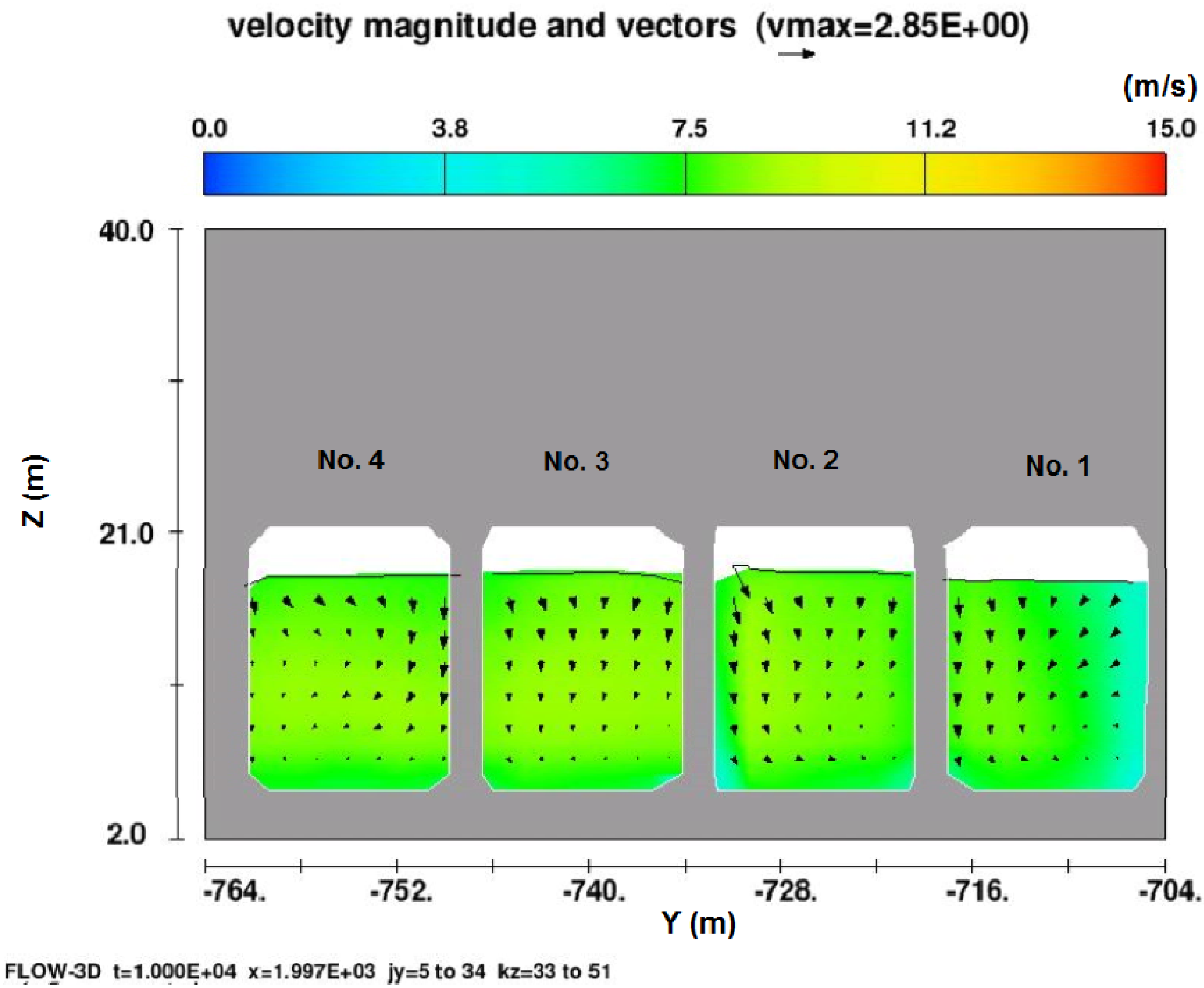
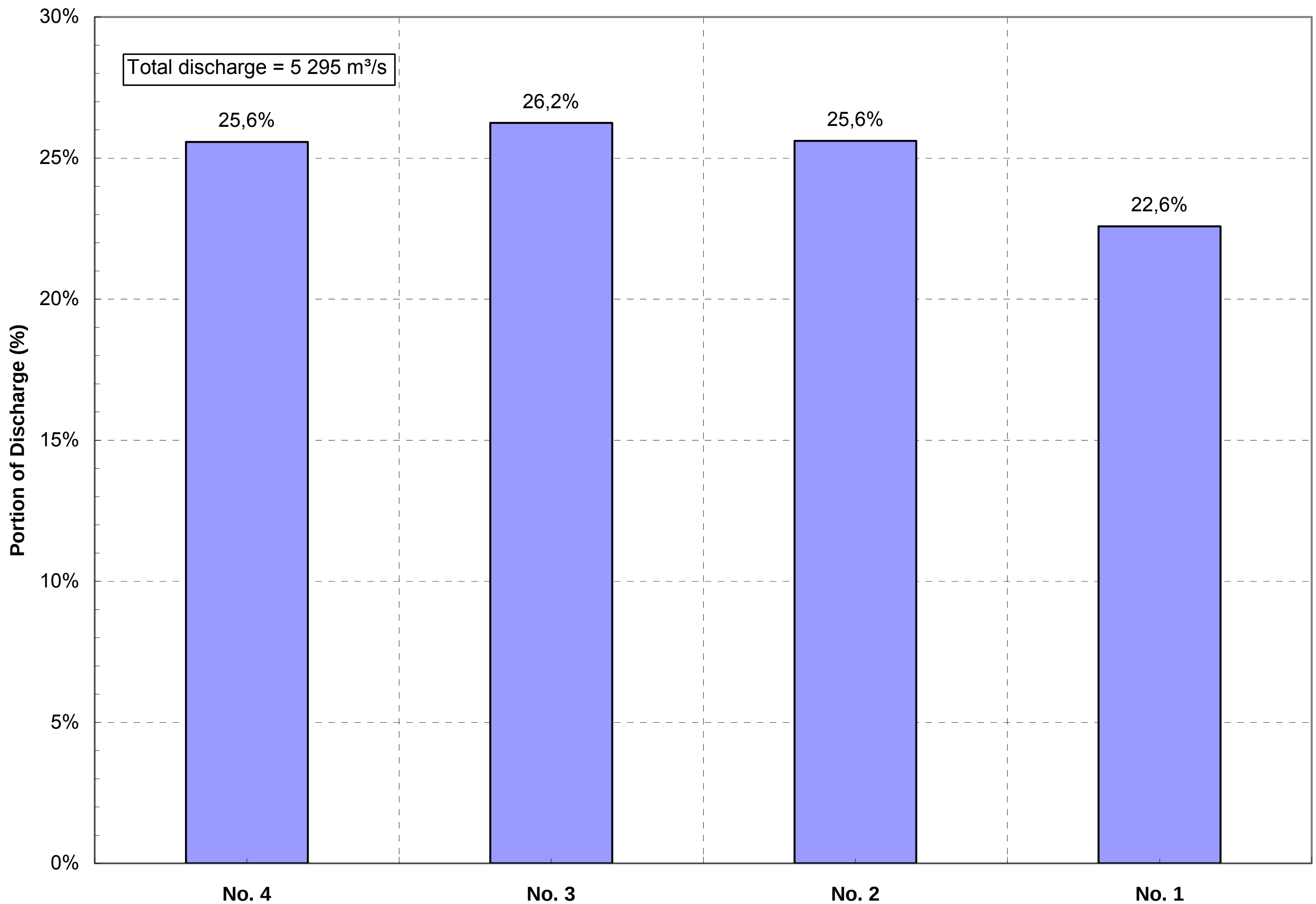


Figure 5-9: Muskrat Falls - Diversion Facilities -
Proposed Layout with Revised Approach Channel Discharge Through the Spillway Sluices



6 POWER INTAKE FACILITIES AND TAILRACE CHANNEL

6.1 POWER INTAKE FACILITIES CHARACTERISTICS

The total installed capacity at Muskrat Falls is 824 MW and the maximum discharge through each of the four units is 665 m³/s.

As mentioned in Section 5, the main variants proposed in the 1999 study were reviewed in 2007. The power intake facilities of the proposed layout (MF1050) are located on the right side of the spillway. The rock between the spillway and the power intake is at about elevation 20 m and a concrete retaining wall (crest at elevation 25 m) protects the power intake during diversion and divides the flow between both structures in normal operation.

The main characteristics of the intake facilities are (Reference 3)[3]:

- Approach channel
  - Invert elevation: 1.0 m
  - Width at invert: 138 m
- Retaining wall (between the power intake and the spillway
  - Crest elevation: 25.0 m
  - Length: 80 m
- Intake structure
  - Sill elevation: 3.0 m
  - Width: 144.5 m
- Number of units: 4

3 The characteristics of the Muskrat Falls power plant have not been revised in the 2007 studies.

- Maximum discharge per unit: 665 m^3/s
- Total capacity: 824 MW
- Intake head gates
  - Number per unit: 3
  - Width: 7.33 m
  - Height: 16.3 m

Appendix A presents a drawing of the proposed variant of the 2007 study.

6.2 ANALYSIS OF THE PROPOSED LAYOUT

Simulations have been performed on the proposed layout (including the modifications to the approach channel of the diversion/spillway) for two (2) cases:

- The power plant operating at maximum capacity, and
- The power plant and the spillway operating simultaneously.

Figure 6-1 illustrates the proposed layout with the revised approach channel.

6.2.1 Power Plant at Maximum Capacity

The operation of the power plant alone (without the operation of the spillway) at the maximum operating level (39 m) corresponds to the normal operating conditions at Muskrat Falls. For this simulation, the four (4) units are operating at maximum capacity.

Figure 6-2 presents the hydraulic conditions for this case at three (3) elevations (18.5 m, 24.5 m and 30.5 m) to illustrate the changes in the flow pattern. This figure shows that the velocity of water above the retaining wall is relatively important and causes discrepancies in the approach conditions for units No. 1 and No. 2 (near the spillway). At elevations 24.5 m and 30.5 m, an eddying zone between units 1 and 2 can be observed, which could become a vortex[4]. At the lowest level (18.5 m), the flow pattern at units No. 1 and No. 2 shows a lateral component near the intake, but at units No. 3 and No. 4 the flow pattern is almost perpendicular to the intake.

Figure 6-3 presents a cross-section upstream of the power intake. It shows that the hydraulic conditions for units No. 3 and No. 4 are good, but at units No. 1 and No. 2 the lateral component of the velocity is relatively important, which indicates a relatively bad flow feed to units No. 1 and No. 2. This problem would not be critical if there was a long penstock between the intake and the unit (the velocity pattern would correct itself), however this is not the case for Muskrat Falls and the velocity pattern at the intake could cause efficiency losses.

Considering these results, it appears that the approach channel to the intake should be modified.

6.2.2 Power Plant and Spillway

A simulation was also performed considering the simultaneous operation of the power plant and the spillway to address the potential problems. For this simulation, the total simulated discharge is the 1:40 year flood (5,300 m³/s); the power plant is operating at full capacity (2,660 m³/s) and the discharge through at the spillway was considered equal to 2,640 m³/s. The spilled flow was divided equally between the four (4) gates (same opening) and the upstream water level is 39 m.

Figure 6-4 illustrates the hydraulic conditions at three (3) different elevations (18.5 m, 24.5 m and 30.5 m). The division of the flow between the two (2) structures gives a better flow pattern than the previous case, because there is no water flowing from over the retaining wall and going to the power intake (no lateral velocity near the intake).

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[4] Eddy designates a circular movement of water covering a small or large area. Vortex is used specifically for an eddy that has a downdraft (vertical movement of water). In this case, the speed and rate of rotation of the fluid are greatest at the center and decrease progressively with distance from the center.

The flow distribution appears adequate at the three levels upstream of the four (4) units. However, on the left side of unit No. 1 (near the spillway), a zone of low velocity can be observed, which might induce the formation of a vortex.

6.3 LAYOUT OPTIMIZATION

6.3.1 Variants Study

The main objective of the layout optimization is the improvement of the hydraulic conditions at the power intake, particularly for unit No. 1. Different options were considered to improve the hydraulic conditions, most of them involving the retaining wall between the spillway and the power intake.

One of the problems observed is the lateral flow over the wall and going to the power intake. A test was performed without the retaining wall to verify the impact of this wall. The distribution pattern was worse than the base case at unit No. 1, which indicates the necessity of a retaining wall.

A possible solution to this problem consists of the construction of a higher retaining wall between the spillway and the power intake. Figure 6-5 illustrates the hydraulic conditions for a variant with a short wall at elevation 39 m. It shows that this wall reduces the lateral flow to the intake, but creates a zone of possible vortices at unit No. 1. Other variants with a longer wall didn't improve the flow conditions upstream of unit No. 1 as shown in Figure 6-6. For this case, the water velocity at the beginning of the wall exceeds 1.0 m/s, because the approach channel is short. The water cannot turn so quickly to reach the power plant and it creates lateral velocity at the intake and a large zone of almost still water.

6.3.2 Proposed Revised Layout

Figure 6-7 illustrates the proposed revised layout, which combines two (2) aspects, a long curved wall at elevation 39 m, to avoid the lateral velocity near the intake, and a longer approach channel at elevation 10 m up to the river, to improve the flow pattern to the intake. The proposed channel will be 500 m longer than the initial approach channel and will have the same width. The channel can be excavated during the construction of the spillway and should have no impact on the schedule.

Figures 6-8 to 6-10 illustrate the flow pattern near the intake at elevations 32 m, 24 m and 18 m respectively. These figures show a better flow pattern to the intake without any vortex or major lateral velocity components. A cross-section at the power intake (Figure 6-11) shows that the flow pattern appears similar in each unit, but still with some lateral components of the velocity.

This solution appears to be acceptable and should be used as a basis for further analysis. The layout optimization of the power intake facilities, including the wall and the approach channel will be finalized using the physical model considering the possible increase of the power plant capacity[5]. The objectives of the final optimization will be to confirm and possibly improve the hydraulic conditions at the intake and to minimize the impact on the construction costs and the schedule.

6.4 TAILRACE CHANNEL

Hydraulic conditions in the tailrace channel of the power plant have been simulated for the power plant at full capacity with and without the use of the spillway.

Figure 6-12 presents a view of the conditions of the tailrace channel at elevation 1 m when the plant operates at full capacity (without the spillway). The maximum velocity is about 1.6 m/s and the flow conditions in the tailrace channel are good.

Figures 6-13 and 6-14 show plan views at elevation 1 m and 5 m when the plant operates at full capacity and the discharge from the spillway is 2,600 m^3/s. The flow conditions in the tailrace channel are not disturbed by the discharge from the spillway.

Based on these results, the tailrace channel, as initially proposed, is acceptable.

[5] During the 2007 studies, the capacity of the Gull Island power plant was increased from 2000 MW to 2250 MW. This may have an impact in the future on the installed capacity at Muskrat Falls.

