Berg River Voëlvlei Augmentation Scheme (BRVAS) River Abstraction Works and Weir

Hydraulic Model Study

Final Report Rev01

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LIST OF ABBREVIATIONS

ADV	Acoustic Doppler Velocimeter
AEJV	Amanzi Entaba Joint Venture
AEP	Annual Exceedance Probability
ARI	Annual Recurrence Interval
BRVAS	Berg River Voëlvlei Augmentation Scheme
DEA	Department of Environmental Affairs
DWA	Department of Water Affairs
DWS	Department of Water and Sanitation
EIA	Environmental Impact Assessment
EWR	Environmental Water Requirement
LiDAR	Light Detection and Ranging
MAR	Mean Annual Runoff
MOL	Minimum Operating Level
NGL	Natural Ground Level
MM	Method of Moment
PWM	Probability Weighted Moment
RDD	Recommended Design Discharge
RI	Recurrence Interval
RMF	Regional Maximum Flood
SED	Safety Evaluation Discharge
SEF	Safety Evaluation Flood
SG	Specific Gravity
SU	Stellenbosch University
TCTA	Trans Caledon Tunnel Authority
TWL	Tailwater Level
VFD	Variable Speed Drive
WRC	Water Research Commission

1. Introduction

Stellenbosch University (SU) was appointed on 15 March 2021 by Bigen Africa Services (Pty) on behalf of the main consultant Amanzi Entaba Joint Venture (AEJV), to carry out a physical hydraulic model study of the proposed abstraction works and weir on the Berg River for the Berg River Voëlvlei Augmentation Scheme (BRVAS).

ASP Technology (Pty) Ltd was appointed during 2011 by the Aurecon Group to carry out the conceptual hydraulic design of the proposed river abstraction works on the Berg River at Voëlvlei Dam, as part of a pre-feasibility study of the proposed BRVAS (refer to ASP, 2012). The following are the outcomes from the latter study on a pre-feasibility level:

- A possible site for the abstraction works was identified on the left bank at a bend in the Berg River as shown in Figures 1-1 and 1-2 below. It was recommended that the abstraction and weir site location should be optimized in the physical model study.
- A topographical survey with limited underwater survey data of the site and geological information were obtained from Aurecon.
- A preliminary hydrological study was carried out to determine flood peaks for different recurrence periods.
- A two dimensional hydrodynamic model was used to simulate the flow patterns and sediment dynamics in the relevant zone of the Berg River to aid in the conceptual hydraulic design of the abstraction works.
- A hydraulic design of the proposed abstraction works was conceptualized (refer Figure 1-3).

Subsequent to the above pre-feasibility study, Stellenbosch University was contracted during 2021 to do a physical model study to refine the preliminary design towards a final detail design of the proposed abstraction works and weir. Model tests were done from June to December 2021.

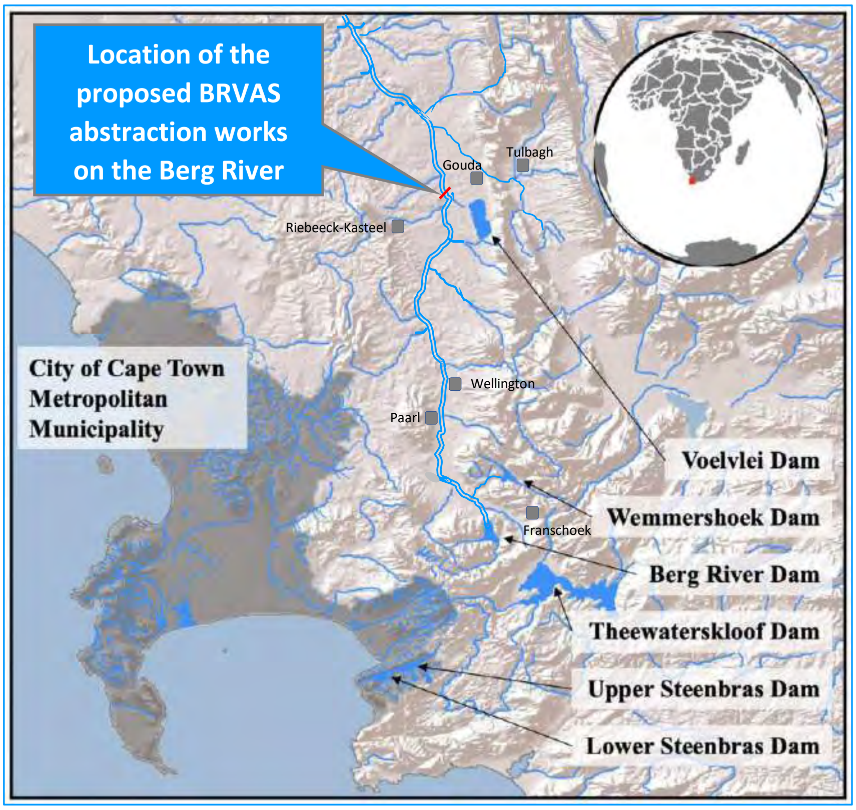


Figure 1-1: Location of the proposed abstraction works on the Berg River

Figure 1-2: Location of the proposed abstraction works on the Berg River as determined in the pre-feasibilty study of 2012 (ASP, 2012)

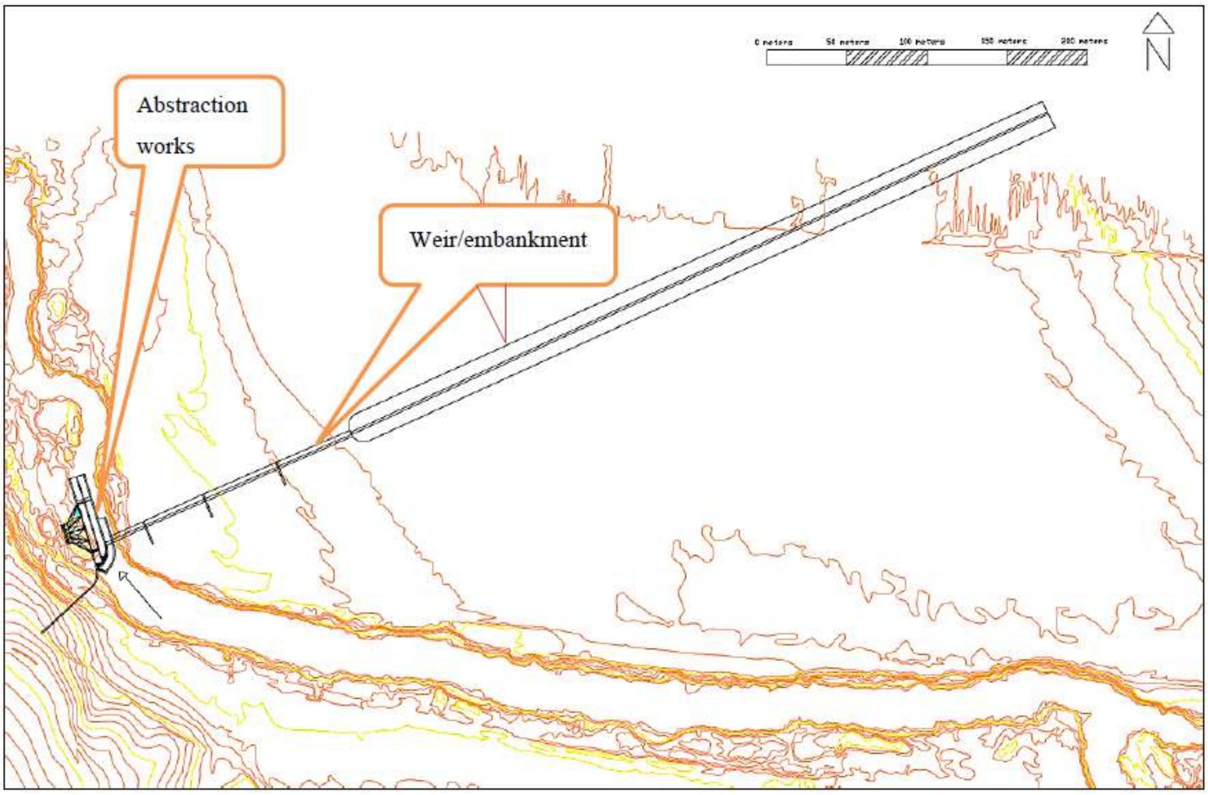


Figure 1-3: Plan layout of preliminary design of the abstraction works and weir as defined in the pre-feasibilty study of 2012 (ASP, 2012)

This report presents the following:

- The scope of the three-dimensional (3D) physical model study.
- A description of the ASP pre-feasibility design of the abstraction works and relevant river reach.
- The updated flood hydrology and the design floods tested for the abstraction works and weir.
- Field work sediment sampling and updated topographic data for this study.
- A description of the design of the 3D physical model including relevant river reach and model boundaries.
- A description of the tests and measuring methodology for the model study.
- Revised hydraulic design of the abstraction works.
- Results from the physical model study including temporary works design and tests.
- Hydrodynamic modelling of the flow patterns and fluvial morphology with the abstraction works and weir in a hybrid approach with the physical modelling.
- The design and physical model study of the proposed fishway-canoe chute.
- Proposed operation of the river abstraction works including sediment loads to be abstracted.

Several iterations of the hydraulic model study were done to optimize the orientation and design of the proposed BRVAS weir and abstraction works. A preliminary study was done for the current scenario based on the survey data of the 2012 feasibility study (with limited underwater survey data). Subsequently, the topographical survey, sediment sampling and flood hydrology were updated based on new data and work done in 2021 (this study). Only the hydraulic results and drawings for the new proposed abstraction works are presented in subsequent sections (unless specified otherwise).