Figure 6-1: Muskrat Falls - Power Intake - Proposed Layout with Revised Approach Channel

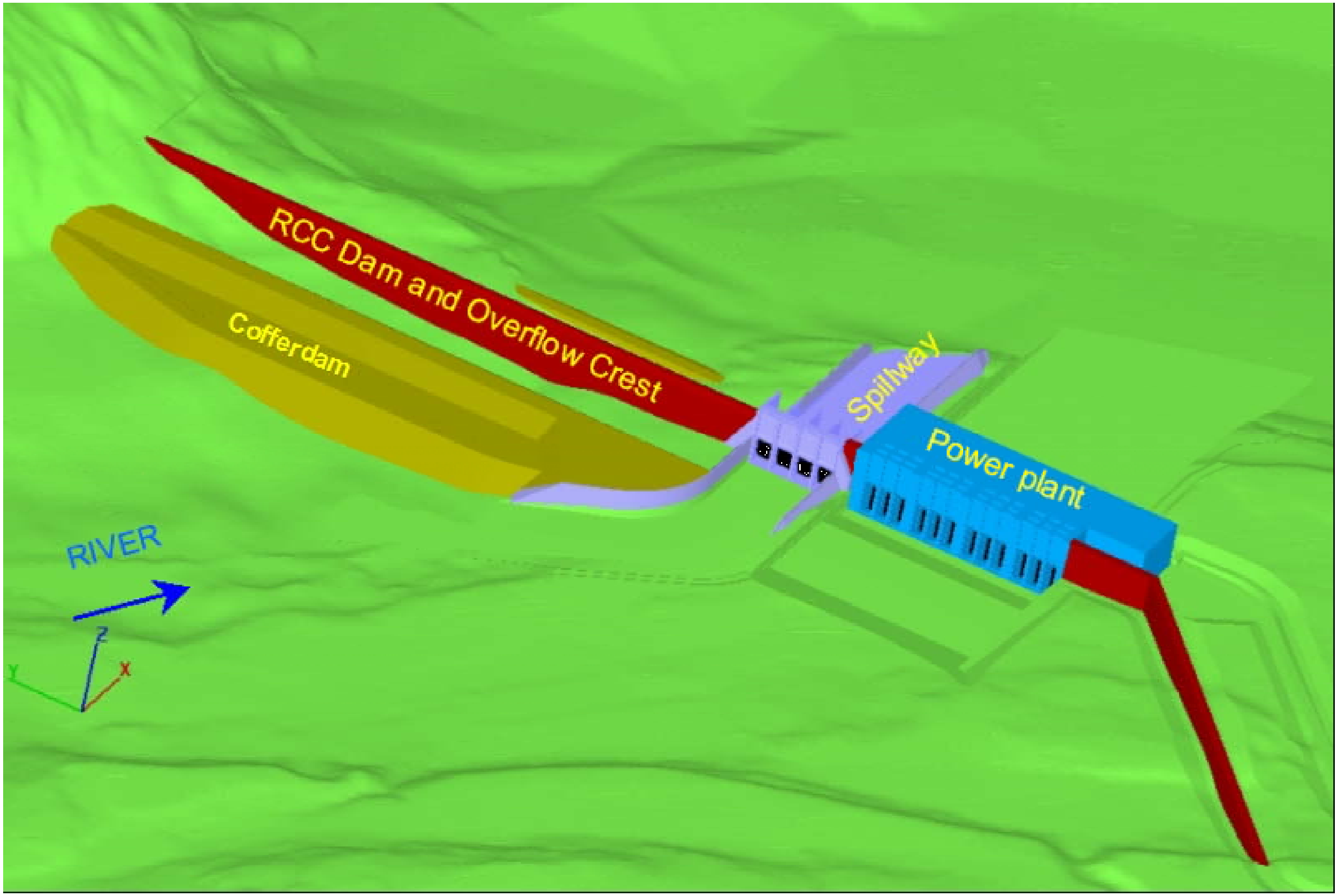


Figure 6-2: Muskrat Falls - Power Intake - Proposed Layout Plan View at Different Elevations – Maximum Turbine Discharge (2,660 m^3/s)

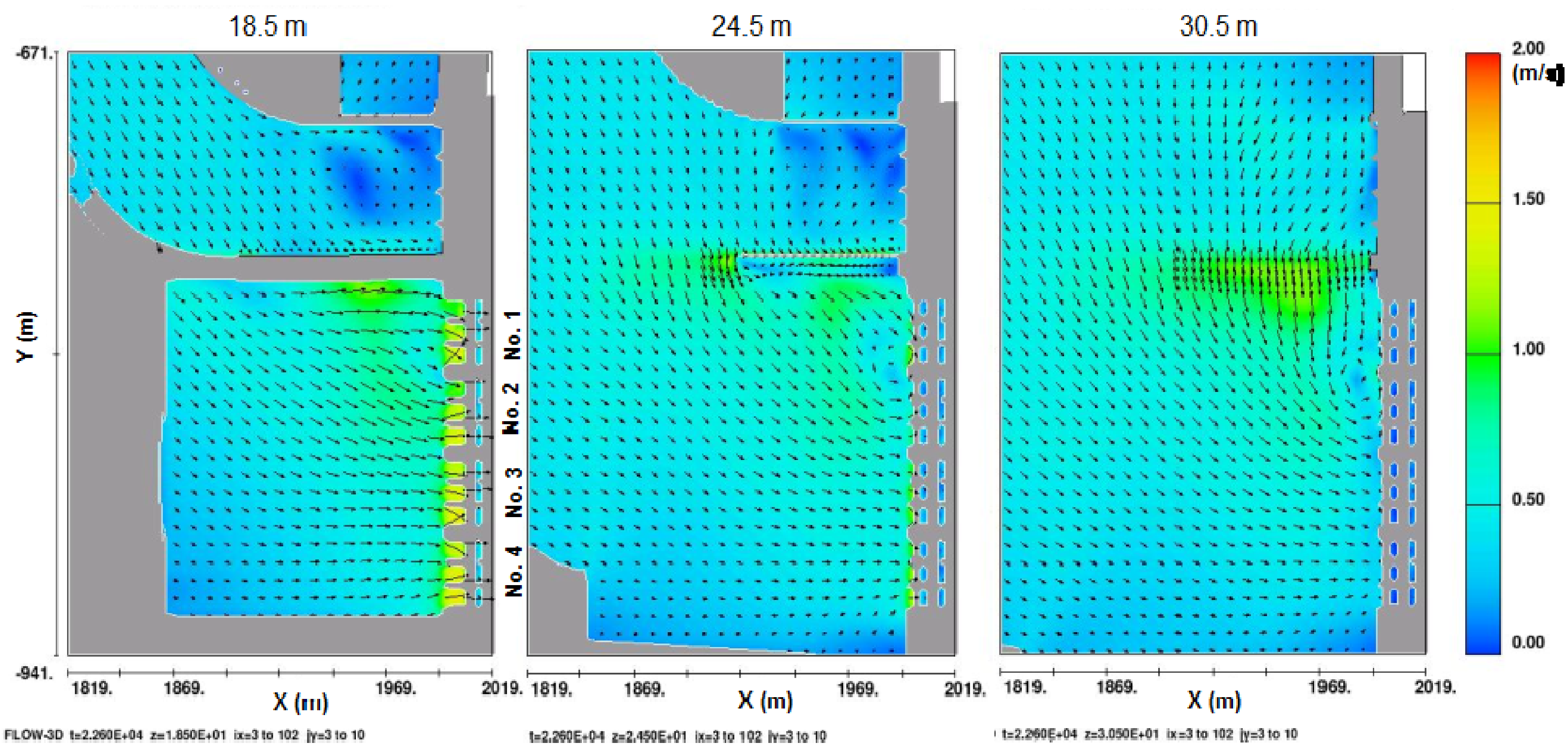


Figure 6-3: Muskrat Falls - Power Intake - Proposed Layout Cross Section Upstream of the Power Intake (looking upstream) - Maximum Turbine Discharge (2,660 m^3/s)

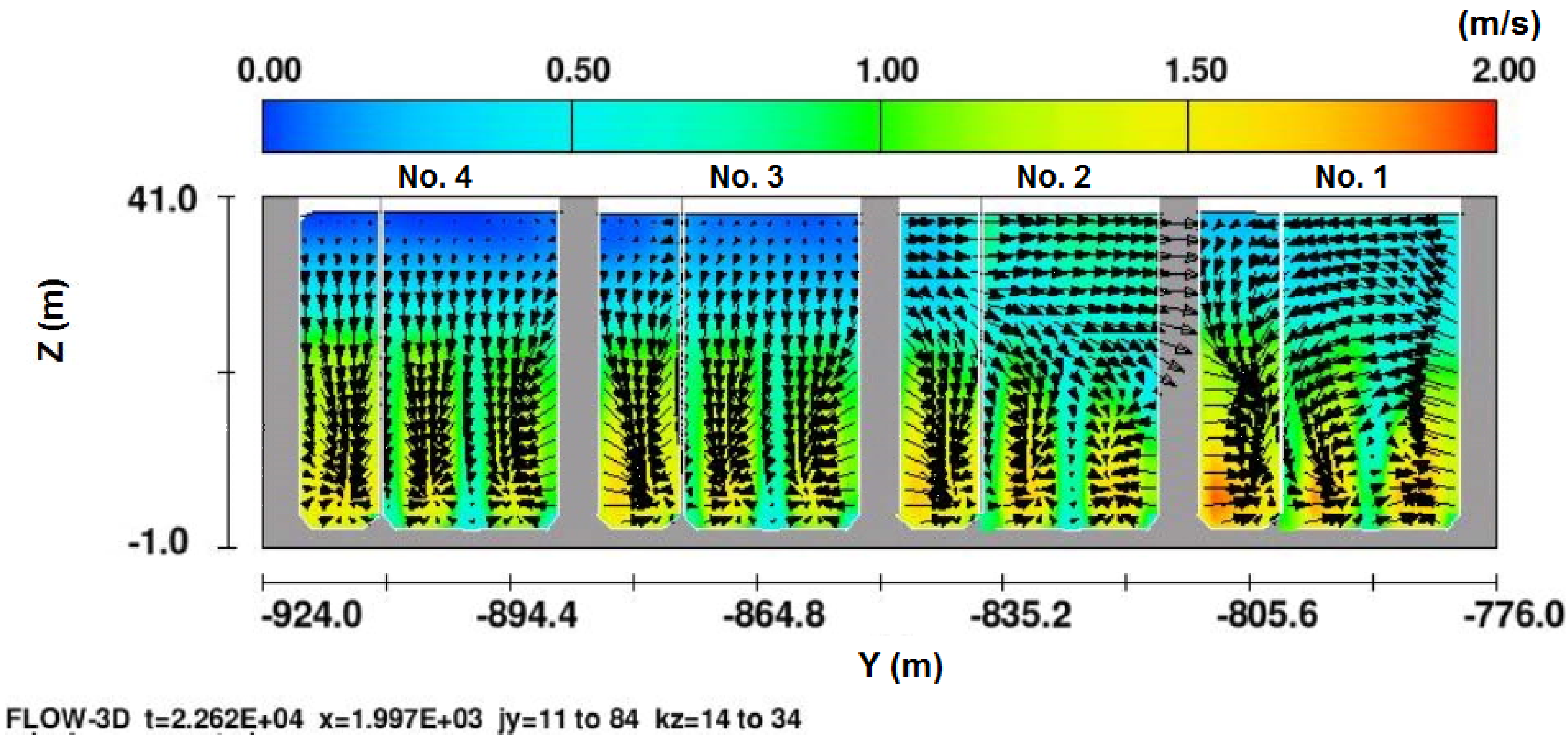


Figure 6-4: Muskrat Falls - Power Intake + Spillway in Operation Plan View at Different Elevations - Total Discharge (5,300 m^3/s)

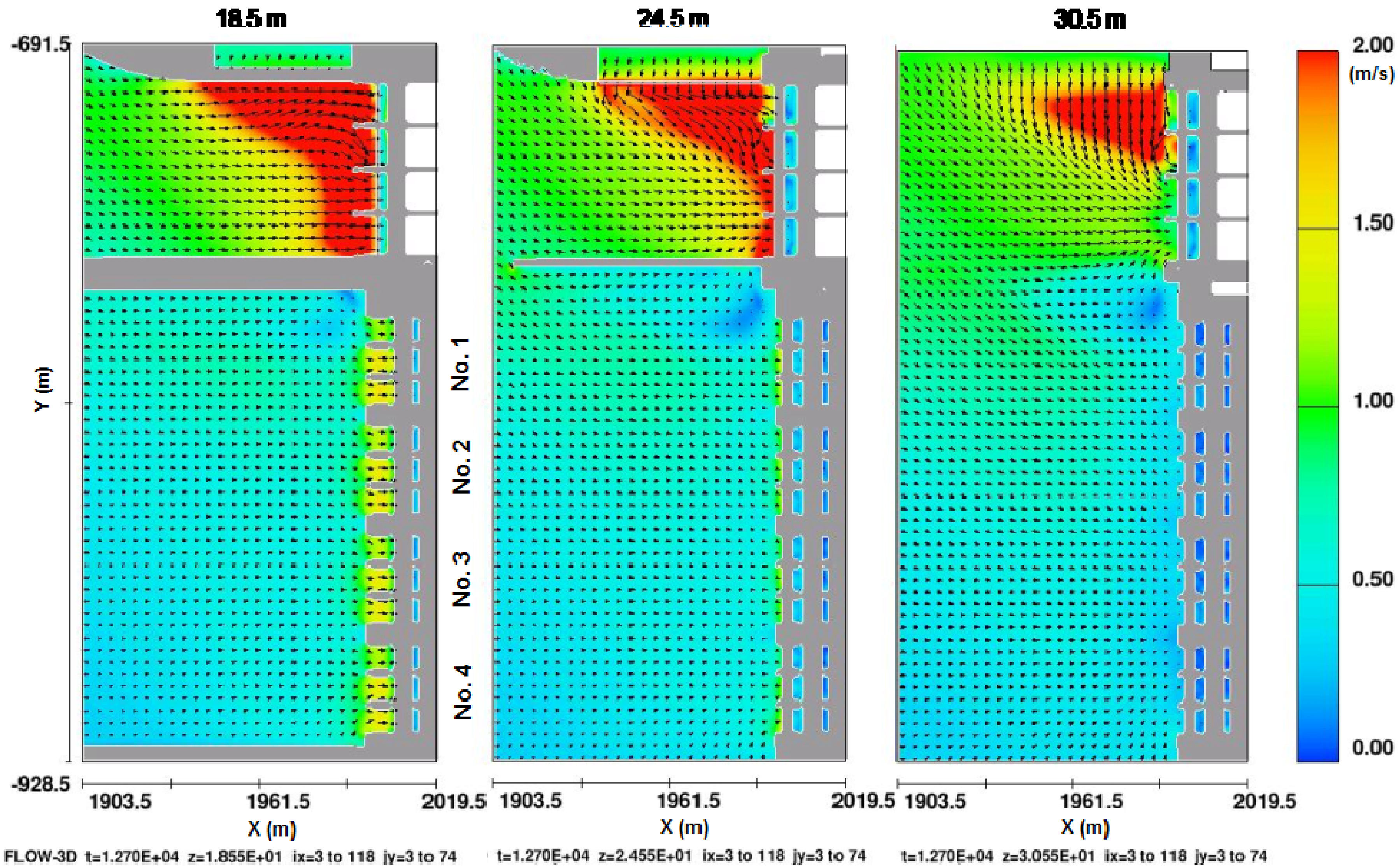


Figure 6-5: Muskrat Falls - Power Intake - Short Wall up to Elevation 39 m Plan View at Elevation 36 m - Maximum Turbine Discharge (2,660 m^3/s)

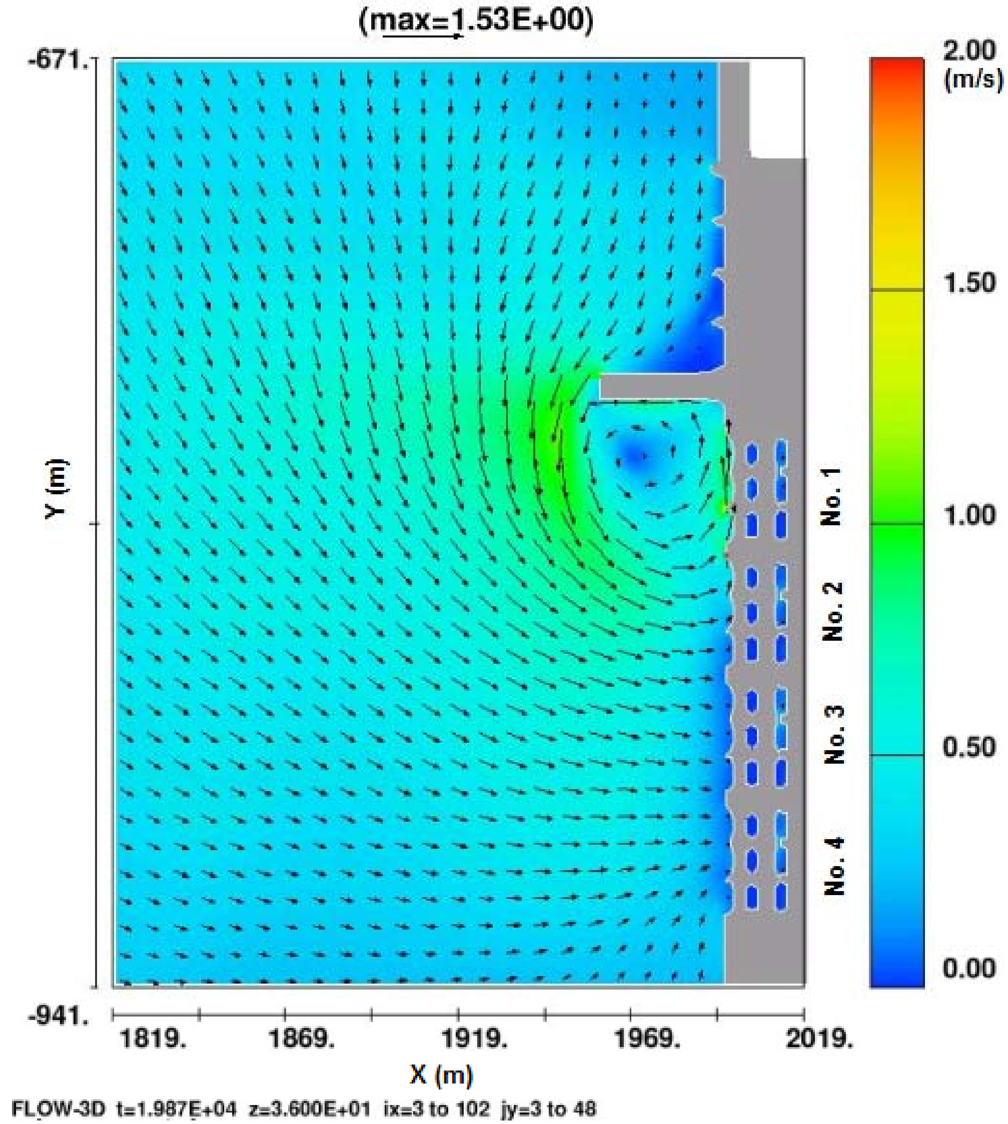


Figure 6-6: Muskrat Falls - Power Intake - Long Curved Wall up to Elevation 39 m - Plan View at Elevation 36 m - Maximum Turbine Discharge (2,660 m^3/s)