2. Main feature and scope of the 3D physical hydraulic model study

2.1 Main features of the physical model

Some of the main features of the model are described below:

a) An undistorted 3D physical model at a scale of 1:40 to minimize scale effects. The model study was based on the Froude scale laws.

b) A river reach of approximately 1750 m was modelled as shown in Figure 1-2 of which 1500 m was on the upstream side of the abstraction works and 250 m downstream of it. The consequent model length and width was 30 m by 27 m respectively.

c) At the proposed scale (1:40) the maximum discharge that could be tested in the laboratory exceeded the 100-year flood with future climate change impact (1468 m^3/s) and it was possible to test the RMF flood with future climate change (RMFcc) of 4494 m^3/s.

d) The abstraction works and weir were constructed from marine ply while the river reach was filled with sand and topped with concrete to the surveyed contour levels. Due to the late arrival of the 2021 topographical survey data the physical model was constructed using the 2012 LiDAR survey data and the main channel of the Berg River was later reconstructed when the 2021 LiDAR and underwater survey data became available. A comparison between the 2012 and 2021 survey levels on the floodplains indicated negligible elevation differences.

e) The upstream inflow was measured by an electro-magnetic flow meter while the downstream tailwater was controlled by gates.

Fixed bed model tests were later complimented with movable bed tests to evaluate the local sediment deposition and erosion patterns at the proposed weir and abstraction works. For the movable bed tests, the river bed was topped with concrete to the expected rock elevations and covered with sediment scaled to be representative of the in situ riverbed sediment (Section 4.2).

2.2 Scope of the physical model study

The general scope of the required model study included the following:

a) Hydraulic design of the Crump weir (height, orientation, design head, notch lengths and dividing walls) as well as physical and numerical modelling to optimize the design. The weir's hydraulic design was also be discussed with DWS to establish accurate flow measurement requirements.

b) Sediment deposition upstream and local scour downstream of the weir were evaluated during movable bed tests. The geotechnical information supplied during the hydraulic model study based on the 2012 feasibility study data indicated that bedrock was available at the proposed site at an elevation of approximately 40 to 45 masl (bedrock durability to be indicated by geotechnical team). If the weir is fixed on the solid bedrock additional energy dissipation or erosion protection (roller bucket; riprap) at the proposed Crump weir is not required.

c) Orientation of the proposed abstraction works at the river bend and also relative to the weir for self-cleaning of the sediment traps during floods.

d) Flow patterns in the abstraction works at peak pumping under normal operating conditions.

e) Provisional selection of suitable duty pumps.

f) Evaluation of the sediment control and flushing efficiency of the sediment traps at the abstraction works (a low weir and high tailwater levels could lead to poor sediment flushing during small floods).

g) Hydraulic design and tests on a proposed fishway-canoe chute, including recommendation for its optimal location.

h) Measurement of the water levels at the right bank berm and testing where erosion protection will be required.

i) Evaluate flood levels and flood lines and the required top of the abstraction works elevation with freeboard.

j) Establishment of the H-Q relation of the weir including the hydraulic head for different river flow rates across the weir available for flushing of the different components of the abstraction works.

k) Recommendations for the temporary works design, with fixed bed and movable bed tests of the proposed works.

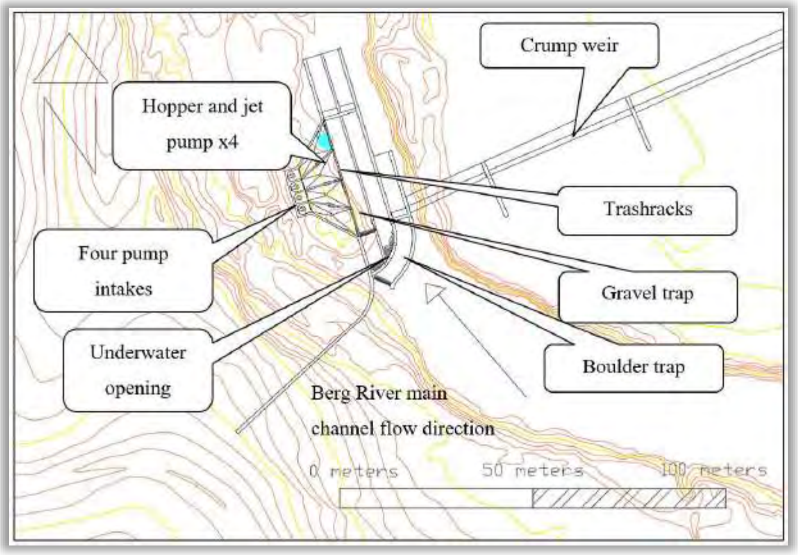
3. Description of the abstraction works and weir as per preliminary study (ASP, 2012) as basis for the new design

The layout and relevant dimensions of the 2012 preliminary design of the abstraction works are presented as excerpts from the ASP (2012) study to describe the following items:

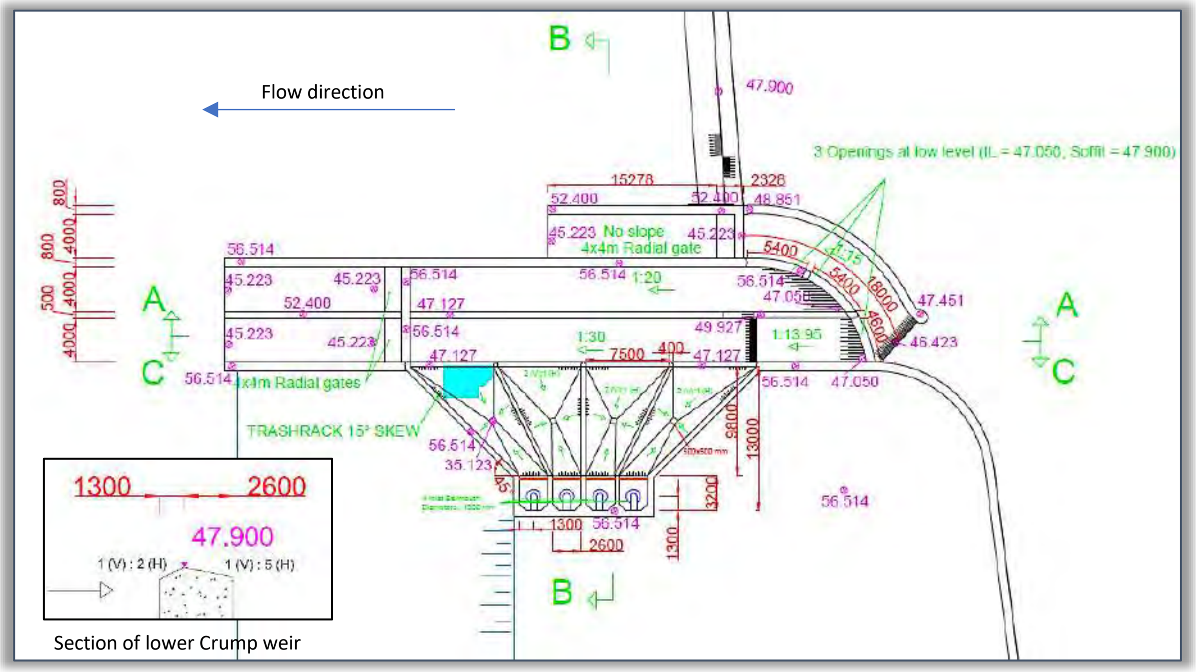
- General plan layout of abstraction works and part of weir (Boxes 3-1 and 3-2).
- Sections of the abstraction works (Box 3-3).
- Summary of main dimensions of preliminary abstraction works design (Table 3-1).

Note that the dimensions and layout of the design were updated in this study and are presented in Section 6 and **Appendix A**.

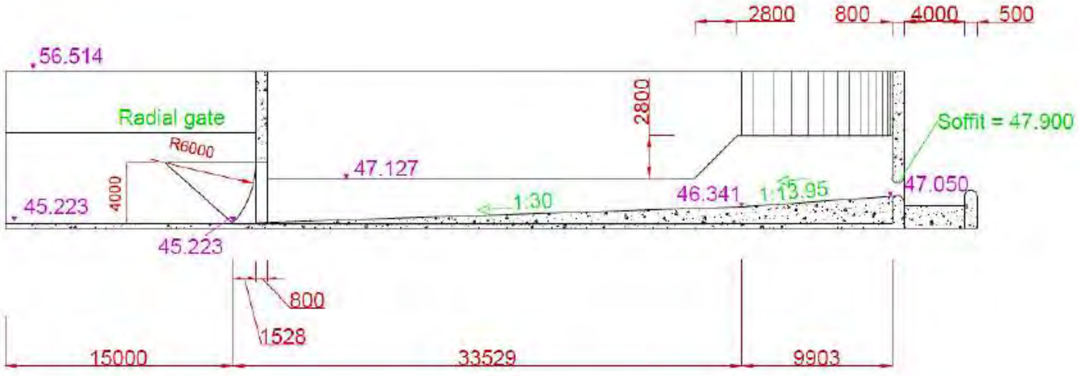
Box 3-1: Preliminary orientation of the different components of the abstraction works relative to 2012 surveyed river contours (excerpt from ASP, 2012)



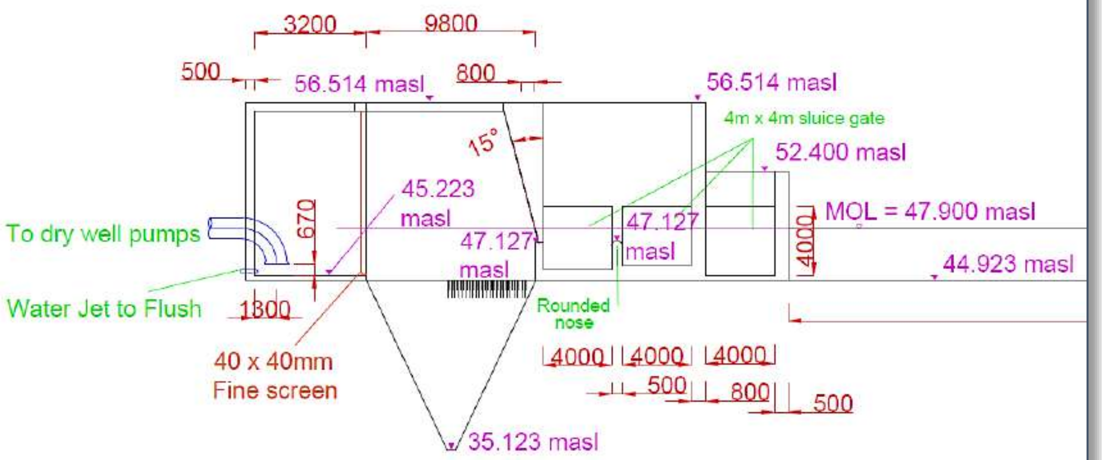
Box 3-2: General plan layout of outdated abstraction works (excerpt from ASP, 2012)