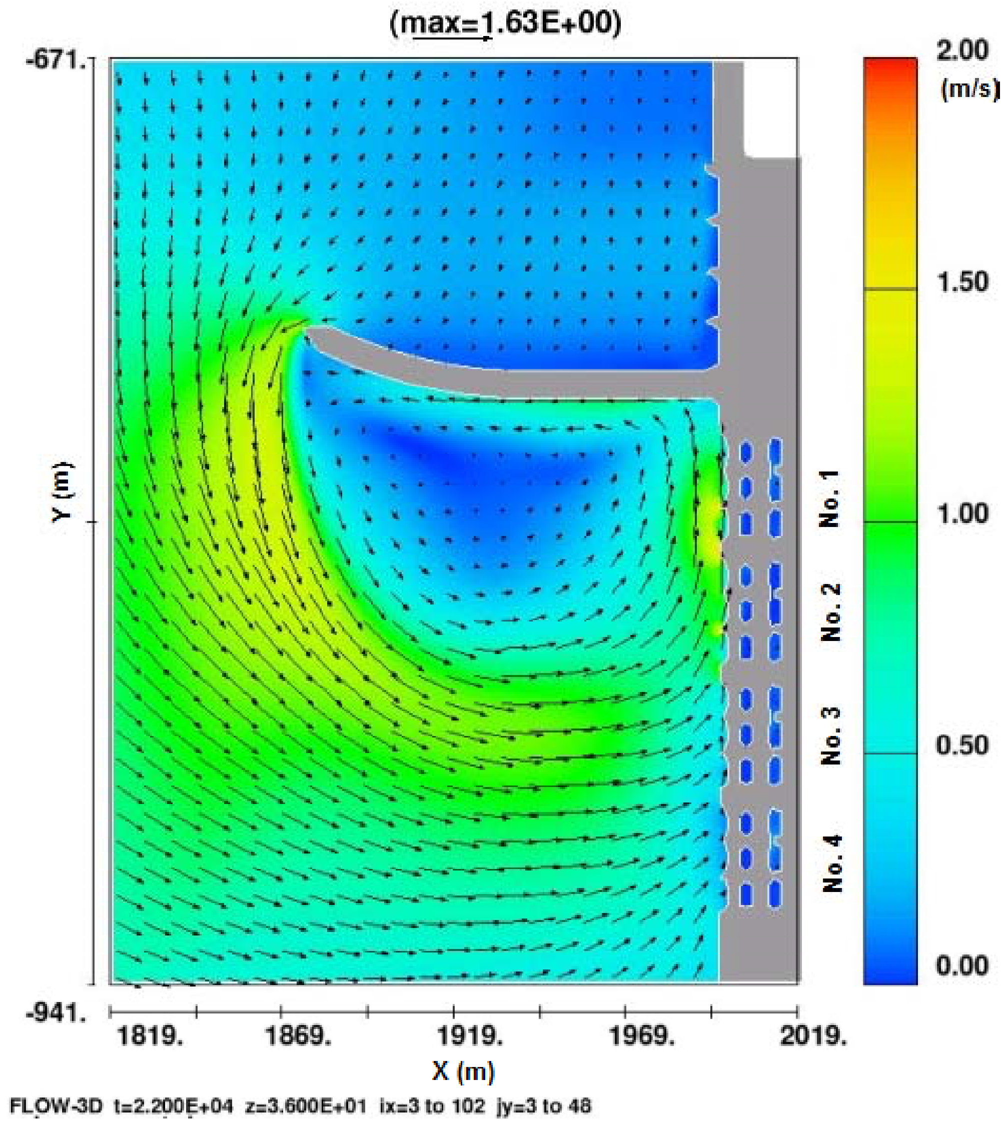


Figure 6-7: Muskrat Falls - Power Intake - Revised Layout with Curved Wall and Approach Channel at Elevation 10 m

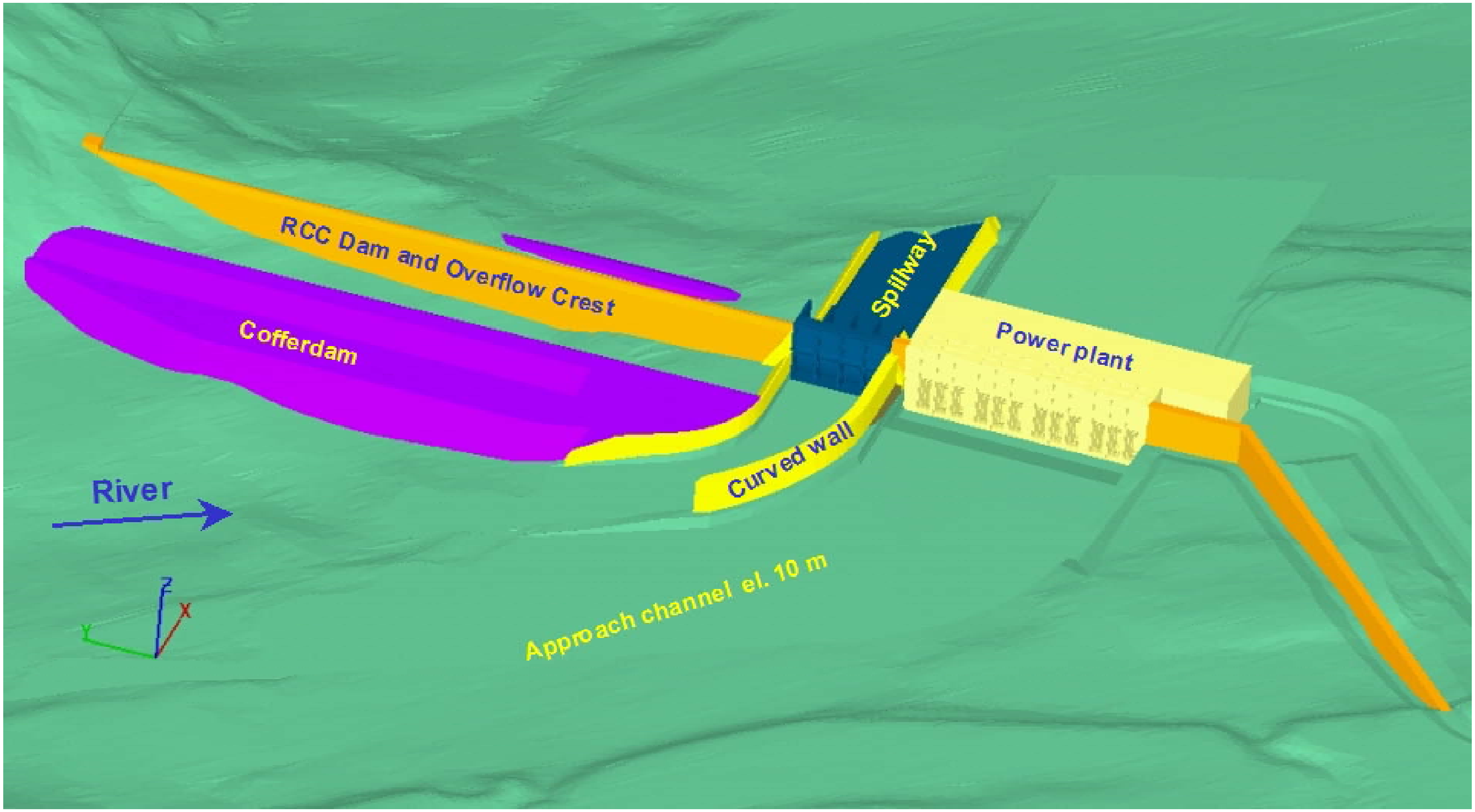


Figure 6-8: Muskrat Falls - Power Intake - Curved Wall up to Elevation 39 m and Approach Channel at Elevation 10 m Plan View at Elevation 32 m - Maximum Turbine Discharge (2,660 m^3/s)

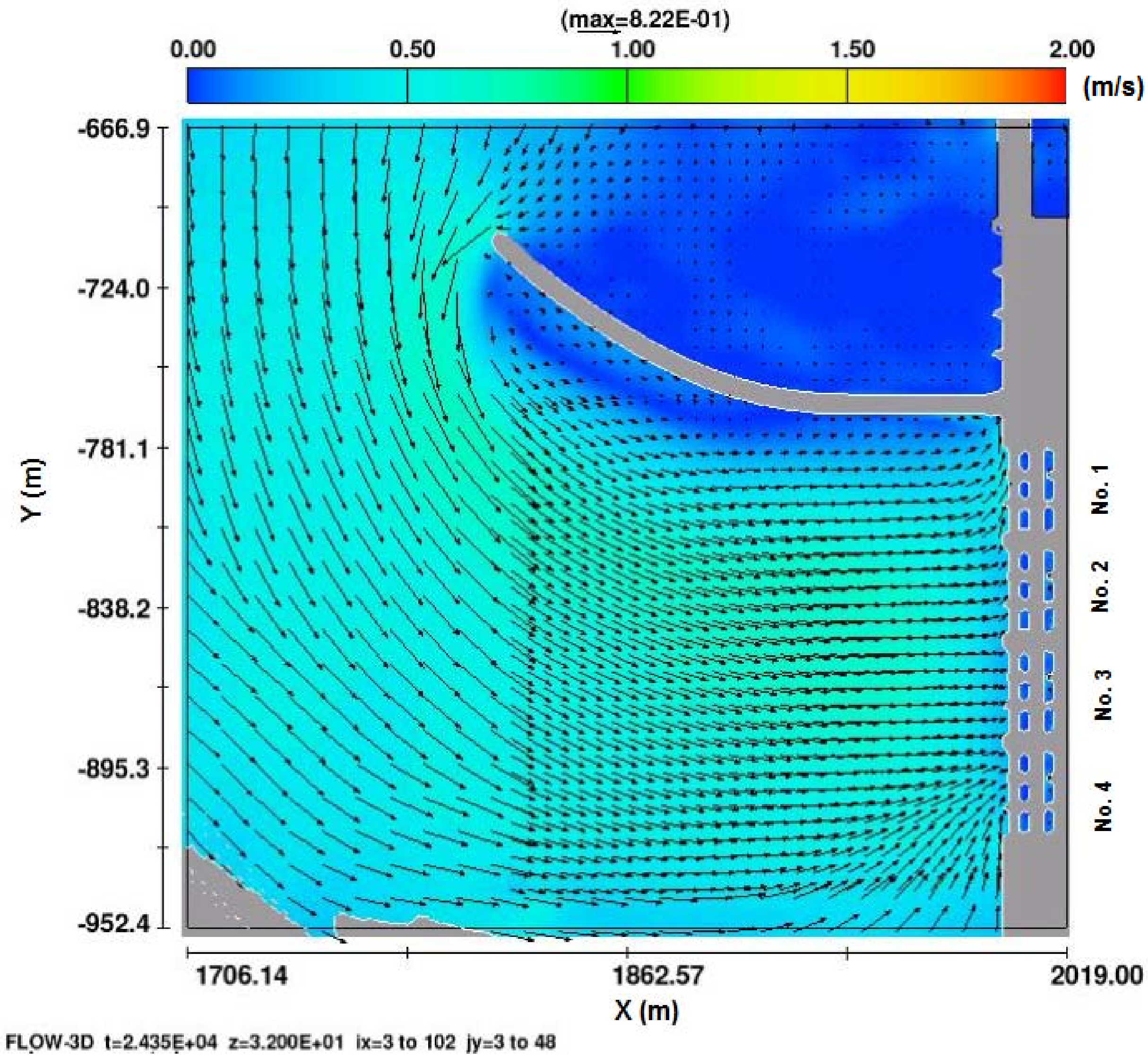


Figure 6-9: Muskrat Falls - Power Intake - Curved Wall up to Elevation 39 m and Approach Channel at Elevation 10 m Plan View at Elevation 24 m - Maximum Turbine Discharge (2,660 m^3/s)

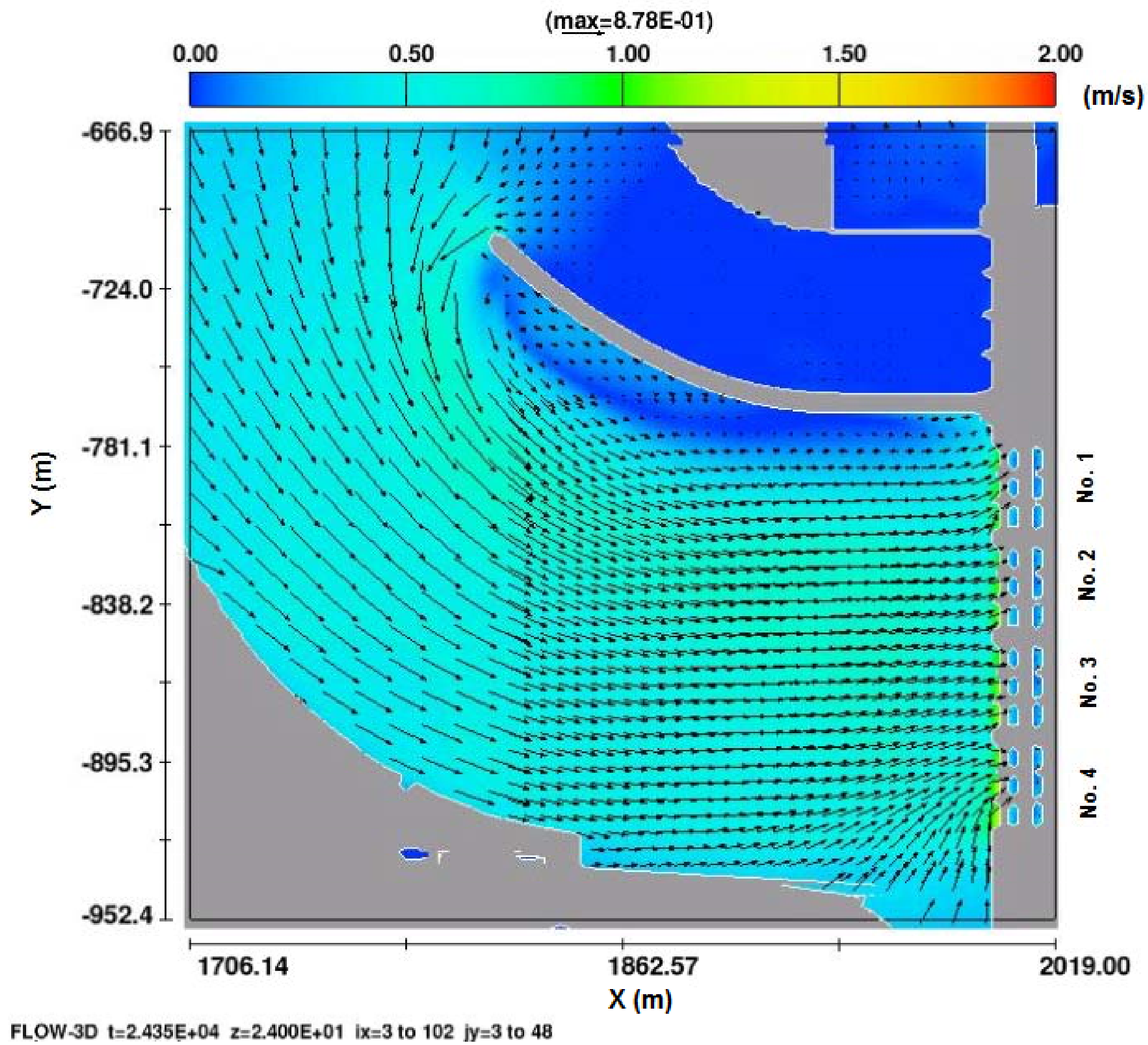


Figure 6-10: Muskrat Falls - Power Intake - Curved Wall up to Elevation 39 m and Approach Channel at Elevation 10 m Plan View at Elevation 18 m - Maximum Turbine Discharge (2,660 m^3/s)