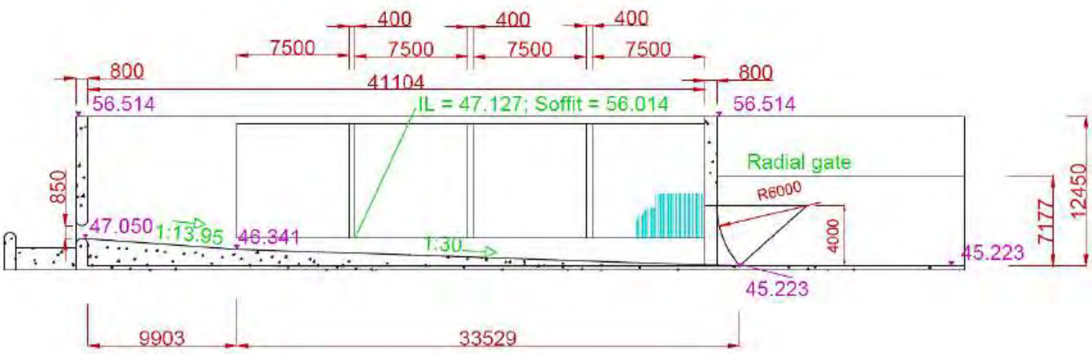
Box 3-3: Sections A-A, B-B and C-C (excerpts from ASP, 2012)



SECTION A-A



SECTION B-B



SECTION C-C

Table 3-1: Summary of dimensions of preliminary abstraction works design (ASP, 2012)

Design summary	Units	ASP (2012)
Low notch length	m	20
2nd notch length	m	40
3rd notch length	m	50
4th notch length (broad crested)	m	50
Low notch crest elevation (Fishway-canoe chute)	masl	47.9 (MOL)
2nd notch crest elevation	masl	50.4
3rd notch crest elevation	masl	52.4
4th notch crest elevation	masl	54
Lowest river bed elevation at site	masl	44.92
Low notch height above river bed	m	2.98
Discharge capacity low notch only	m^3/s	71
Opening length	m	15.4
Opening height	m	0.85
Opening invert level	masl	47.113
Trashrack length	m	30
Trashrack minimum height required	m	0.774
Trashrack invert level	masl	47.127
Number of pump bays	#	4
Width of pump bays	m	2.6
Number of duty pumps	#	3
Number of standby pumps	#	1
Total duty pump capacity	m^3/s	6
Total standby pump capacity	m^3/s	2
Number of hoppers	#	4
Max width of hoppers	m	7.5
Hopper invert levels	masl	35.123
Min volume required (100Q)	m^3	600
Max volume required (200Q)	m^3	1200
Volume provided	m^3	970
Boulder trap (x1) width	m	4
Gravel trap (x2) width	m	4
Radial gate opening height (R = 6 m)	m	4
Min floor level downstream of traps	masl	45.223
Top of structure i.e. Q100cc	masl	56.514
Right bank side berm crest designed for RMF = SED without freeboard	masl	61.0

Note: "cc" means future climate change impact included

Note that the dimensions and layout of the design were updated in this study and are presented in Section 6 and **Appendix A**. A brief comparison of the 2012 and 2021 designs is also given in Table 9-1.

4. Flood hydrology, sediment grain sizes and topographic data relevant to the study

4.1 Flood peaks

The preliminary study of ASP (2012) derived flood peaks for the BRVAS site but the probabilistic flood hydrology analysis had to be updated with more recent data. Flood peak data was obtained for gauging station G1H013 at Drieheuvels for the period 1964 to 2011 for the ASP (2012) study and for the period 1964 to 2020 for this study. The catchment area at the gauging station is 2 934 km^2 while the proposed abstraction works site has a catchment area of 1 527 km^2. The observed flood peaks at G1H013 therefore had to be scaled for the BRVAS site by using the square root of the catchment ratio. More details on the flood hydrology analysis is enclosed in **Appendix C**.

Table 4-1 shows the flood peaks proposed at the abstraction site. Despite the updated flood peak record, the proposed flood peaks for the current scenario compare well with the ASP (2012) study which excluded any climate change impacts (denoted by “cc”). However, the proposed flood peaks show better agreement with the unit hydrograph deterministic method and Kovacs empirical method as explained in **Appendix C**. Table 4-1 shows a 15% flood peak increase that was incorporated to account for the impact of climate change on future flood scenarios. This is in agreement with the standard 15% approach by the City of Cape Town and with the DEA (2014) study of five (5) climate models for South Africa. More evidence for the proposed 15% increase for climate change impacts is given in **Appendix C**. Note that the ASP (2012) study designed the abstraction works for a final theoretical 100-year flood peak of 1 500 m^3/s which is near identical to the flood peak of 1 468 m^3/s that was calculated in this study.

Table 4-1: Proposed updated flood peaks for current and future scenarios at the abstraction site

Flood recurrence interval (years)	Flood peak from previous ASP (2012) study	Flood peak for current scenario (m^3/s)	Flood peak for future scenario including climate change (cc) with 15% increased Q (m^3/s)
2	223	182	210
5	403	369	424
10	551	533	613
20	715	722	830
50	969	1 016	1 169
100 (RDD)	1 168	1 276	**1 468**
200	1 400	1 572	1 808
RMF (SED)		3 908	**4 494**

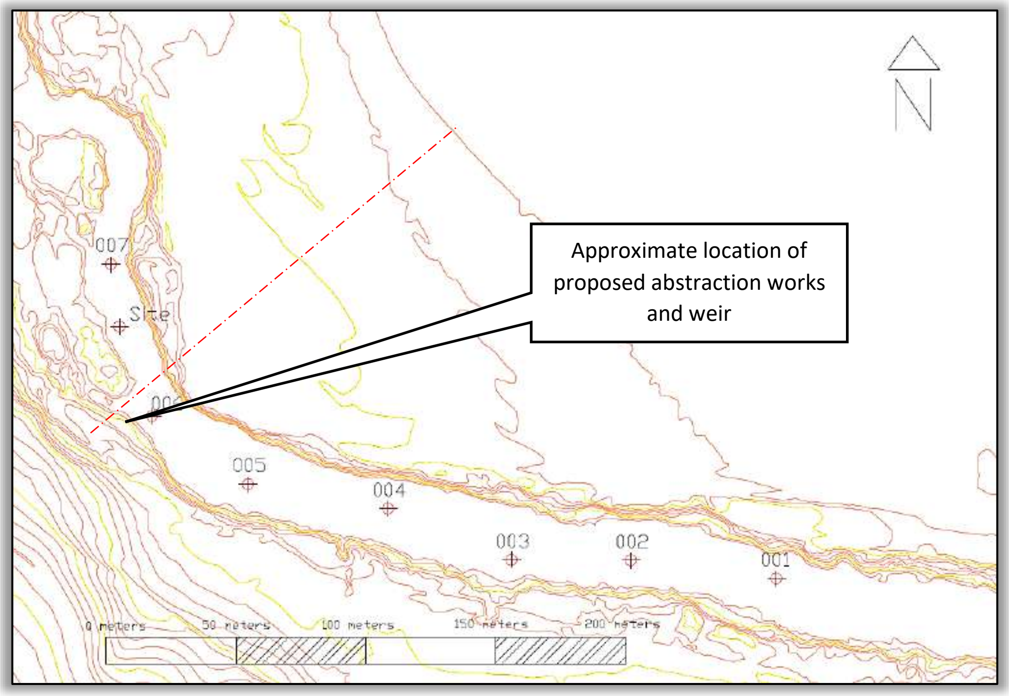
Note: “cc” means future climate change impact included

The Recommended Design Discharge (RDD) is the Q100cc which was used to establish the recommended elevations of the BRVAS structures including freeboard. The Regional Maximum Flood (RMF) based on TR137 was used as the Safety Evaluation Discharge (SED) in consideration of the earthfill embankment on the right bank floodplain. The current scenario RMF peak is 3 908 m^3/s. Note that the RMFcc of 4 494 m^3/s was tested in the laboratory to determine the berm crest level.

4.2 Sediment grain sizes in the Berg River bed

The ASP (2012) study collected bed samples in the Berg River at different locations on the upstream side of the proposed abstraction works (Box 4.2-1) for which the grain size distribution at these locations are presented in Box 4.2-2. However, during this study new bed grab samples were collected in the river bed and on the floodplain at the weir site. Grading (sieve and hydrometer test) and specific gravity (SG) tests were conducted at a geotechnical laboratory. The data is enclosed in **Appendix E** for which a summary of the d_{50} sediment sizes and SG values are given in Table 4-2. The locations for the sediment samples are shown in Figure 4-1.

Box 4.2-1: Locations 001 to 007 where river bed samples were taken (2012)



Box 4.2-2: Summary of sediment grain size distribution on riverbed (2012)

		Unit	Sample 001	Sample 002	Sample 003	Sample 004	Sample 005	Sample 006	Sample 007	Average d_{50} used*
Class 1	0-20%	mm	0.01	0.031	0.18	0.06	0.14	0.075	19	0.083
Class 2	20-40%	mm	0.05	0.07	0.37	0.33	0.40	0.26	30.00	0.247
Class 3	40-60%	mm	0.13	0.15	0.54	0.50	0.64	0.42	39.00	0.397
Class 4	60-80%	mm	0.23	0.30	0.84	0.80	1.00	0.65	42.00	0.637
Class 5	80-100%	mm	0.44	0.54	1.50	1.70	2.20	1.20	47.00	1.263

Note:* Used in numerical model. Average of samples 001 to 006. Sample 007 not representative.

Table 4-2: Summary of median bed sediment particle size and SG obtained for this study (2021)

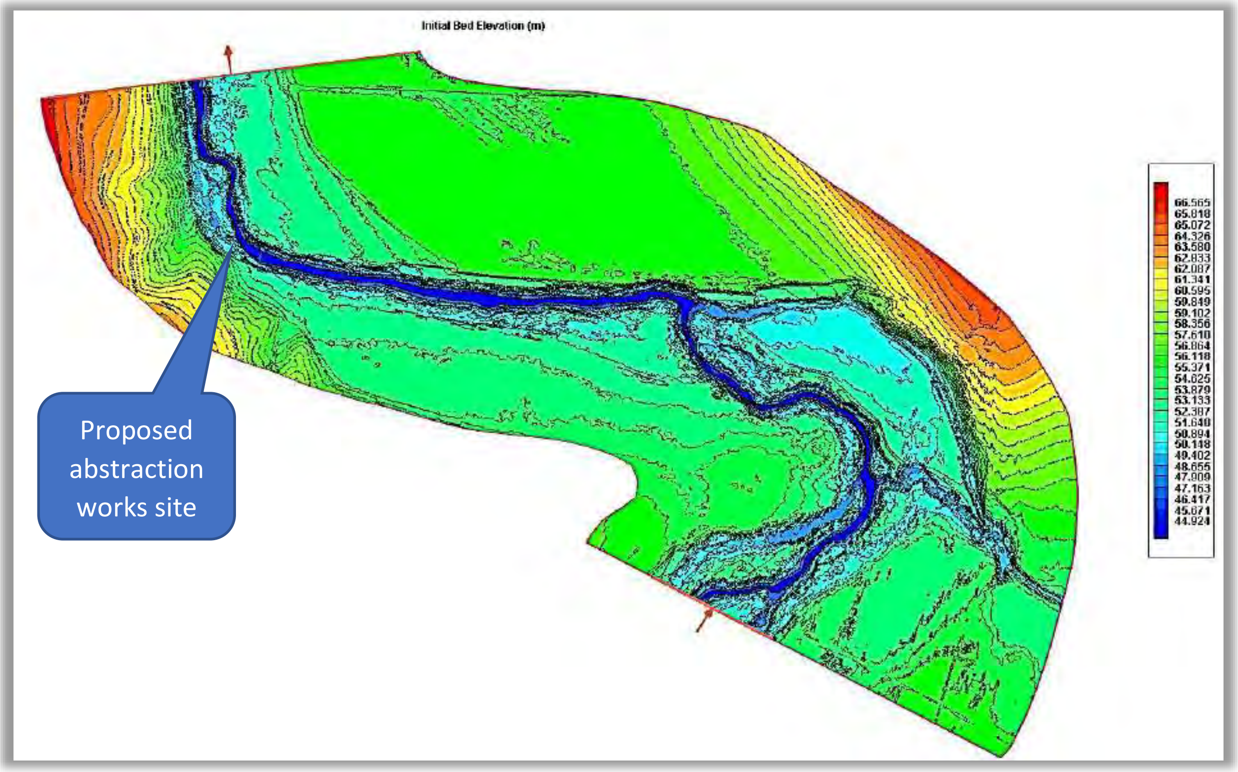
Sample Name	Sample Number	Co-Ordinates		Sieve & Hydrometer Analysis d_{50} (mm)	SG
		Latitude (South)	Longitude (East)		
Right Bank Samples	BG 1	-33°19.699'	18°58.846'	0.200	2.62
	BG 2	-33°19.689'	18°58.873'	0.200	2.51
	BG 3	-33°19.679'	18°58.900'	0.100	2.40
	BG 4	-33°19.684'	18°58.838'	0.220	2.57
	BG 5	-33°19.674'	18°58.865'	0.183	2.56
	BG 6	-33°19.664'	18°58.892'	0.117	2.45
	BG 7	-33°19.677'	18°58.934'	0.111	2.49
	BG 8	-33°19.666'	18°58.965'	0.139	2.52
	BG 9	-33°19.654'	18°58.996'	0.156	2.54
River Bed Grab Samples	BG 10	-33°19.835'	18°59.327'	0.128	2.56
	BG 11	-33°19.834'	18°59.326'	2.000	2.51
	BG 12	-33°19.833'	18°59.327'	0.167	2.47
	BG 13	-33°19.767'	18°59.127'	0.260	2.59
	BG 14	-33°19.763'	18°59.126'	0.880	2.63
	BG 15	-33°19.762'	18°59.119'	0.150	2.56
	BG 16	-33°19.751'	18°58.967'	0.178	2.56
	BG 17	-33°19.749'	18°58.968'	0.375	2.63
	BG 18	-33°19.747'	18°58.968'	0.106	2.56
	BG 25	-33°19.662'	18°58.810'	0.240	2.65
	BG 26	-33°19.663'	18°58.814'	2.200	2.65
	BG 27	-33°19.660'	18°58.814'	0.296	2.65

Figure 4-1: Locations of new bed grab samples collected for this study (2021)

4.3 Survey data of the relevant river reach

The physical model was initially constructed from available survey data from the ASP (2012) study to do a preliminary study for the current scenario. The survey data consisted of a LiDAR survey provided by Aurecon and 3 river cross-sections that were combined with the ASP study (shown in Box 4.3-1). However, the LiDAR survey data downstream of the proposed abstraction works was lacking and more detailed topographic data in this zone had to be obtained for the physical model study to extend the survey data shown in Box 4.3-1 further downstream for reliable tailwater levels in the physical model. Subsequently, new river survey data (LiDAR and underwater surveys) was received on 26 May 2021 as shown in Figure 4-2.

Box 4.3-1: LiDAR surveyed topography of relevant river reach in masl (excerpt from ASP, 2012)