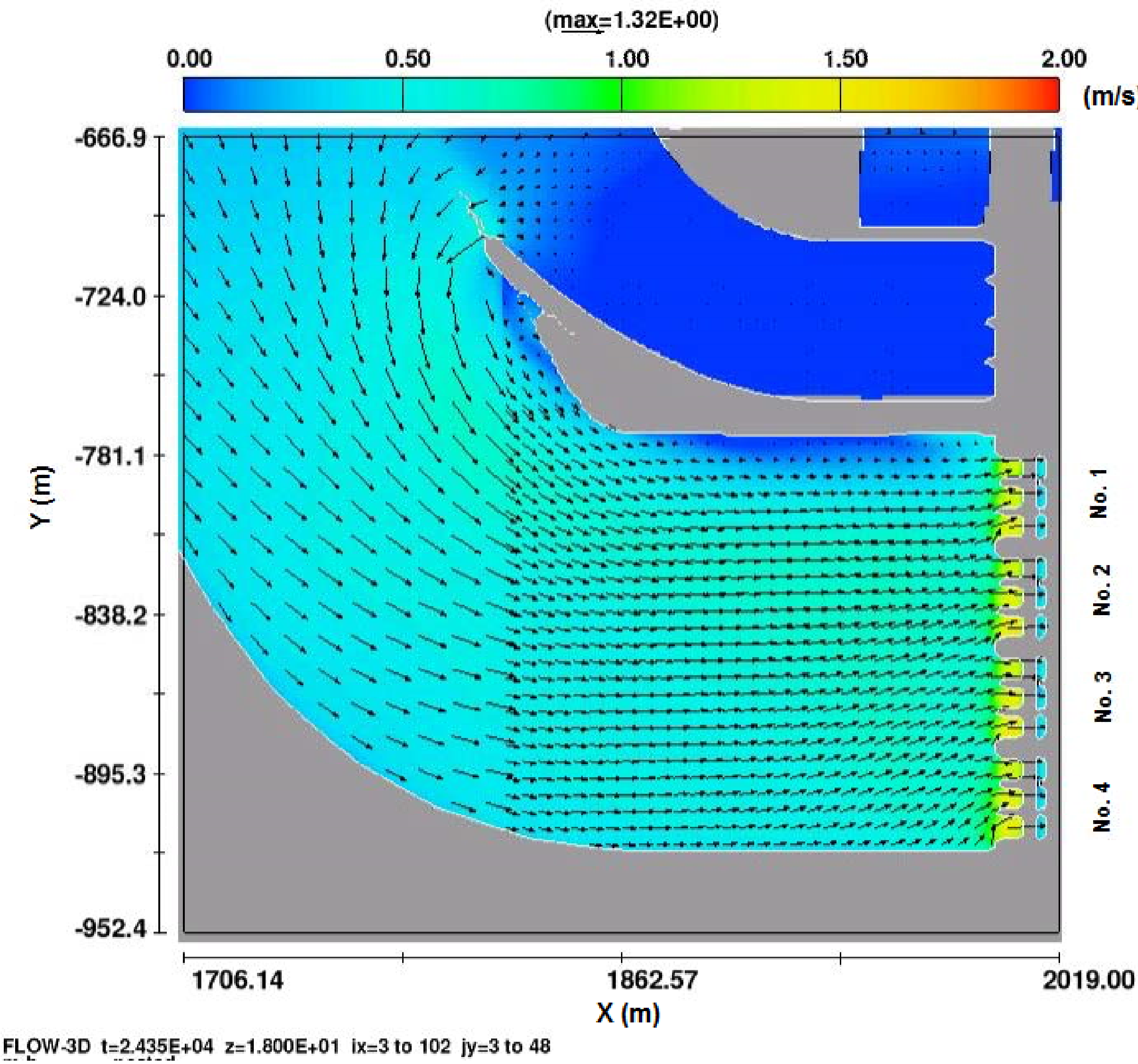


Figure 6-11: Muskrat Falls - Power Intake - Curved Wall up to Elevation 39 m and Approach Channel at Elevation 10 m Cross Section Upstream of the Power Intake (looking upstream) - Maximum Turbine Discharge (2,660 m^3/s)

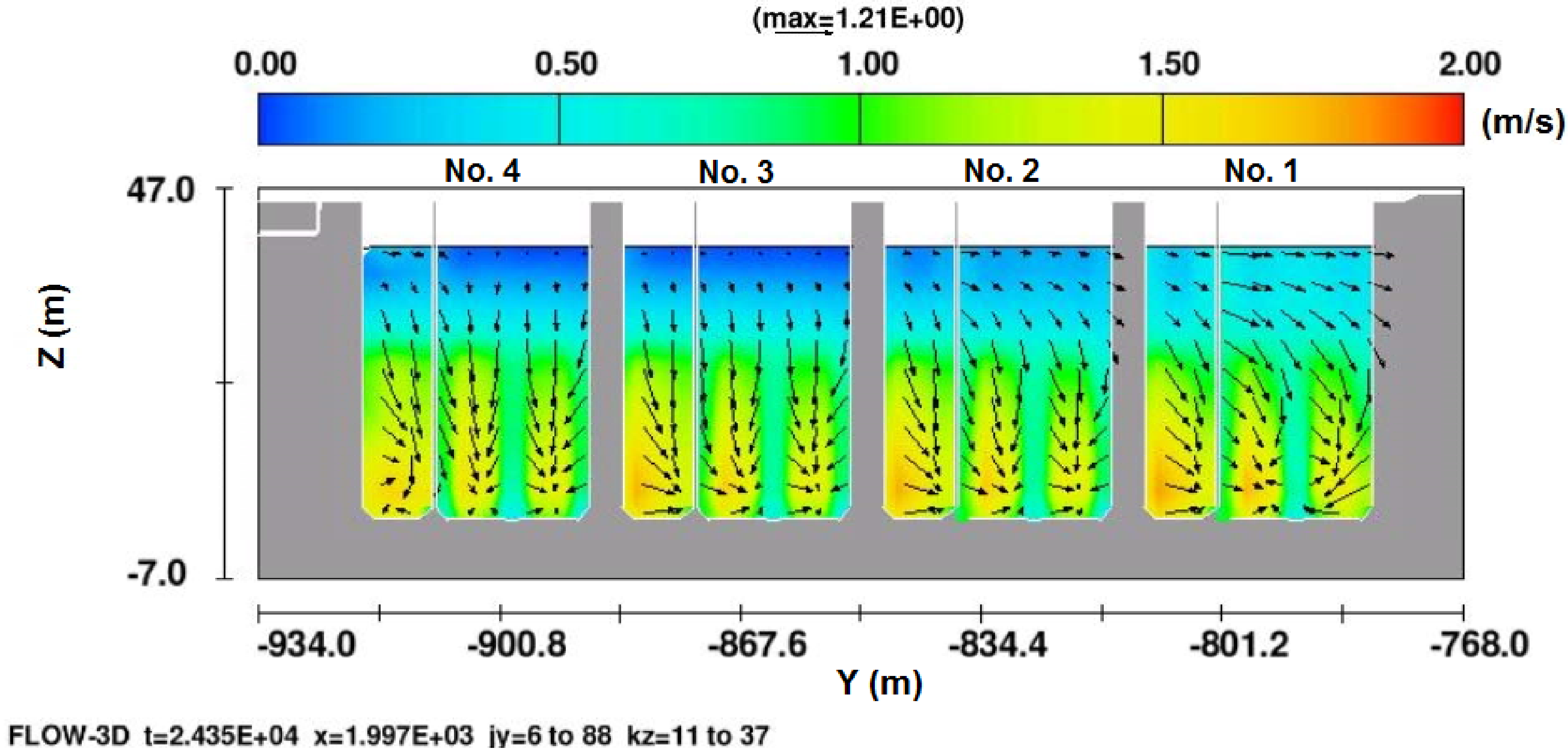


Figure 6-12: Muskrat Falls - Tailrace Channel - Proposed Layout Plan View at Elevation 1 m - Maximum Turbine Discharge (2,660 m^3/s)

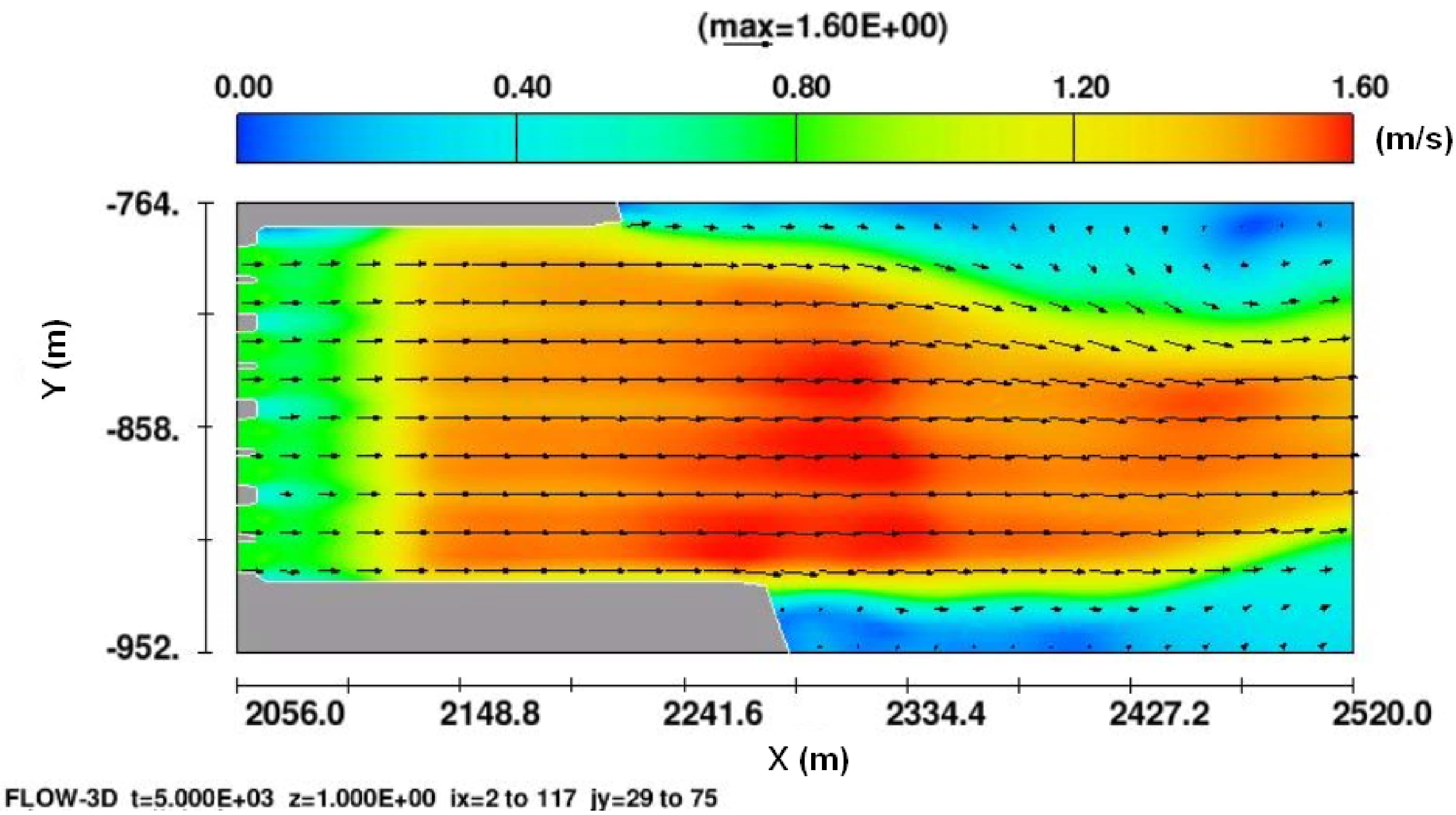


Figure 6-13: Muskrat Falls - Tailrace Channel - Proposed Layout Plan View at Elevation 1 m - Total Discharge of (5,300 m^3/s)

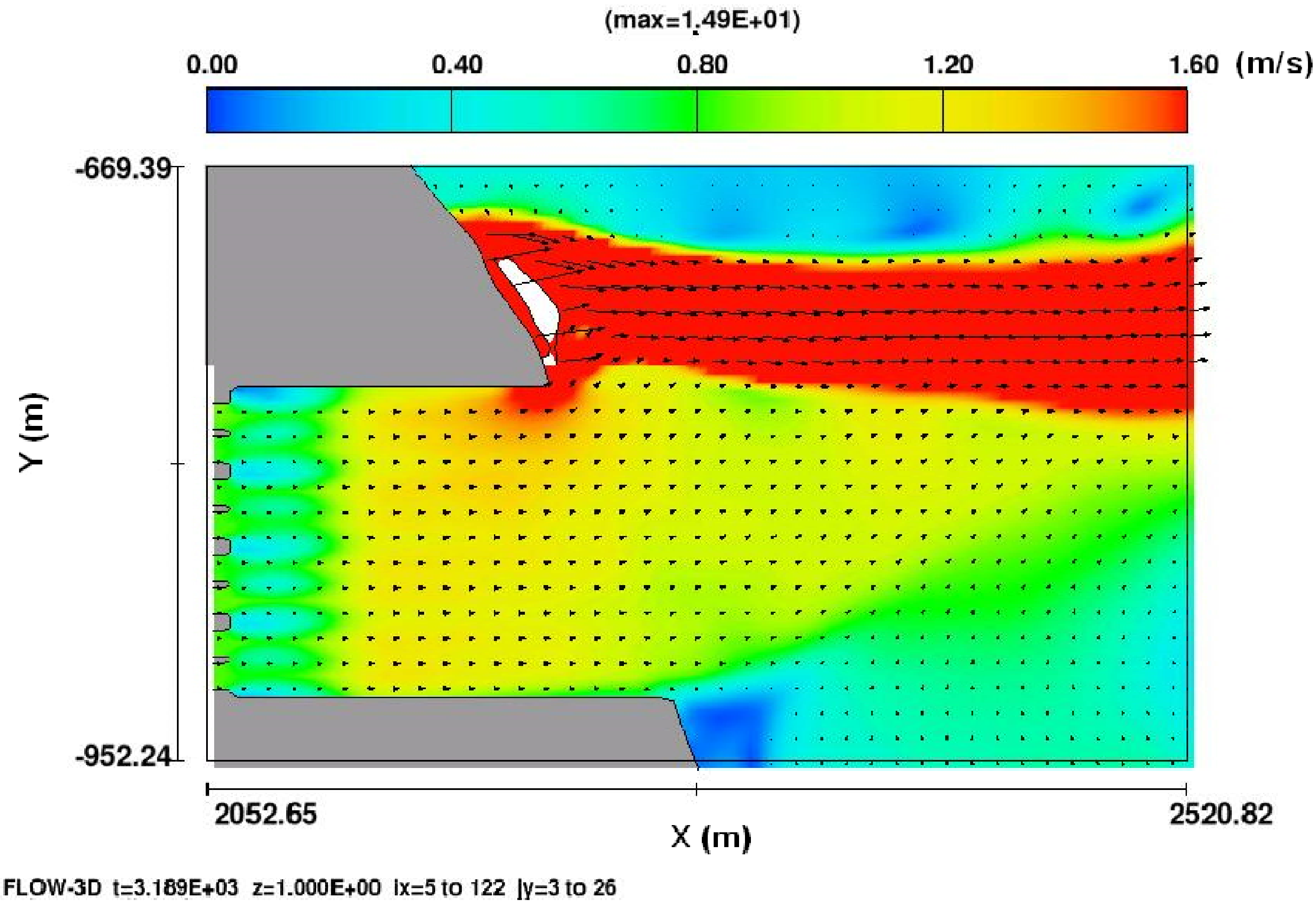
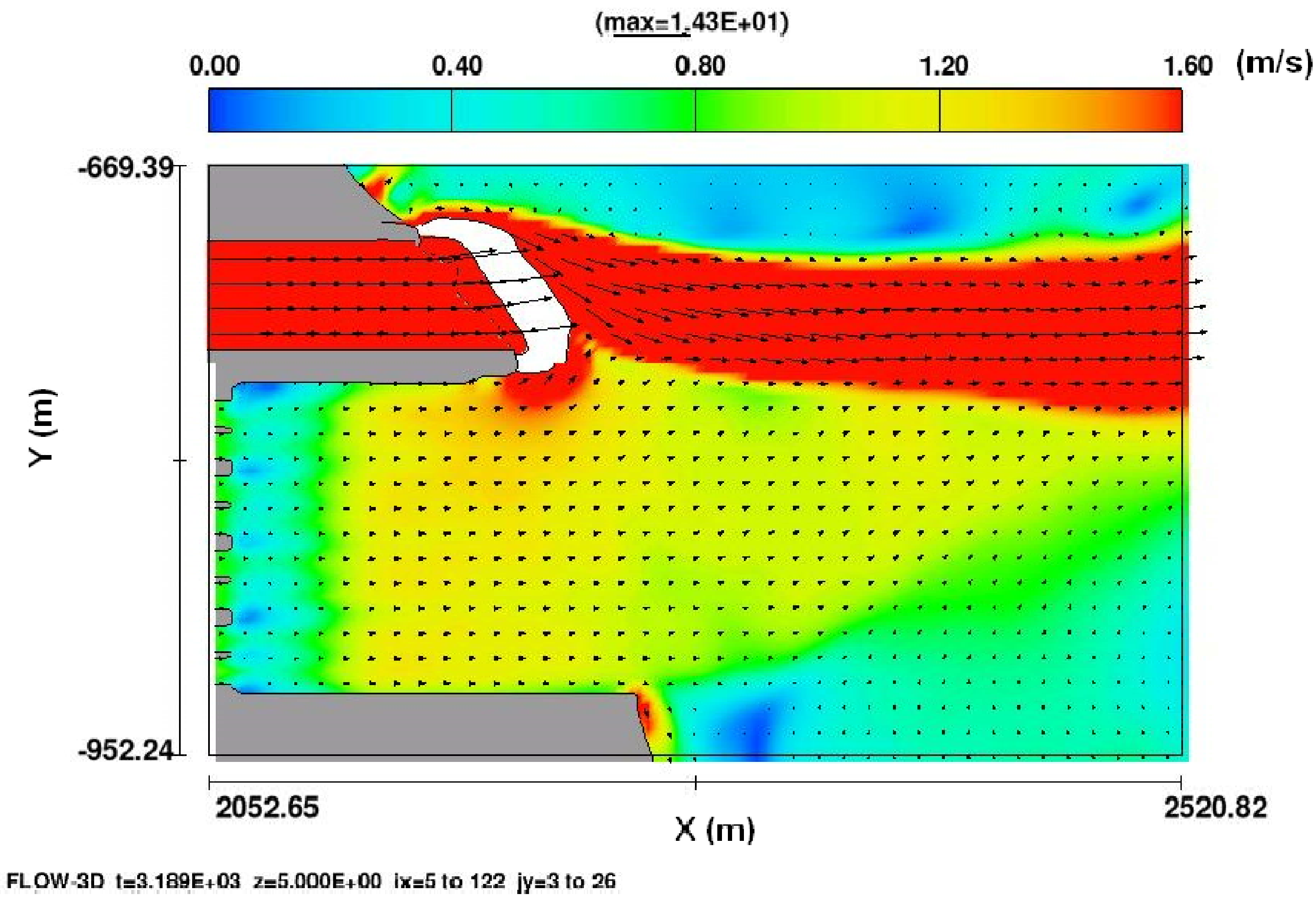


Figure 6-14: Muskrat Falls - Tailrace Channel - Proposed Layout Plan View at Elevation 5 m - Total Discharge of (5,300 m^3/s)



7 SPILLWAY FACILITIES

7.1 SPILLWAY FACILITIES CHARACTERISTICS

The spillway facilities were reviewed in the 2007 studies (MF1050). The proposed layout consists of a four (4) bay gated spillway, with submerged radial gates 12.5 m wide and 14.8 m high and a fixed crest overflow section on the north dam with the weir at elevation 39.5 m.