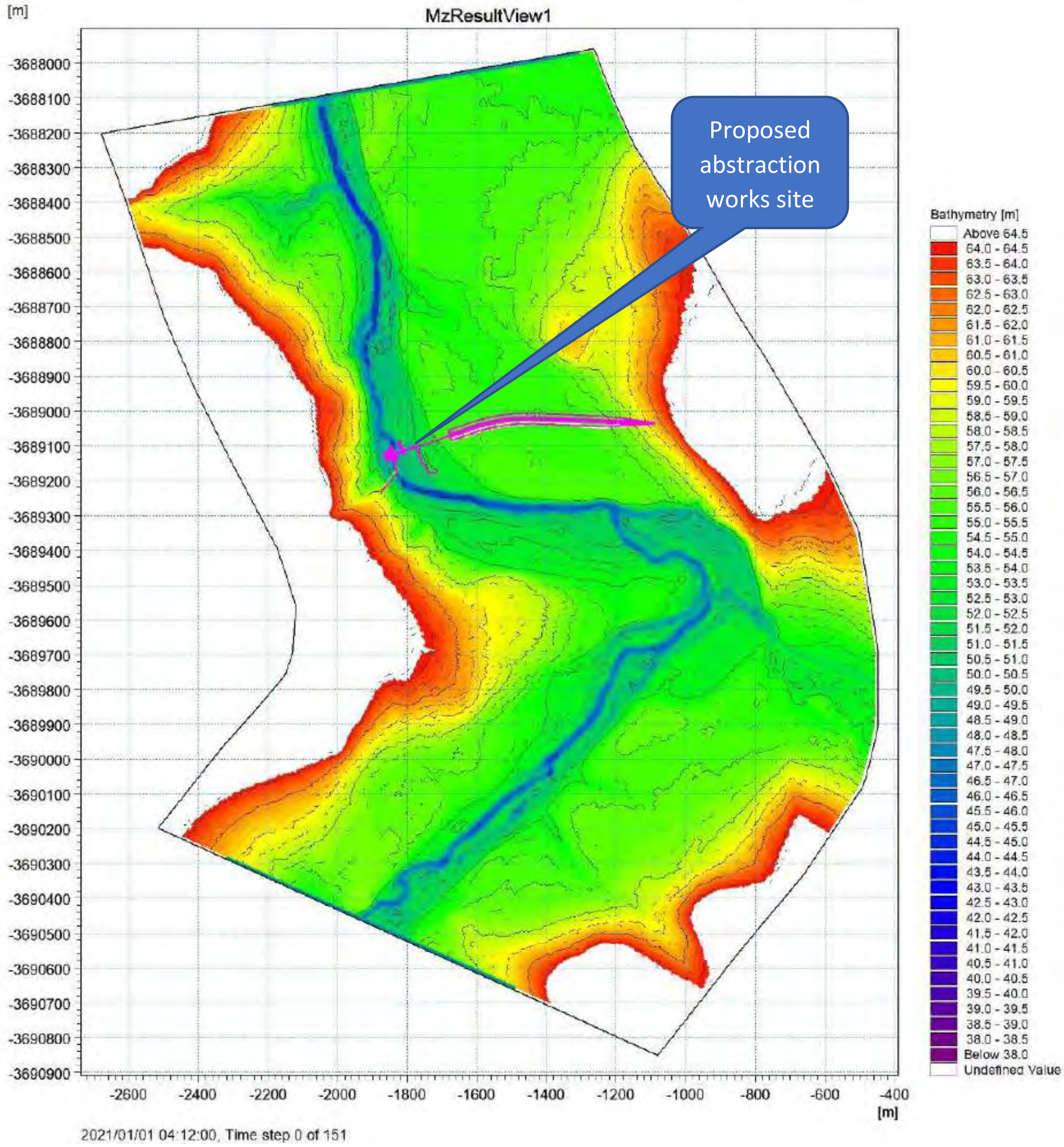


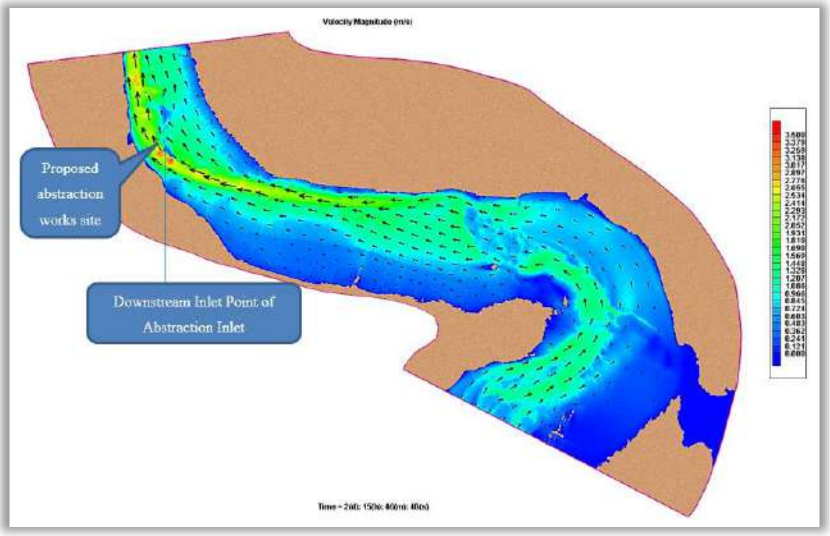
Figure 4-2: Updated bathymetry in masl (2021 LiDAR and underwater surveys) used in this study

5. Hydrodynamic modelling of flow distribution and fluvial morphology in the relevant river reach

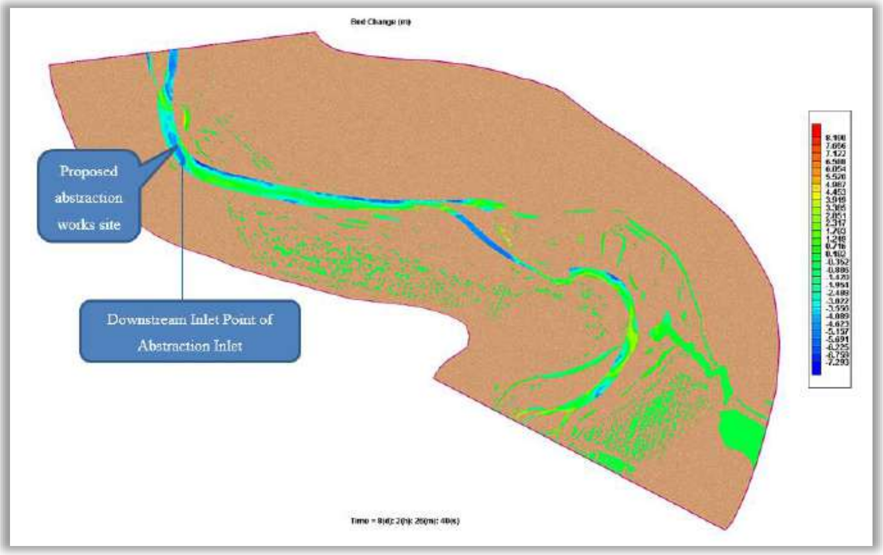
5.1 Preliminary numerical model simulations by ASP (2012)

Preliminary numerical simulations by means of numerical hydrodynamic and morphological models (CCHE2D) were performed in the preliminary study by ASP (2012), examples of the results of these simulations are presented in Boxes 5-1 and 5-2 for the 10 year flood. Subsequently, in the current study, additional hydrodynamic modelling were done by using the 2D hydrodynamic model Mike21C, for the 2-year, 100-year floods and Regional Maximum Flood (RMF), results of which are presented in Figures 5.1-1, 5.1-2 and 5.1-3, respectively. The physical model boundaries were selected based on the results of these simulations. The Manning's roughness coefficient (n) was used to account for bed roughness in the numerical models, a Manning's n coefficient of 0.045 (typical for a sand bed river) was used for the main channel and n = 0.06 was used for the flood plains.

Box 5-1: Simulated flow velocity and direction distribution for the 1:10 year flood (excerpt from ASP, 2012)



Box 5-2: Simulated bed change for the 1:10 year flood (excerpt from ASP, 2012)



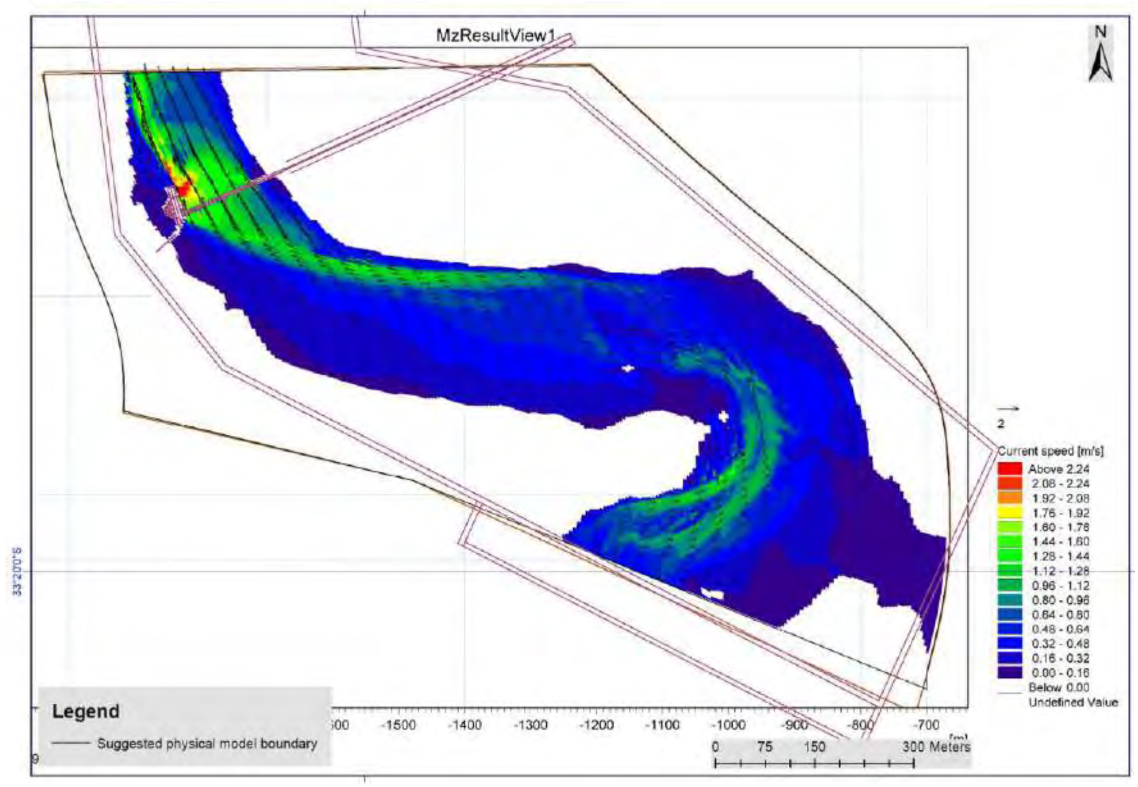


Figure 5.1-1: Simulated flow velocity and direction distribution for the 1:2 year flood (simulation of the current study based on the 2012 survey data)

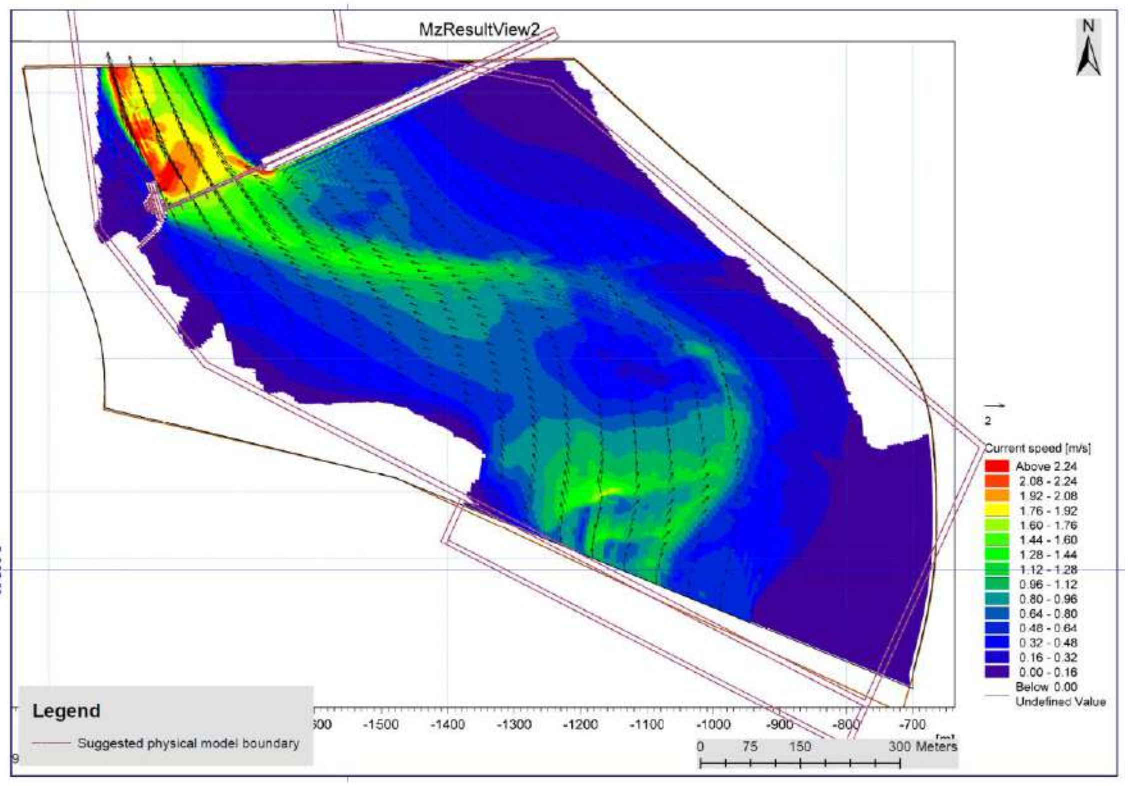


Figure 5.1-2: Simulated flow velocity and direction distribution for 1:100 year flood (simulation of the current study based on the 2012 survey data)

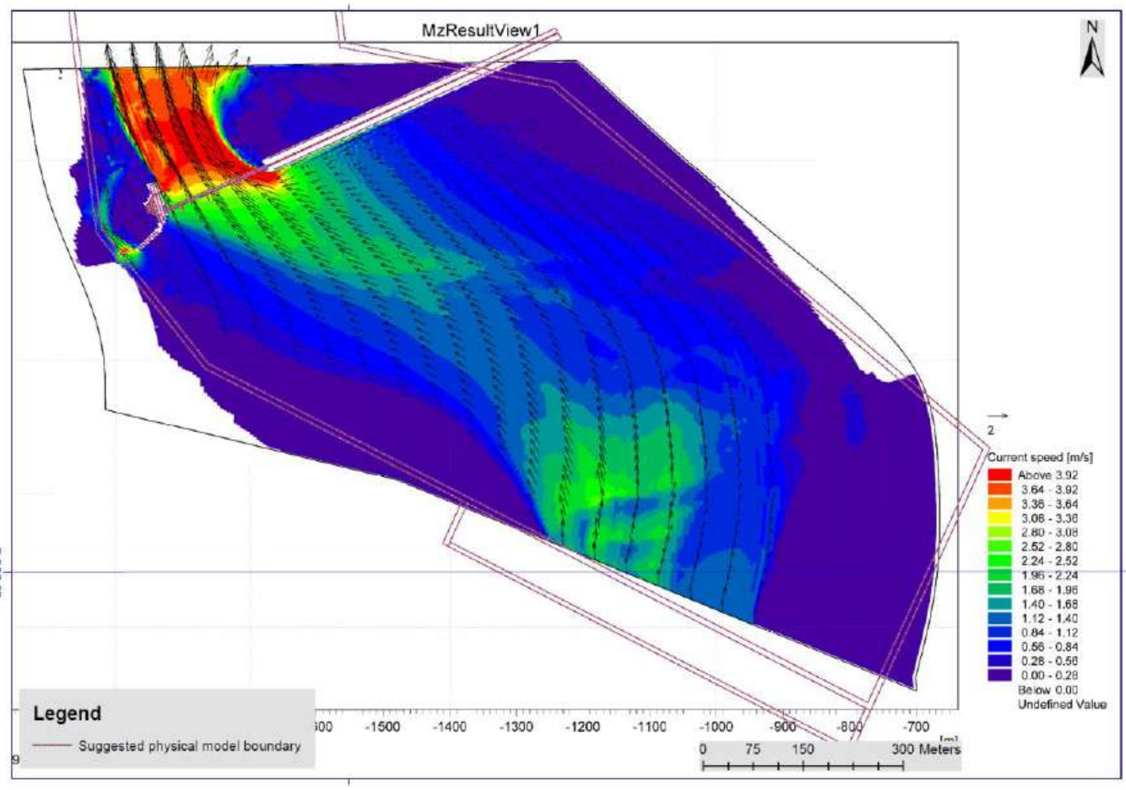


Figure 5.1-3: Simulated flow velocity and direction distribution for RMFcc (simulation of the current study based on the 2012 survey data)

5.2 Subsequent numerical model simulations based on the 2012 survey data

5.2.1 Objectives

Subsequent to the preliminary simulations by ASP (2012), additional Mike21C numerical model simulations were done in this study. The aims of the hydrodynamic modelling are as follows:

a) The physical modelling of the abstraction works adopted an approach with hydrodynamic modelling used in a hybrid approach. The numerical modelling provided the flow patterns at the inflow boundary of the physical model which are used to calibrate the physical model. The tailwater levels in the physical model which are controlled by gates were also obtained from the hydrodynamic model. The hydrodynamic model simulated flood levels were used to determine the freeboard of the physical model walls and abstraction works for the design flood and SED.
b) The hydrodynamic model was also used to simulate the low flow to flood water levels along the main channel with the weir constructed and with the current scenario, and with movable bed conditions, to evaluate the effect of the scour caused by the weir on the local tailwater levels. The data was used for the preliminary hydraulic design of the sediment traps, canoe chute and fishway prior to physical model testing.
c) For the final abstraction works and weir design, based on the new LiDAR and bathymetric survey data of 2021, the long term sedimentation upstream and downstream of the weir was simulated by using a 15 year historical observed river flow record. (Only 15 years of reliable record was available from DWS, but this was deemed long enough for the river to reach a new equilibrium). The sedimentation upstream of the weir could affect flood levels and floodlines were also determined based on the maximum water levels from the hydrodynamic and physical model tests.

5.2.2 Hydrodynamic model setup

The hydrodynamic model was set up based on contour survey data of the 2012 feasibility study. The model bathymetry was generated using a curvilinear mesh with cell sizes of about 5 m in the flow direction and 2 m in the direction perpendicular to the flow. The Berg River reach included in the model is 1.6 km long along the main channel. The distance from the downstream boundary to the proposed abstraction works is 230 m. The underwater profile of the main channel of the river is based on very limited survey data which was only updated later based on the new survey data of 2021 received during this study. Figure 5.2-1 shows the model bathymetry based on the 2012 survey data.

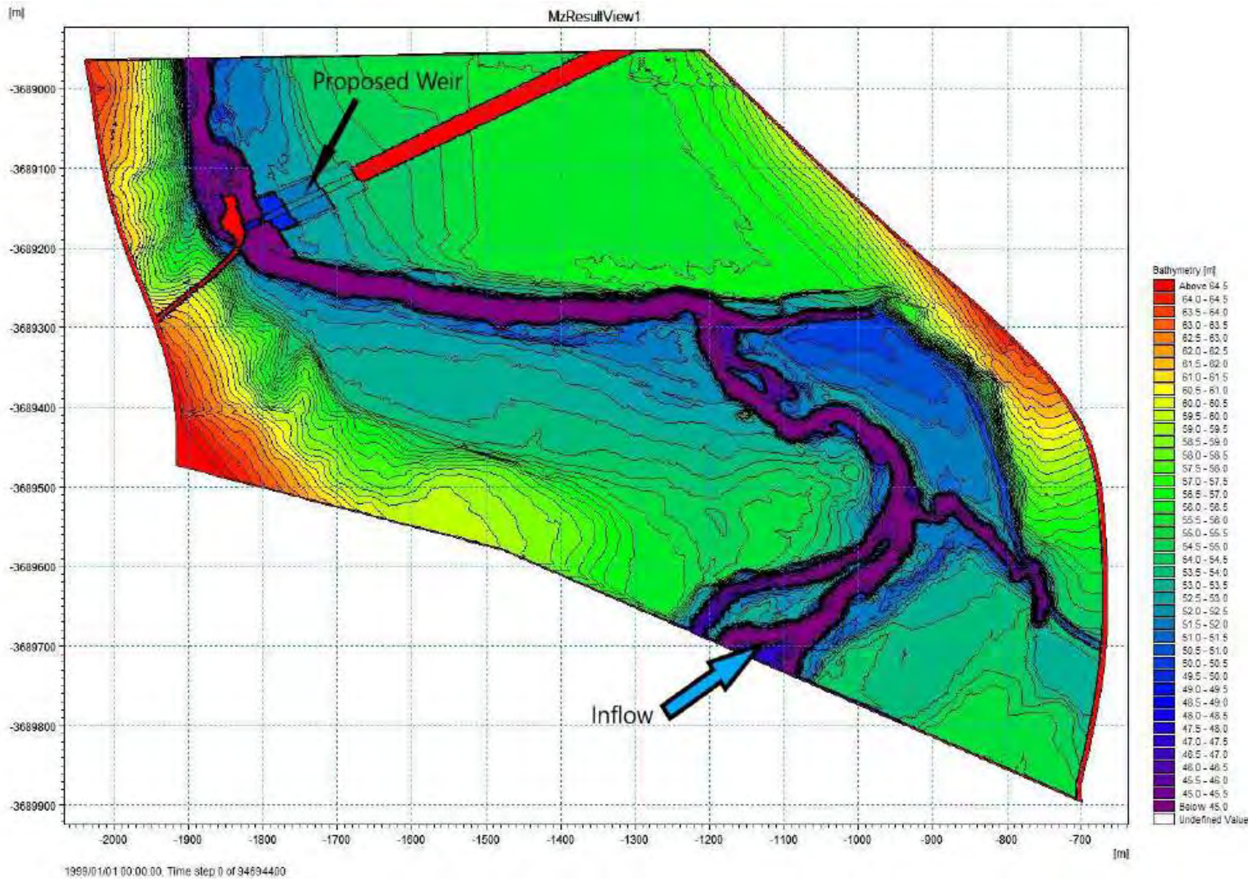


Figure 5.2-1: Initial bathymetry based on the 2012 survey data used in the Mike 21C model (masl) with the proposed abstraction works and weir added

The hydraulic roughness values of Manning n = 0.045 was used for the sand bedded main channel and n = 0.06 was used for the floodplains in the model. Bedrock geotechnical data (DWS, 2012) received was added in the model in order to get the profile of the bedrock elevation as well as the erodible bed thickness in the numerical model. The initial bed 20 m upstream and 20 m downstream of the weir was excavated (bed lowered) to 0.5 m below the respective weir crest levels because the ground levels are higher than the weir in places . The proposed abstraction works and a weir was set up with a high flank wall (70 masl) in the numerical model (final design top of wall is significantly lower). Figure 5.2-2 shows the cross-section of the bedrock elevation at the proposed weir site.