The main characteristics of the spillway facilities are (Reference 2):

- Design flow (PMF): 22 100 m³/s [6]
- Reservoir level for design flow: 44.0 m
- Reservoir normal operation level: 39.0 m
- Spillway rating curve: see Figure 7.1
- Sluices
  - Number of sluices: 4
  - Type of gates: Submerged radial
  - Width: 12.5 m
  - Height: 14.8 m
  - Sill elevation: 5.0 m
- North Dam – Fixed crest section
  - Crest elevation: 39.5 m
  - Length: 430 m

[6] The value of the PMF is based on the 1999 study. A larger value for the PMF was determined in the 2007 studies and the larger value will be used in any future modeling.

Appendix A presents a drawing of the original layout. It must be noticed that the modifications proposed for the diversion facilities (modifications of the approach channel and the left wall) were included in this analysis, but the modifications proposed for the power intake (rock excavation upstream of the intake approach channel and curved wall between the sluices and the power intake) were not included, because the two (2) analyses were performed in parallel. However, the proposed modifications for the power intake should not have significant impact on the hydraulic conditions at the spillway facilities, because the power plant is located on the right side of the river and the spillway facilities are located in the middle of the river where the flow is going naturally.

7.2 ANALYSIS OF THE PROPOSED LAYOUT

The analysis of the proposed layout was performed for two (2) cases:

- Evaluation of the hydraulic conditions for the PMF;
- Evaluation of the hydraulic conditions at maximum normal operation level.

7.2.1 PMF Conditions

Under PMF conditions, the spillway should be able to release 22,100 m^3/s[7] at elevation 44.0 m (about 13,300 m^3/s from the four sluices and 8,800 m^3/s from the overflow crest).

Figures 7-2 and 7-3 illustrate the general hydraulic conditions for the system. It should be noted that the water velocities at the toe of the RCC dam and downstream of the four (4) sluices are 25 m/s and 22 m/s respectively, which exceeds the criteria for the maximum velocity on rock (20 m/s). These results justify the necessity of a concrete slab as already planned. Figure 7-3 shows a depression upstream of sluice No. 1. This depression appears to be related to a vortex, but considering the discharge over the crest of the RCC dam, the depression seems to have no major impact on the spillway capacity.

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[7] The value of the PMF is based on the 1999 study. A larger value for the PMF was determined in the 2007 studies and the larger value will be used in any future modeling.

For this elevation the total discharge from the model was 21,960 m/s, which is 0.6% lower than the expected discharge. The difference can be considered as negligible at this phase of the study.

Hydraulics conditions at the sluiceway are shown on Figures 7-4 and 7-5. Figure 7-4 presents a cross-section in the sluiceway and Figure 7-5 presents a cross-section of the four (4) sluices. This last figure shows that the hydraulic conditions in sluice No. 1 are slightly disturbed by the presence of the wall and the overflow crest. This is similar to the conditions observed for the diversion and could slightly reduce the discharge capacity. The total discharge from the four (4) sluices was estimated to 12,830 m^3/s, which is lower than the expected value.

Figures 7-6 and 7-7 present a cross-section at two (2) different locations in the RCC dam. As mentioned earlier, the water velocity exceeds 20 m/s at the toe of the dam and the hydraulic jump will depend of the rock profile downstream. The discharge computed by FLOW-3D for the overflow crest is 9,130 m^3/s, which is slightly above the theoretical value.

The simulation results for the PMF conditions show no major problem. However, the discharge capacity of the sluices should be confirmed.

7.2.2 Maximum Normal Operation Level

At the maximum normal operation level of 39 m, only the four (4) sluices will be available to spill water downstream (the crest of the RCC dam is at elevation 39.5 m). Based on theoretical equations, the discharge capacity of the four (4) sluices should be about 12,400 m^3/s, however the simulation results show that the discharge from the sluices will be about 11,550 m^3/s. This difference can be caused by an over estimation of the discharge coefficient of the sluices and/or by the flow pattern upstream of the sluices.

Figure 7-8 presents a 3D view of the hydraulic conditions at the spillway considering the four (4) sluices totally opened. It shows a vortex upstream of sluice No. 1, which is relatively important. This sub-optimal flow pattern could reduce the capacity of the sluice as observed for the diversion conditions.

Figure 7-9 presents a plan view at elevation 34 m and shows the depression of the water level upstream of sluice No. 1 (the white section indicates that the water level is lower than 34 m). This figure shows the flow pattern coming laterally along the RCC dam, which causes the vortex formation. The velocity at the vortex is about 8.8 m/s compared to 2 to 3 m/s at the same elevation for the middle sluices. This increase in velocity will also reduce the energy available and increase the losses. A smaller vortex can be noted near sluice No. 4, but the presence of the retaining wall between the sluices and the power intake (see Section 6) should eliminate this problem.

Figure 7-10 presents a cross-section along the four (4) sluices. The flow patterns for sluices No. 2 and 3 are good and the one for sluice No. 4 will be better with a higher retaining wall as proposed in Section 6. However, the influence of the vortex can clearly be seen for sluice No. 1 and will have an impact on the discharge capacity.

The cross-sections along the sluiceway for sluice No. 2 also illustrate the changes in the flow pattern between sluice No. 2 (Figure 7-11) and sluice No. 1 (Figure 7-12).

The simulation results for the normal level show major concerns compared to the PMF results, particularly concerning the discharge capacity of the sluices. Two (2) factors should be considered, the upstream flow pattern with the vortex and the geometry of the structure. Theoretical equations show that the discharge capacity at full opening depends on the angle of the wall at the entrance of the sluice (Reference 4), which should be about 30º based on these equations to obtain the discharge coefficient expected.

7.3 LAYOUT OPTIMIZATION

To minimize the problems observed upstream of sluice No. 1, a simulation was performed considering a wall at elevation 39 m between the sluices and the RCC dam. Figure 7-13 illustrates the flow pattern for this case. An eddy zone appears upstream of sluice No. 1 and a vortex can be noted at sluice No. 4. As mentioned earlier, the wall between the power intake and the sluices is not represented in this simulation, which will change the hydraulic conditions particularly for sluice No. 4.

The approach to optimize the scheme should be to determine the optimum configuration for the power intake (including the wall) and, based on these modifications, determine the best configuration for the approach channel to the sluices.

Based on the possible increase of the PMF at Muskrat Falls, a fifth sluice could also be considered. It will increase the spill capacity of the system and will give more flexibility during the construction (increase of the design flood for normal operation at Churchill Falls).

The layout optimization of the spillway facilities will be finalized using the physical model.

Figure 7-1: Muskrat Falls - Spillway Facilities - Theoretical Discharge Rating Curve

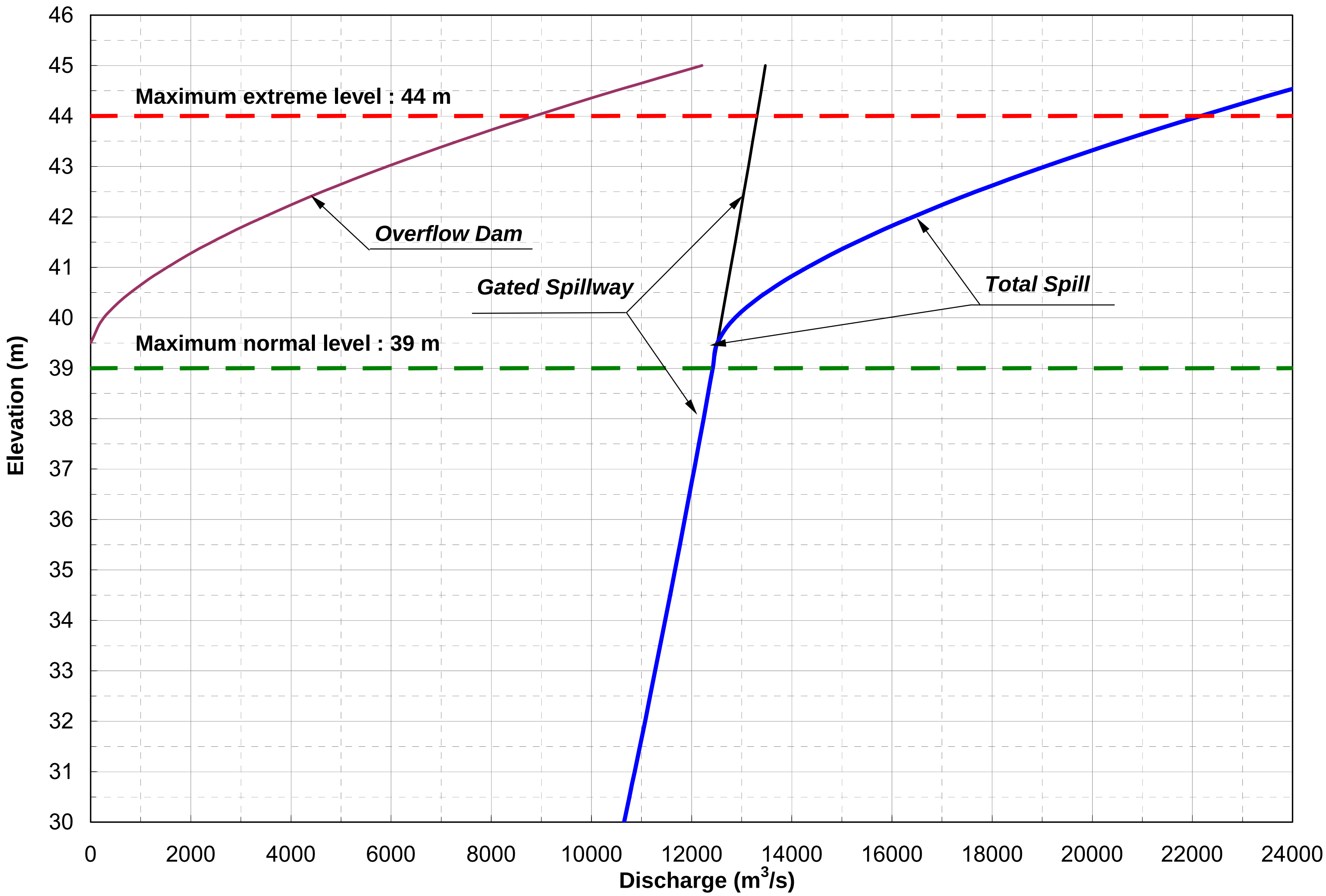


Figure 7-2: Muskrat Falls - Spillway Facilities - Proposed Layout Design Flow (22,100 m3/s) - 3D View from Downstream

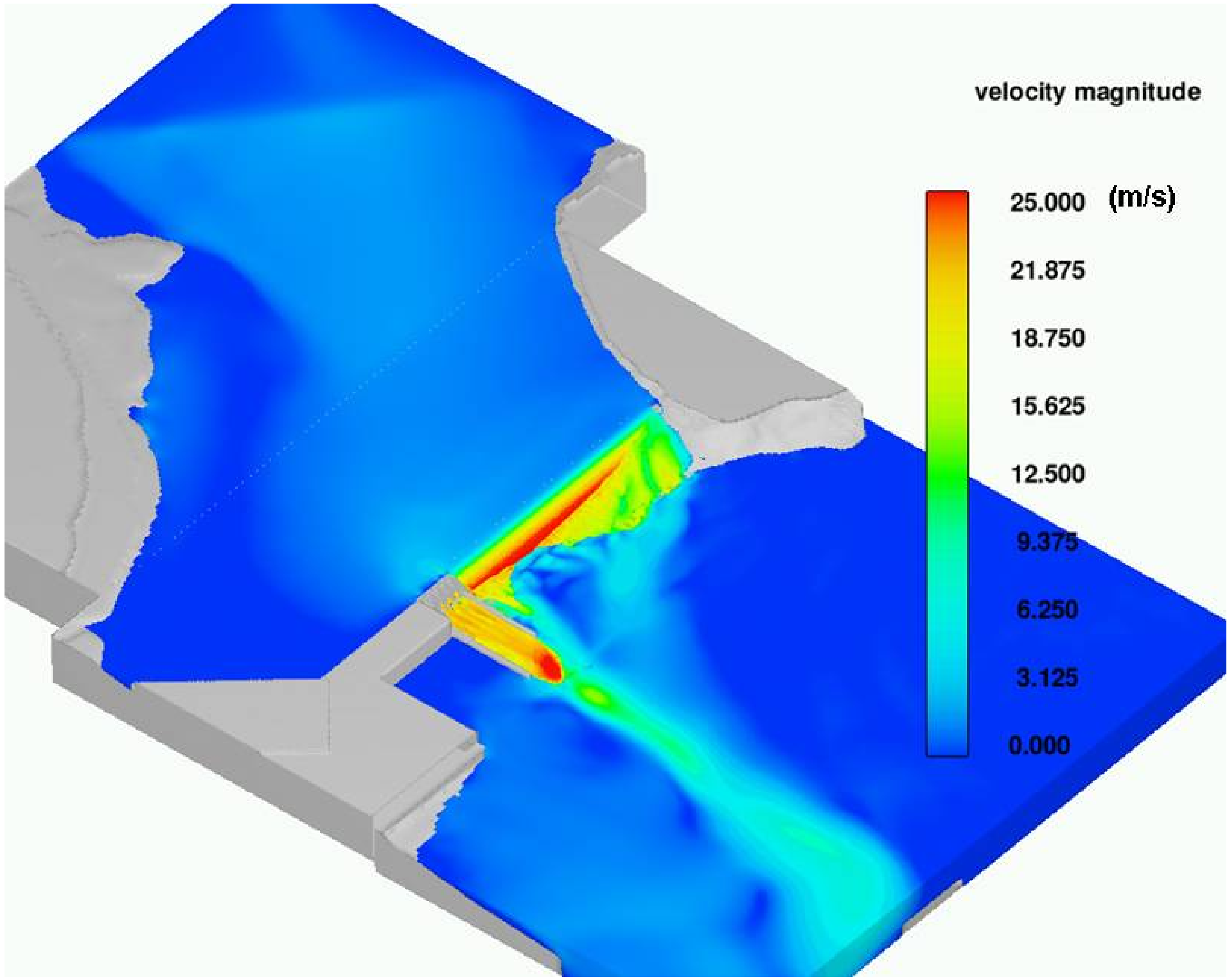


Figure 7-3: Muskrat Falls - Spillway Facilities - Proposed Layout Design Flow (22,100 m^3/s) - 3D View of the Spillway