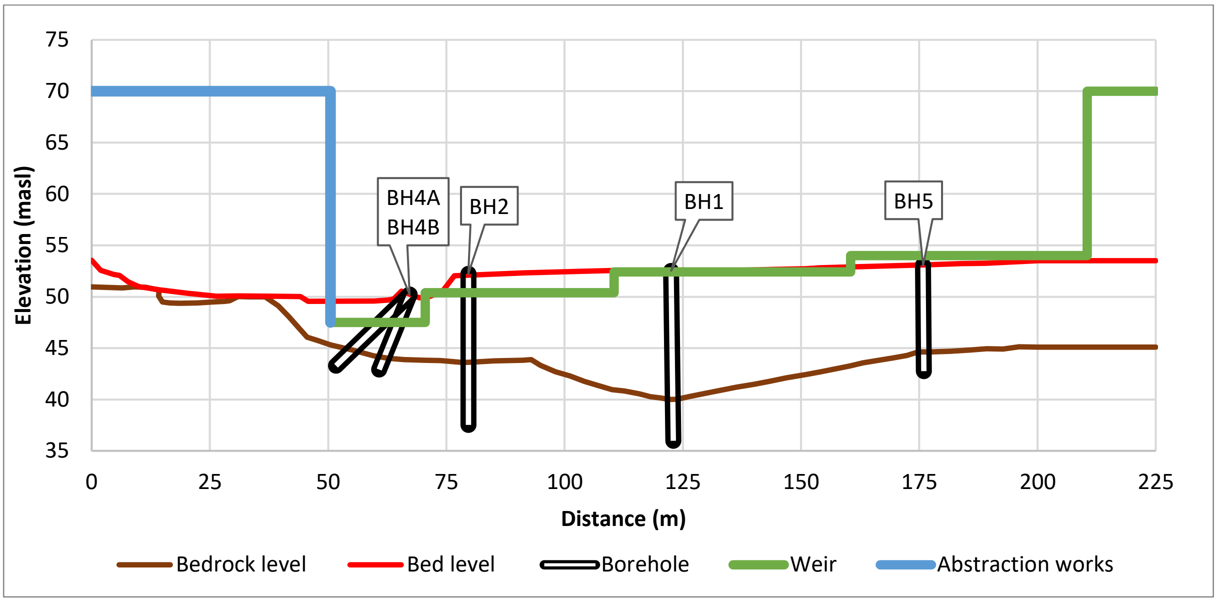


Figure 5.2-2: Cross- section of the bedrock levels at the proposed weir site viewed looking downstream

5.2.3 Hydrodynamic modelling scenarios and boundary conditions

Constant inflows ranging from 5 m³/s to Q200cc=1808 m³/s were first simulated without bed change (no moving bed), and later a 10 year flood hydrograph was routed in the model with sediment transport and movable bed. The simulated bed levels obtained at the end of 10 year flood was then used as the bathymetry for the simulation of above-mentioned constant flows. The downstream boundary water levels are specified based on normal flow depth calculation of a stage - discharge relationship. (Figure 5.2-3).

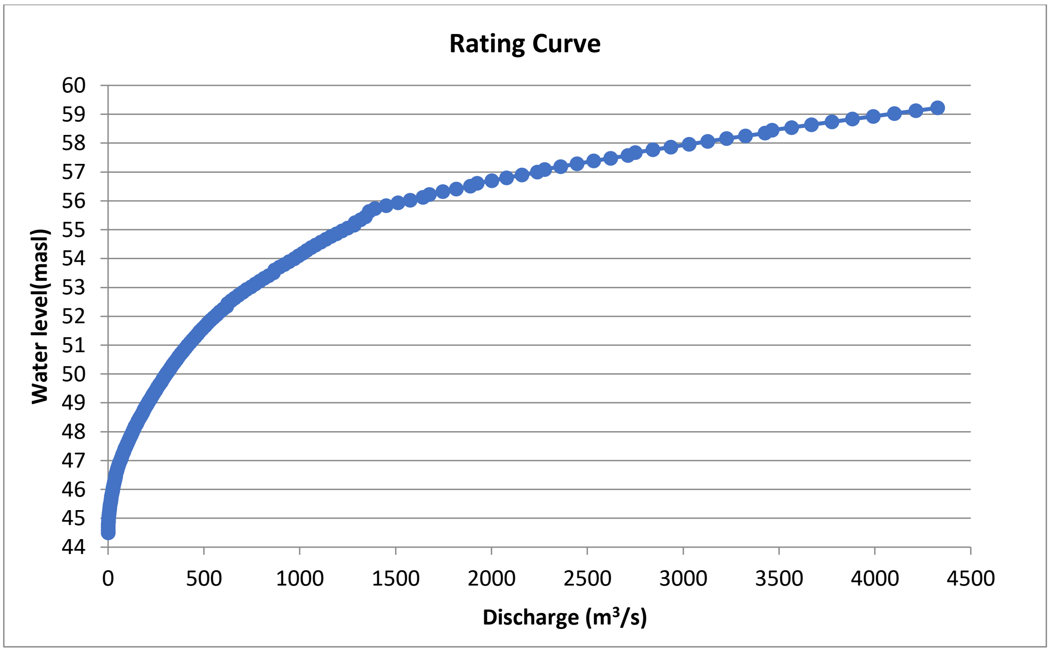


Figure 5.2-3: Discharge-water level relationship at the downstream boundary of the 2D numerical model based on 2012 survey data (Later revised based on 2021 survey data)

Table 5.2-1 gives the constant discharges and the corresponding water levels used at the downstream boundary. Figure 5.2-4 shows the 10 year flood hydrograph and Figure 5.2-5 the corresponding 10 year flood water level time series at the downstream boundary.

Table 5.2-1: Low flow or flood peak discharges and corresponding water levels

Annual recurrence interval flood	Discharge/Flood peak (m^3/s)	Water level (masl)
<Q1cc	5	45.16
<Q1cc	10	45.43
<Q1cc	25	46.02
<Q1cc	50	46.74
Q1cc	100	47.6
Q2cc	210	49.01
Q5cc	424	51.06
Q10cc	613	52.32
Q20cc	830	53.38
Q50cc	1169	54.78
Q100cc	1468	55.86
Q200cc	1808	56.40

Note: “cc” means future climate change impact included

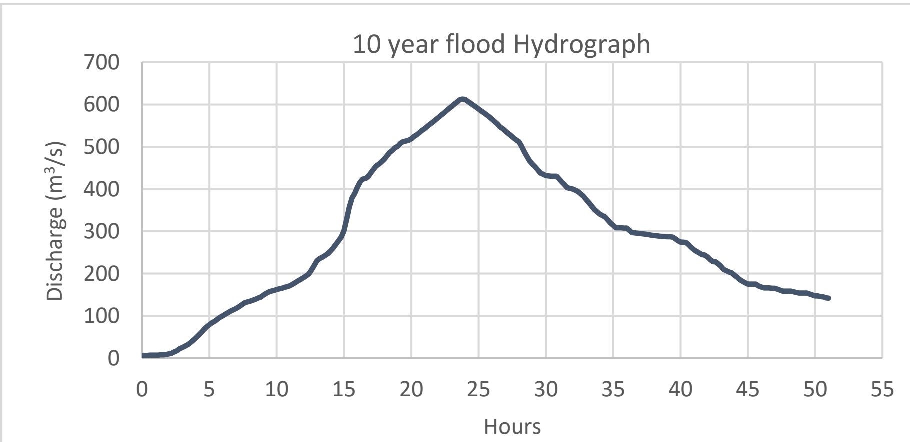


Figure 5.2-4: 10 year flood hydrograph

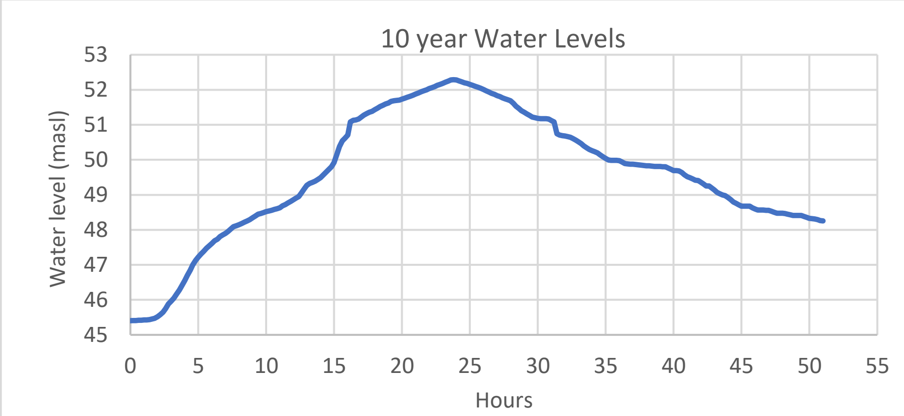


Figure 5.2-5: Downstream open boundary water levels during 10 year flood

5.3 Numerical modelling based on the new 2021 underwater and topographical survey

5.2.4 Simulation results

a) Simulated water levels for different flows with the initial bathymetry based on the 2012 survey data and with the abstraction works and weir constructed

Figure 5.2-6 shows the simulated water level profiles along the main channel of the Berg River for different inflows with the initial bathymetry without a movable bed. At 100 m^3/s (Q1cc), the tailwater level reaches the low notch level, while the low notch of the Crump weir seems to be almost fully submerged at a discharge of 424 m^3/s, the 5 year flood.