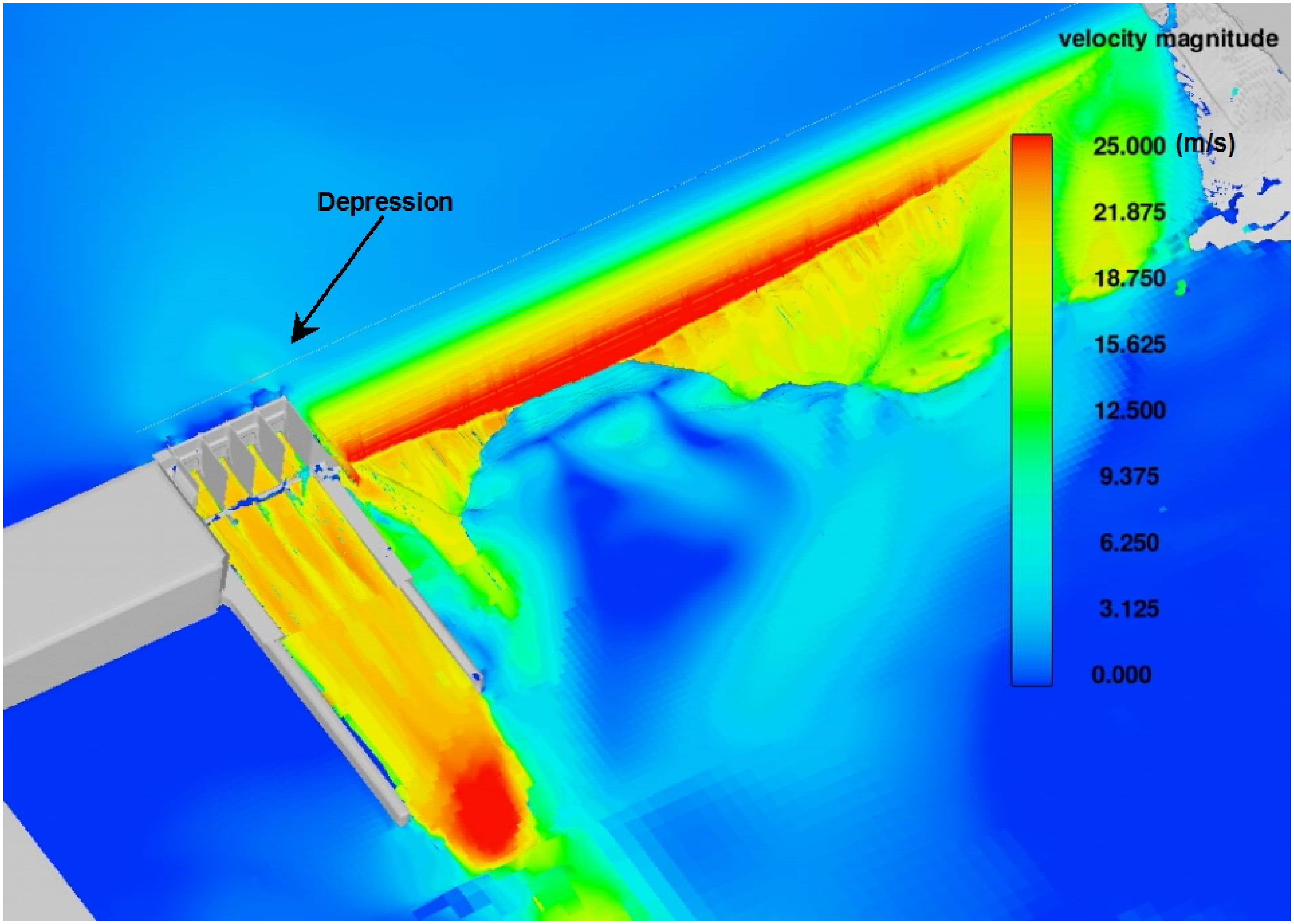


Figure 7-4: Muskrat Falls - Spillway Facilities - Proposed Layout Design Flow (22,100 m^3/s) - Cross Section View of the Sluiceway

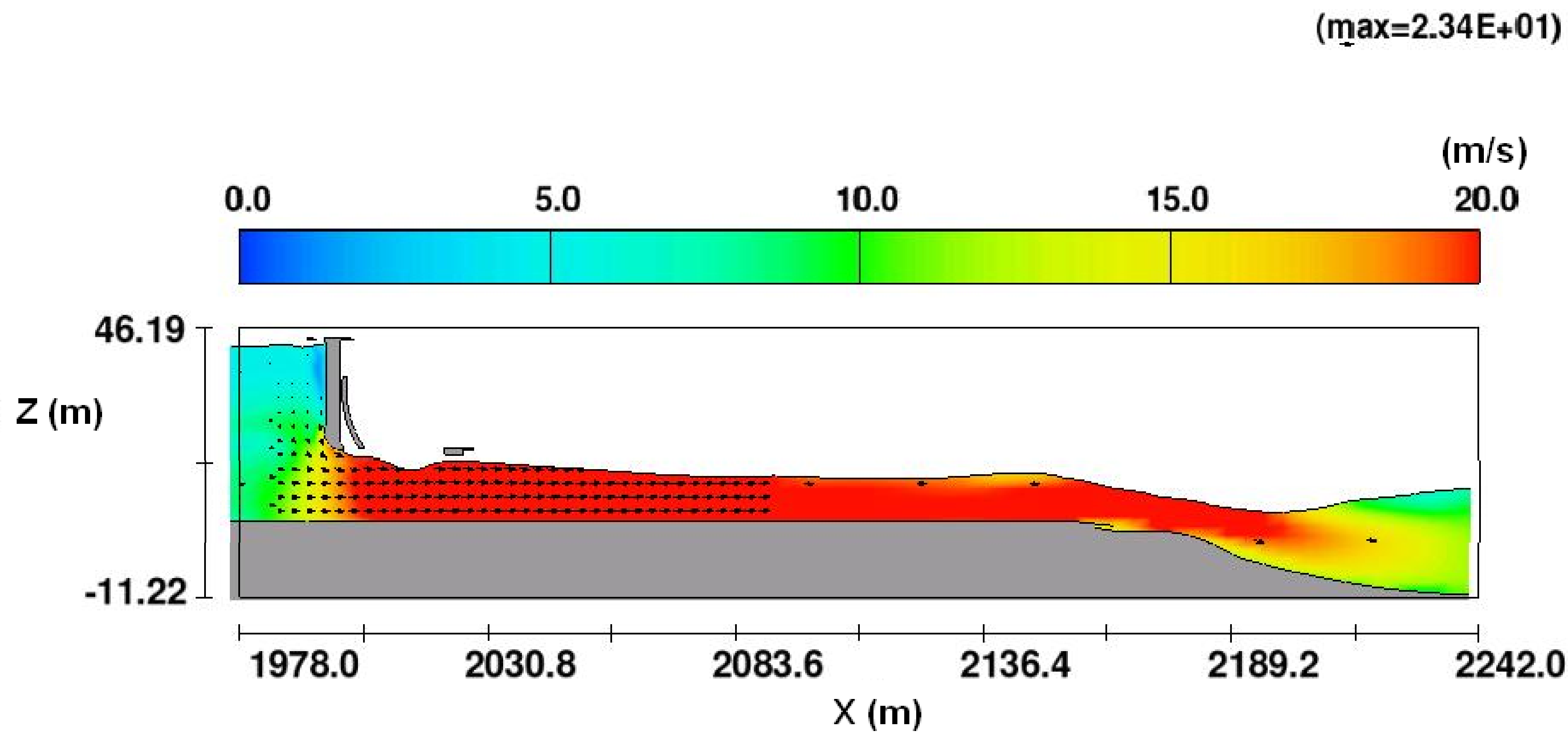


Figure 7-5: Muskrat Falls - Spillway Facilities - Proposed Layout Design Flow (22,100 m^3/s) - Cross Section View of the Four Sluices (looking upstream)

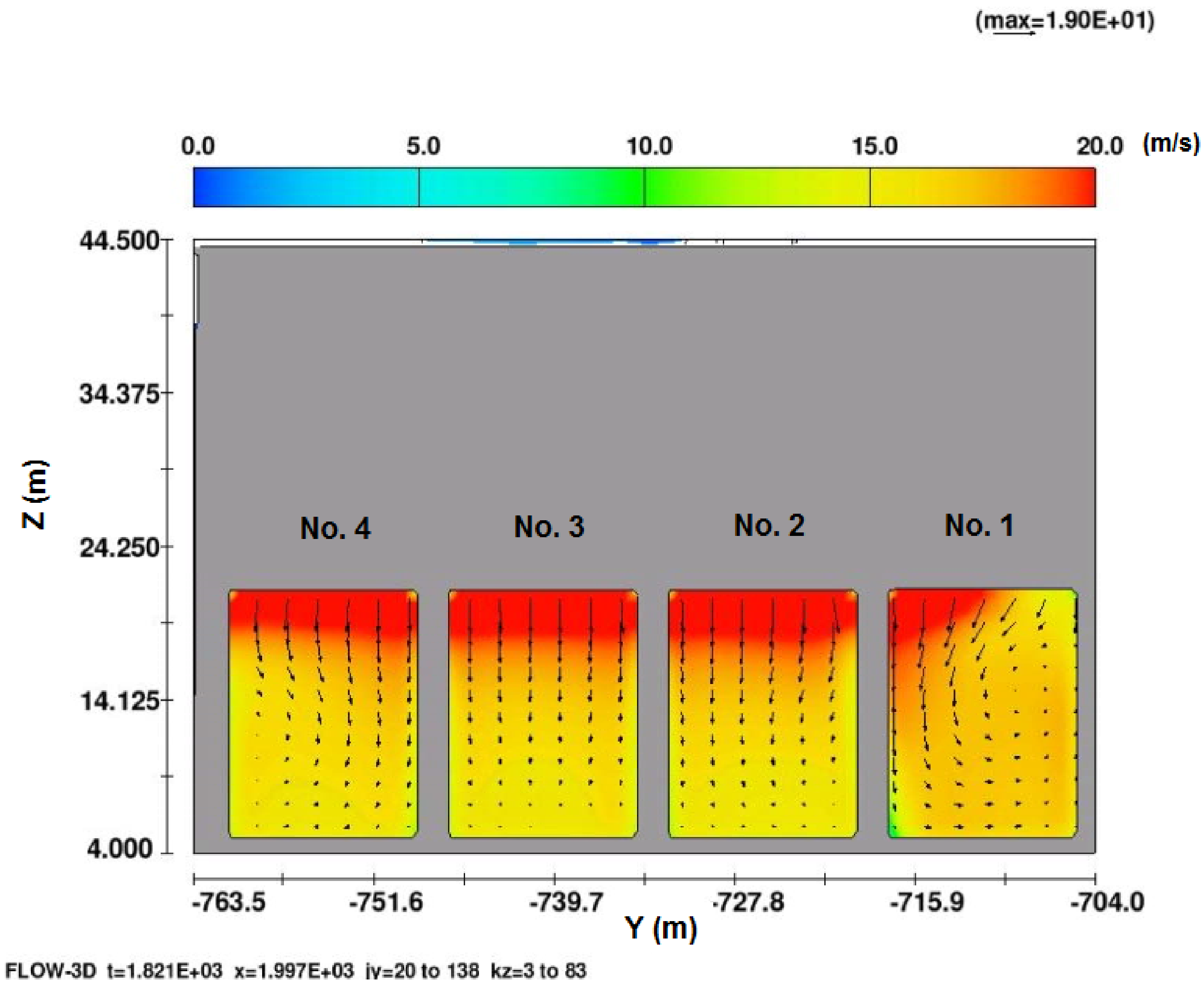


Figure 7-6: Muskrat Falls - Spillway Facilities - Proposed Layout Design Flow (22,100 m^3/s) - Cross Section View No. 1 of the RCC Dam

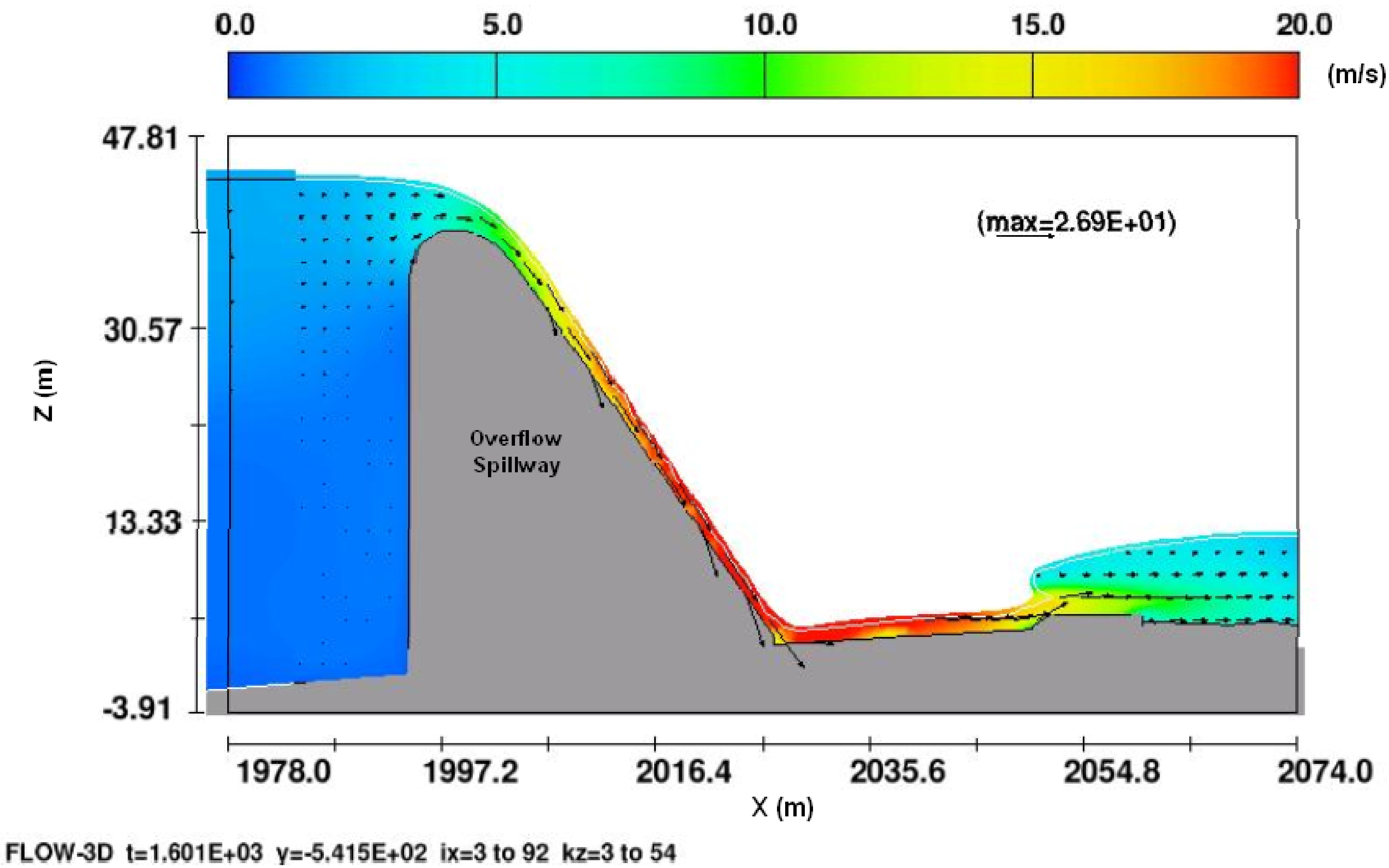


Figure 7-7: Muskrat Falls - Spillway Facilities - Proposed Layout Design Flow (22,100 m^3/s) - Cross Section View No. 2 of the RCC Dam

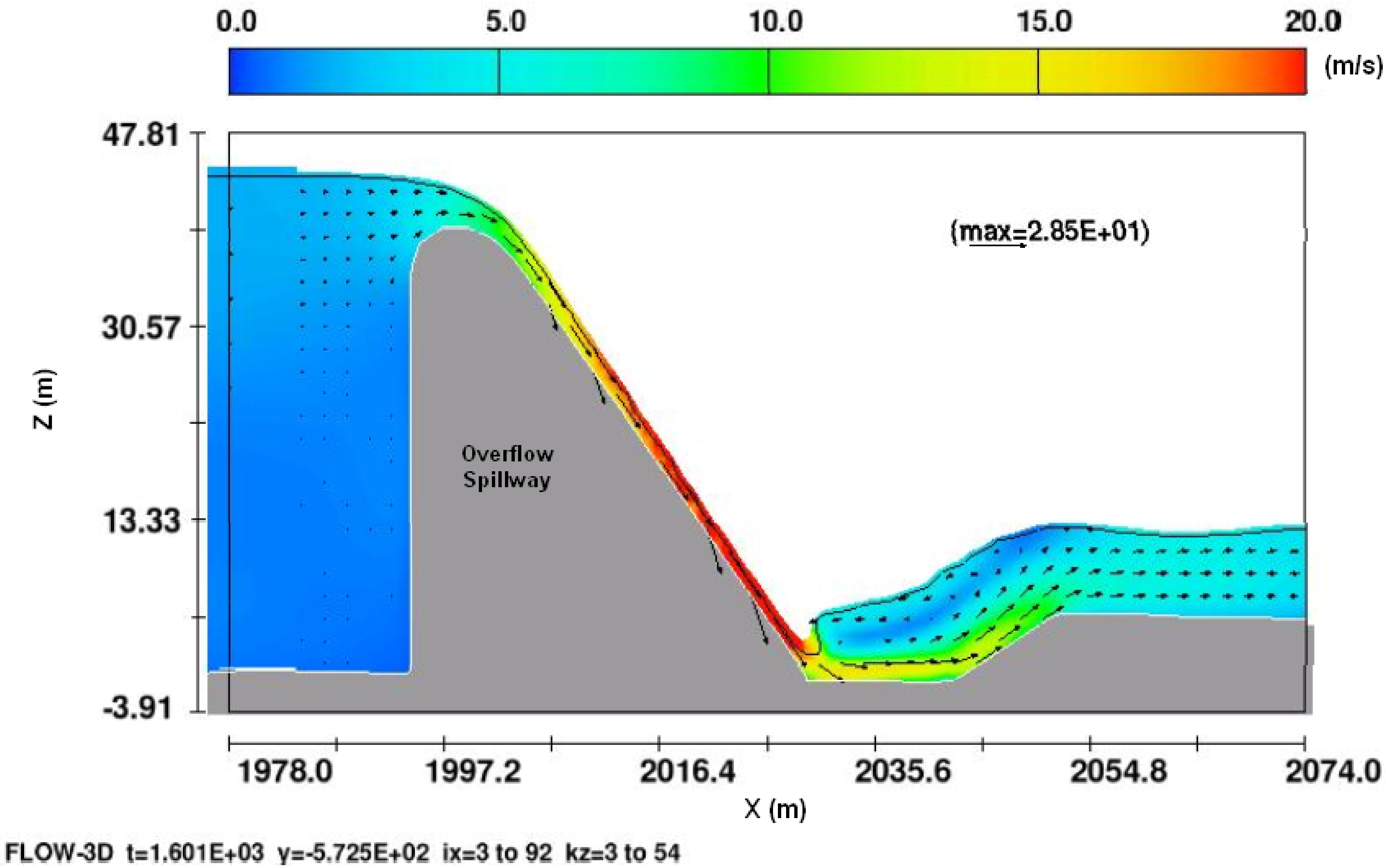


Figure 7-8: Muskrat Falls - Spillway Facilities - Proposed Layout Reservoir Level: 39 m - 4 Sluices Open - 3D View of the Spillway with Upstream View of the Vortex

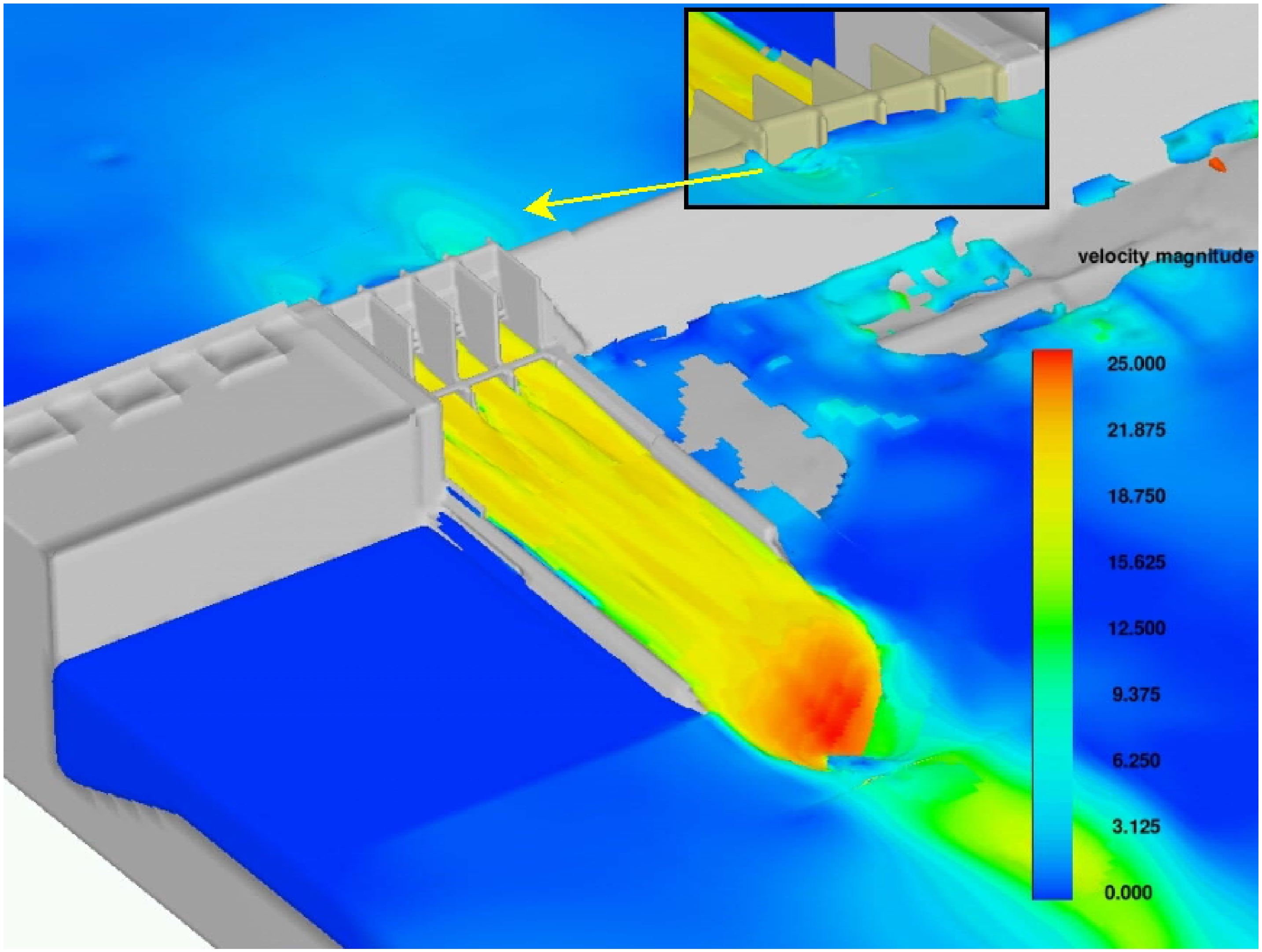


Figure 7-9: Muskrat Falls - Spillway Facilities - Proposed Layout Reservoir Level: 39 m - 4 Sluices Open – Plan View at Elevation 34 m

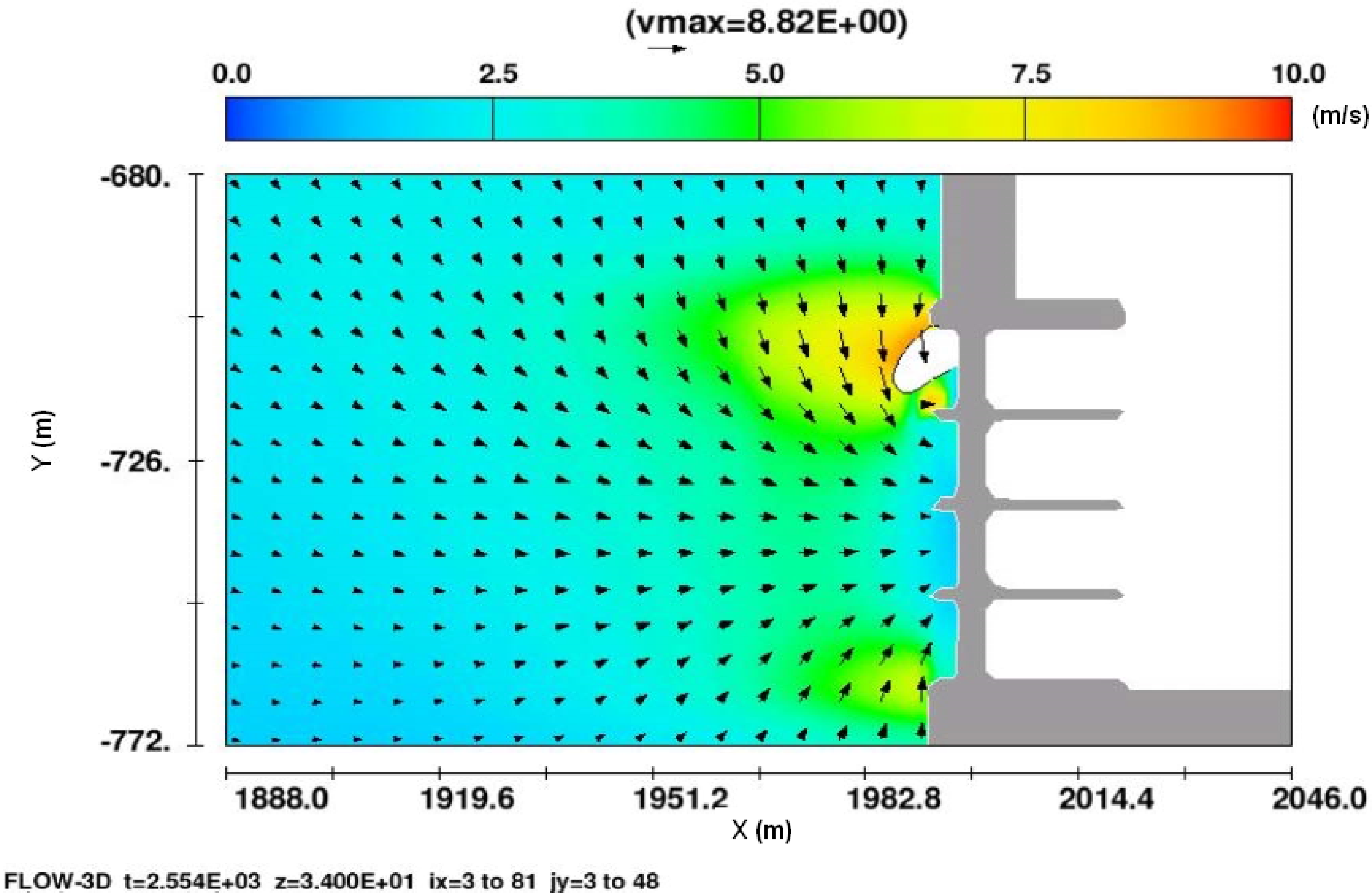


Figure 7-10: Muskrat Falls - Spillway Facilities - Proposed Layout Reservoir Level: 39 m - Cross Section View of the Four Sluices (looking upstream)

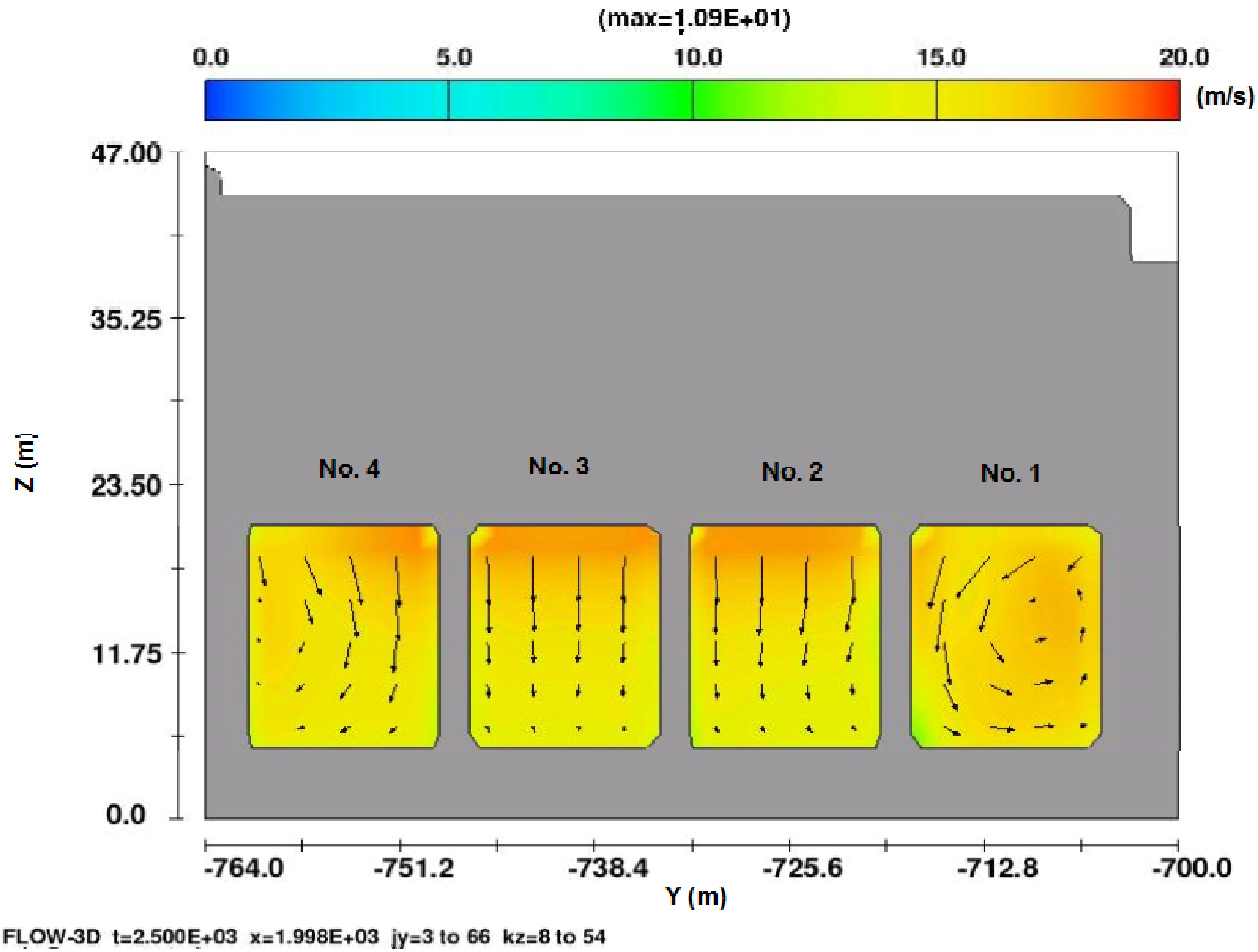


Figure 7-11: Muskrat Falls - Spillway Facilities - Proposed Layout Reservoir Level: 39 m - 4 Sluices Open - Cross-Section Sluice No. 2

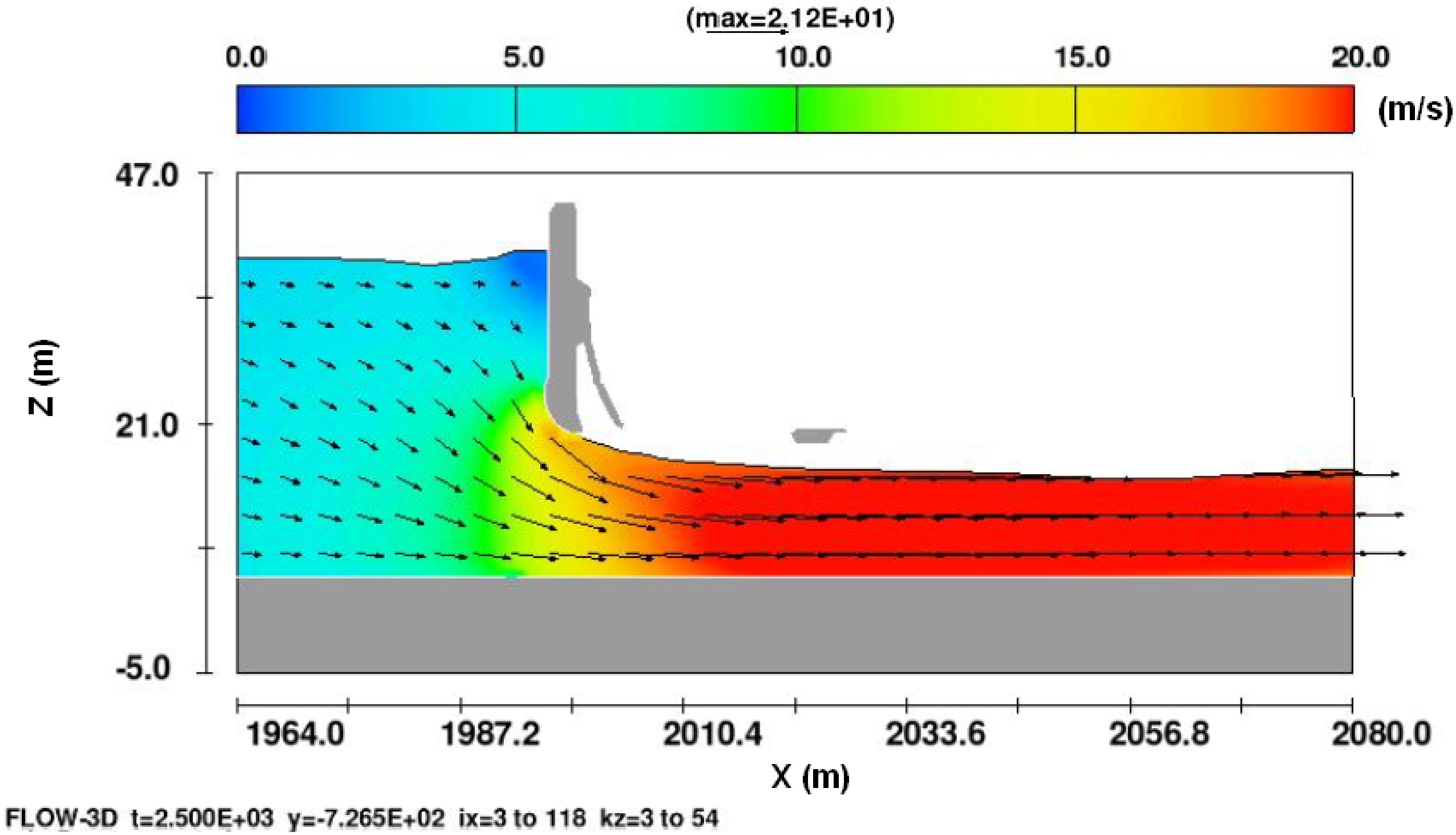


Figure 7-12: Muskrat Falls - Spillway Facilities - Proposed Layout Reservoir Level: 39 m - 4 Sluices Open - Cross-Section Sluice No. 1

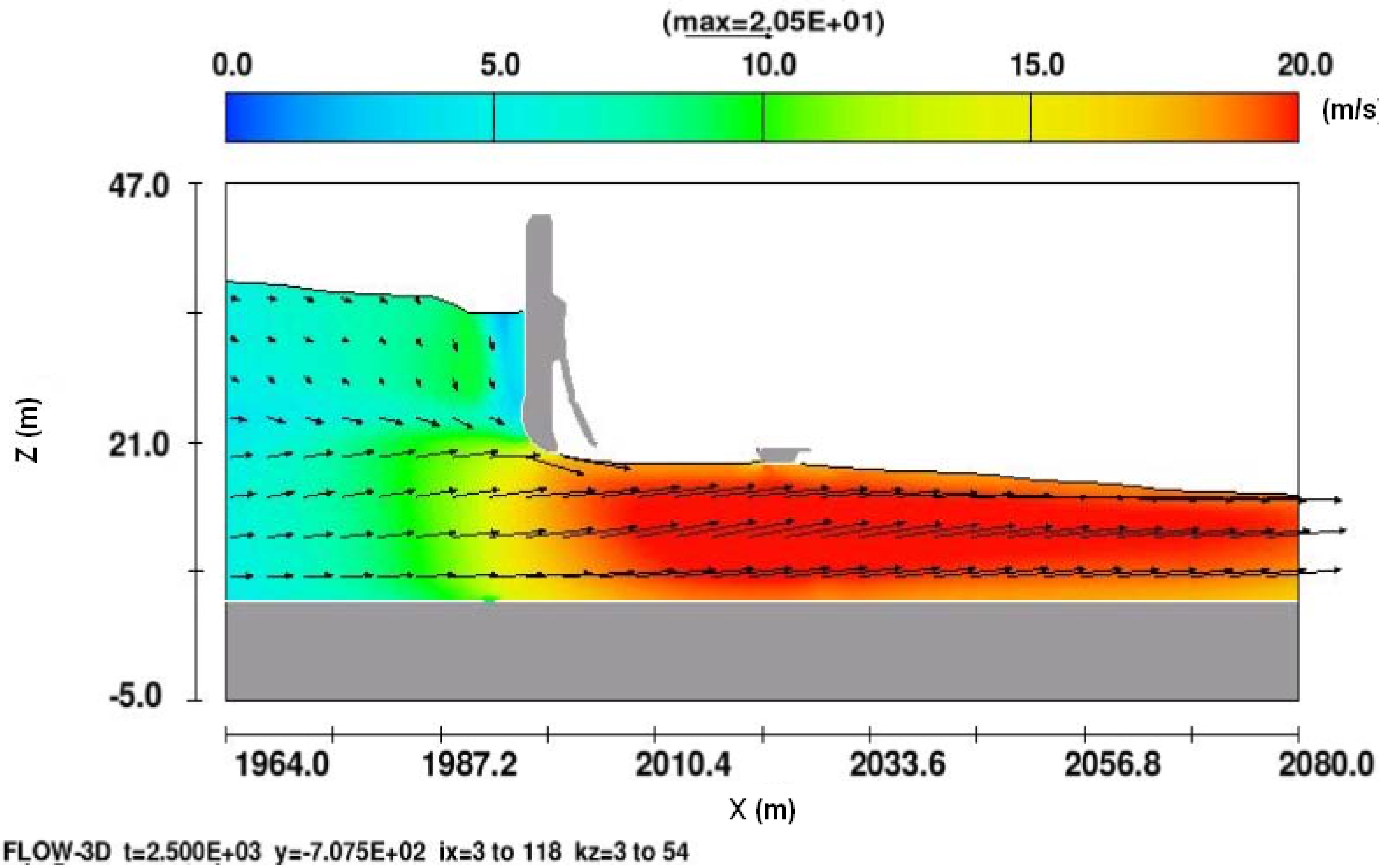
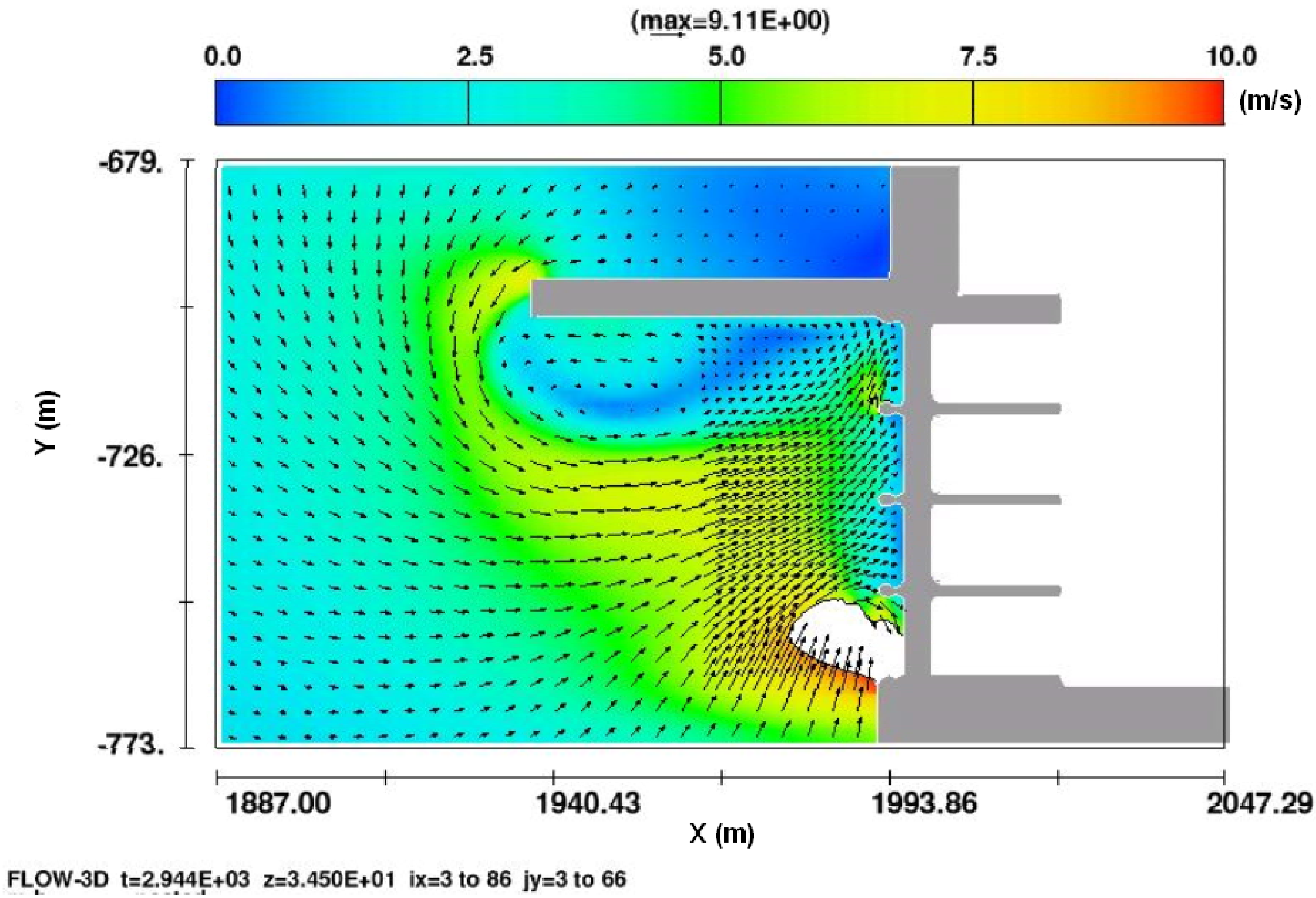


Figure 7-13: Muskrat Falls - Spillway Facilities - Proposed Layout + Left Wall Reservoir Level: 39 m - 4 Sluices Open - Plan View at Elevation 34.5 m



8 CONCLUSIONS

The hydraulic conditions at the main structures of the Muskrat Falls Hydroelectric Project have been evaluated by numerical modeling to simulate conditions expected to occur during the diversion phase (construction period) and/or during the operations phase.

For the diversion facilities, the numerical modeling has shown that the proposed approach channel should be modified to improve the flow conditions for the left sluices. A curved wall was added between the approach channel and the upstream cofferdam to improve the flow conditions and increase the capacity of the system.

At the power intake, the flow pattern at unit No. 1 (near the spillway) was disturbed by a strong lateral velocity, which can reduce the unit efficiency. Different variants were considered and it appears that a longer and higher retaining wall will be required, combined with an increase of excavation for the approach channel. This alternative improves the hydraulic conditions at the power intake considerably, but it should be further optimized to minimize the construction costs.

The analysis of the spillway facilities shows the presence of a vortex upstream of sluice No. 1. This vortex disturbs the flow pattern and probably reduces the discharge capacity of the system. Considering the possible increase of the PMF, a fifth sluice could be considered. It will increase the spill capacity of the system and will give more flexibility during the construction considering a possible increase of the design flood (normal operation at Churchill Falls). The impact of the wall between the power intake and the sluices should be considered in the next phase. A wall between the sluices and the RCC dam did not solve the problems noted on the proposed layout.

Final optimization of the layout should be performed by numerical and/or physical modeling after a review of the layout based on an update of the hydraulic conditions at Muskrat Falls and the results of the present report.

REFERENCES

1. Flow Science. "Flow 3D Software". http://www.flow3d.com/

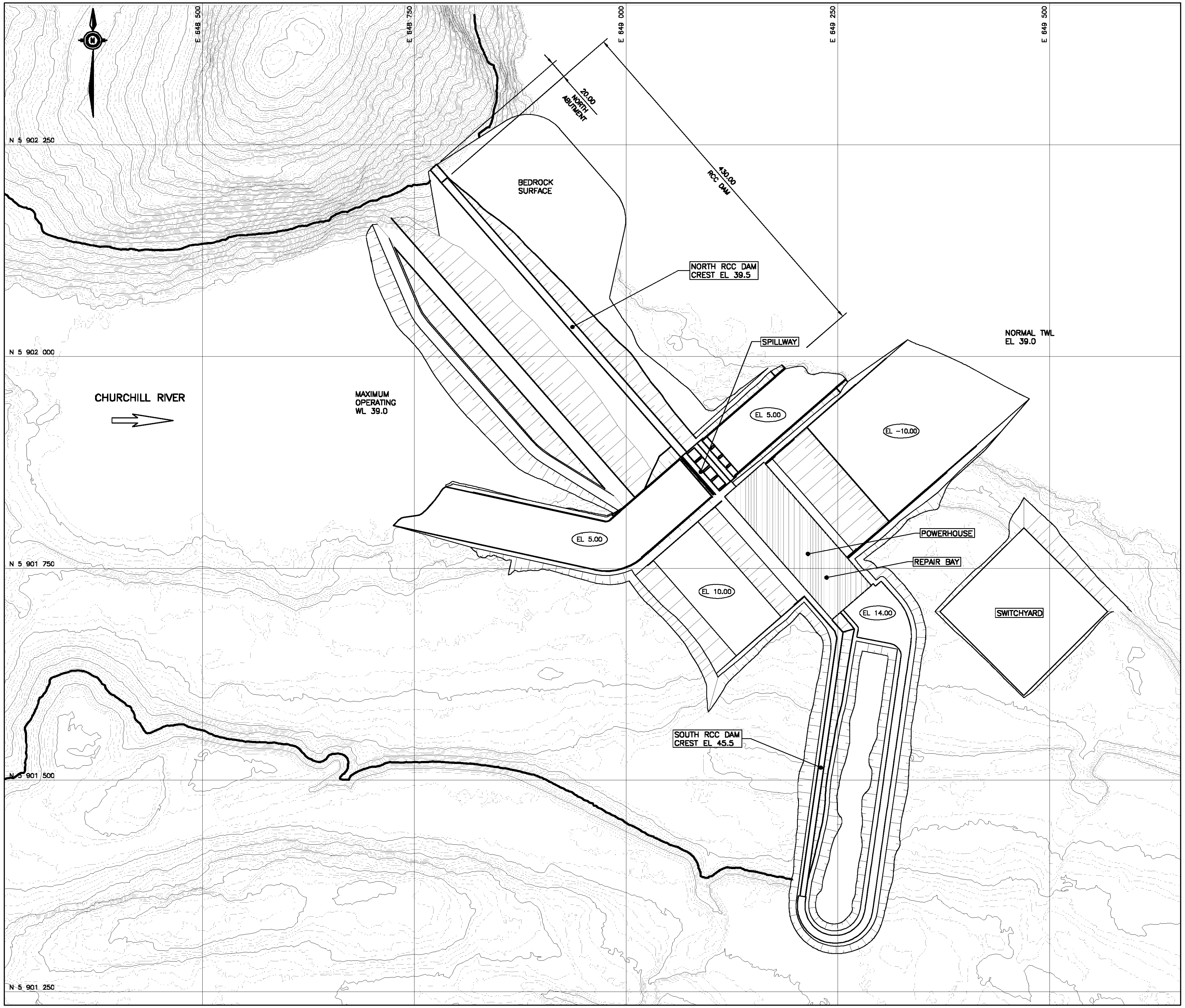
2. SNC-Lavalin, Muskrat Falls Hydroelectric Project, Final Report of WTO MF1050, Spillway Design Review, December 2007, 722850-MF1050-40ER-0001-00.

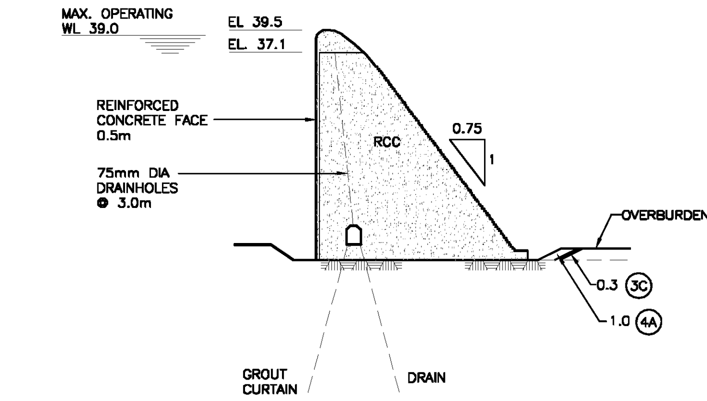
3. SNC-Agra, Muskrat Falls Hydroelectric Development, Final Feasibility Study, Volume 1 – Engineering Report, January 1999.

4. Richard O. Sinniger et Willi H. Hager "Constructions Hydrauliques Écoulements Stationnaires", Volume 15, Presses Polytechniques Romandes.

APPENDIX A

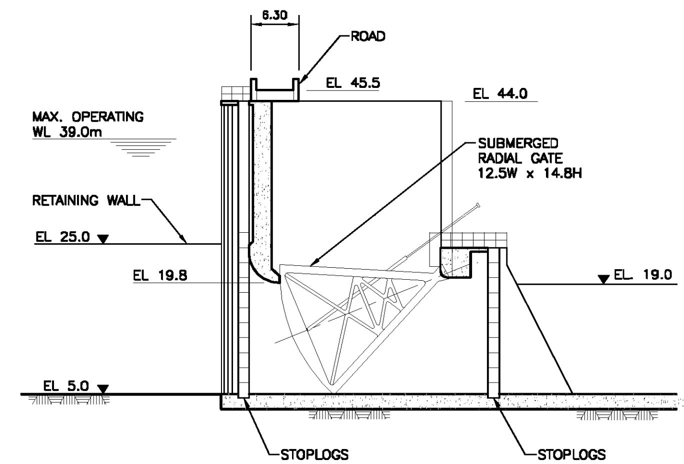
DRAWING





SECTION THRU NORTH RCC DAM

0 5 10 15 20 25 30 35
METRES



SECTION THRU SPILLWAY

0 5 10 15 20 25 30 35
METRES

SNC+LAVALIN
BAE-Newplan

BAE-Newplan Group Limited
1133 Topsail Rd. Mt. Pearl, NL, Canada A1N 5G2

NL HYDRO

NEWFOUNDLAND AND LABRADOR HYDRO

DWG.NO.	TITLE	NO.	DATE	DESCRIPTION	DWN.	DESIGN.	CHK.	APP'D
REFERENCE DRAWINGS		REVISIONS						

ELECT.	SCALE: AS NOTED
CIVIL	DESIGNED:
TRANS.	DRAWN:
MECH.	DATE:
P&C	CHECKED:
TELC.	APPROVED:

LOWER CHURCHILL PROJECT - MUSKRAT FALLS
ARRANGEMENT OF PRINCIPAL STRUCTURES - VARIANT 10
SPILLWAY SCHEME 3B (SUBMERGED RADIAL GATES)

W.O. NO. | DWG. NO. 722850- MF1050-41DD-0001 | REV.

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