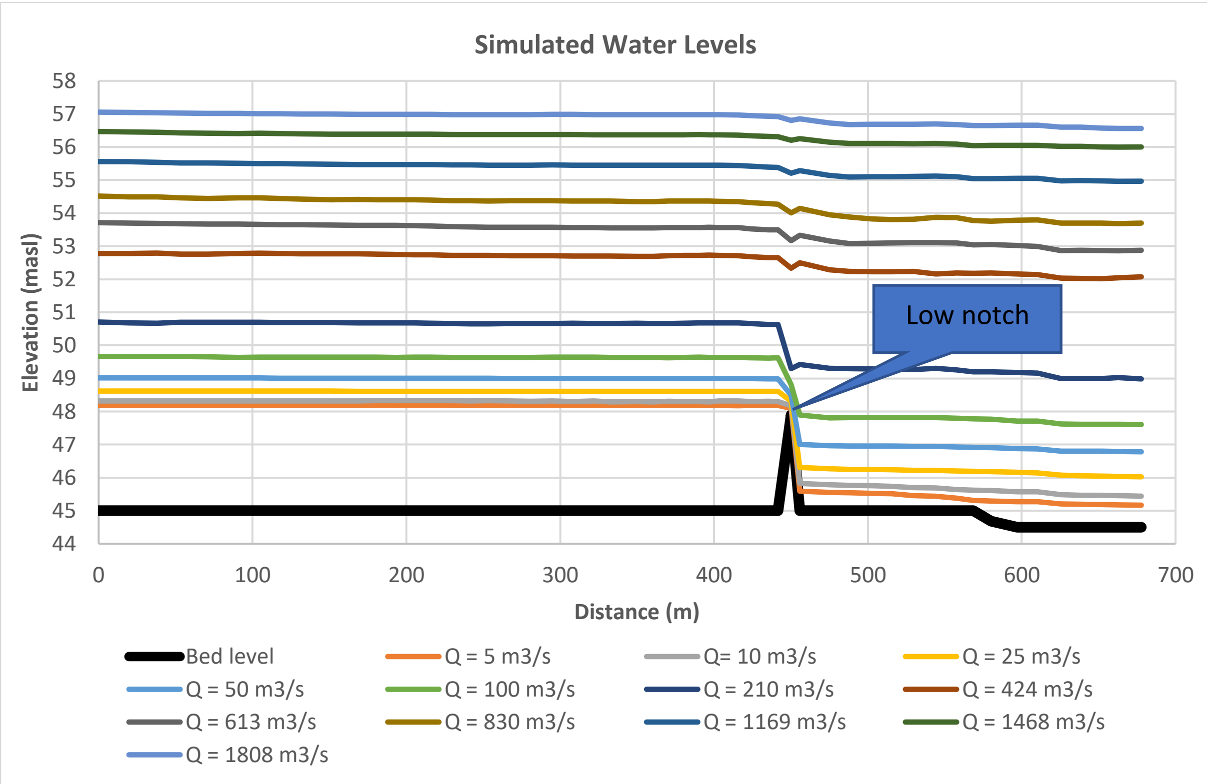


Figure 5.2-6: Simulated water levels along the main channel of the Berg River with the initially proposed weir and with the initial bathymetry without a movable bed

b) Sediment transport simulation results for the 10 year flood hydrograph (based on the 2012 survey data and with the abstraction works and weir constructed)

Figures 5.2-7 and 5.2-8 show the simulated bed change at the end of the 10 year flood and the simulated bed level at the end of the 10 year flood. Figure 5.2-9 shows the simulated bed levels at the weir following the 10 year flood.

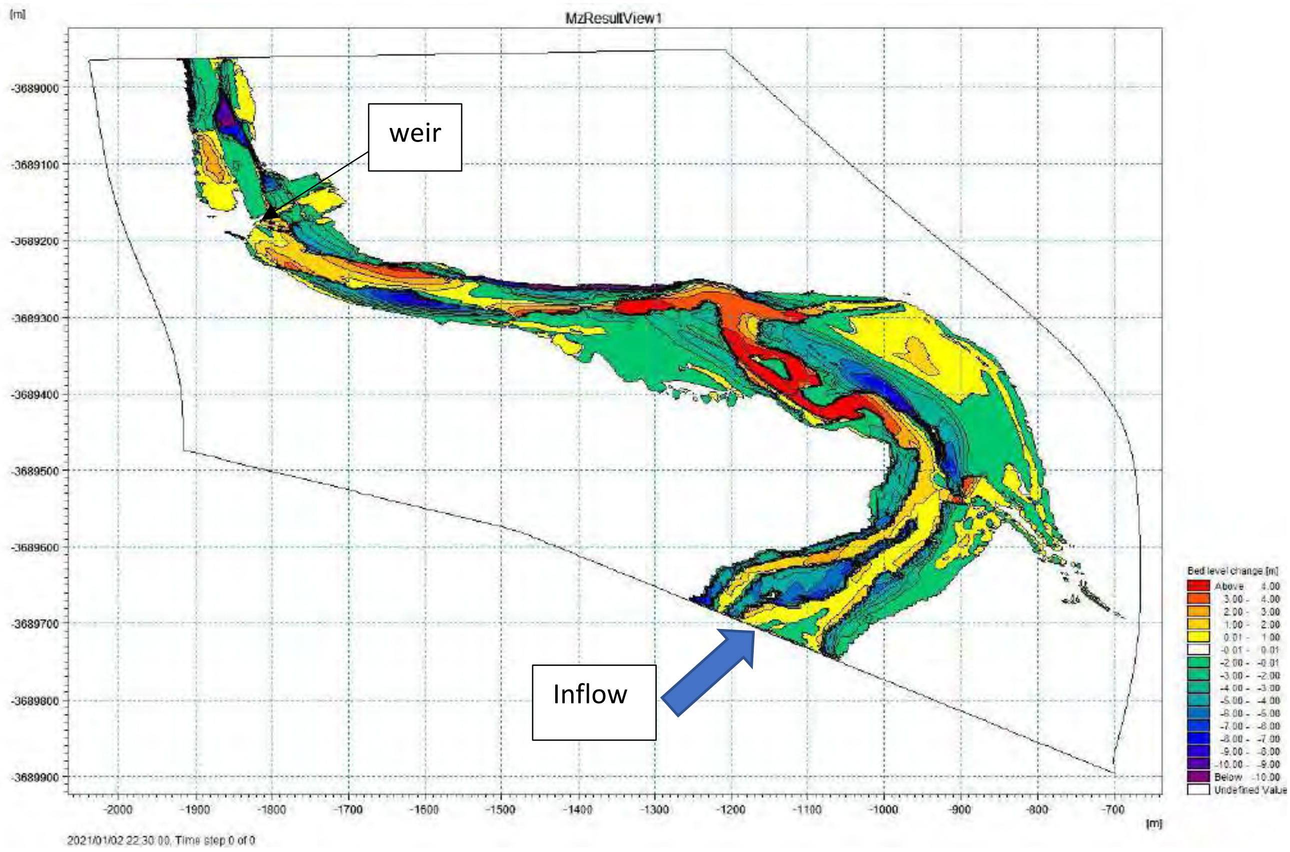


Figure 5.2-7: Simulated bed level change at the end of the 10 year flood hydrograph

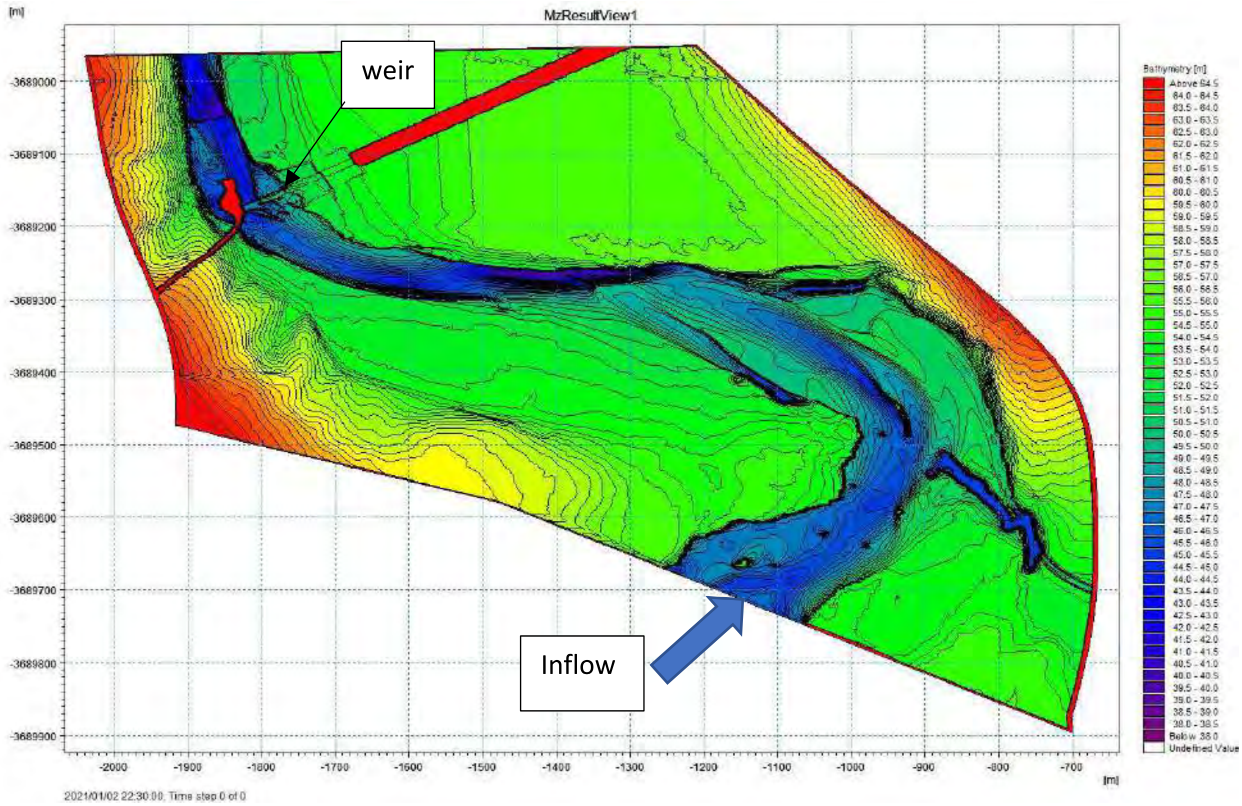


Figure 5.2-8: Simulated bed levels at the end of the 10 year flood (masl)

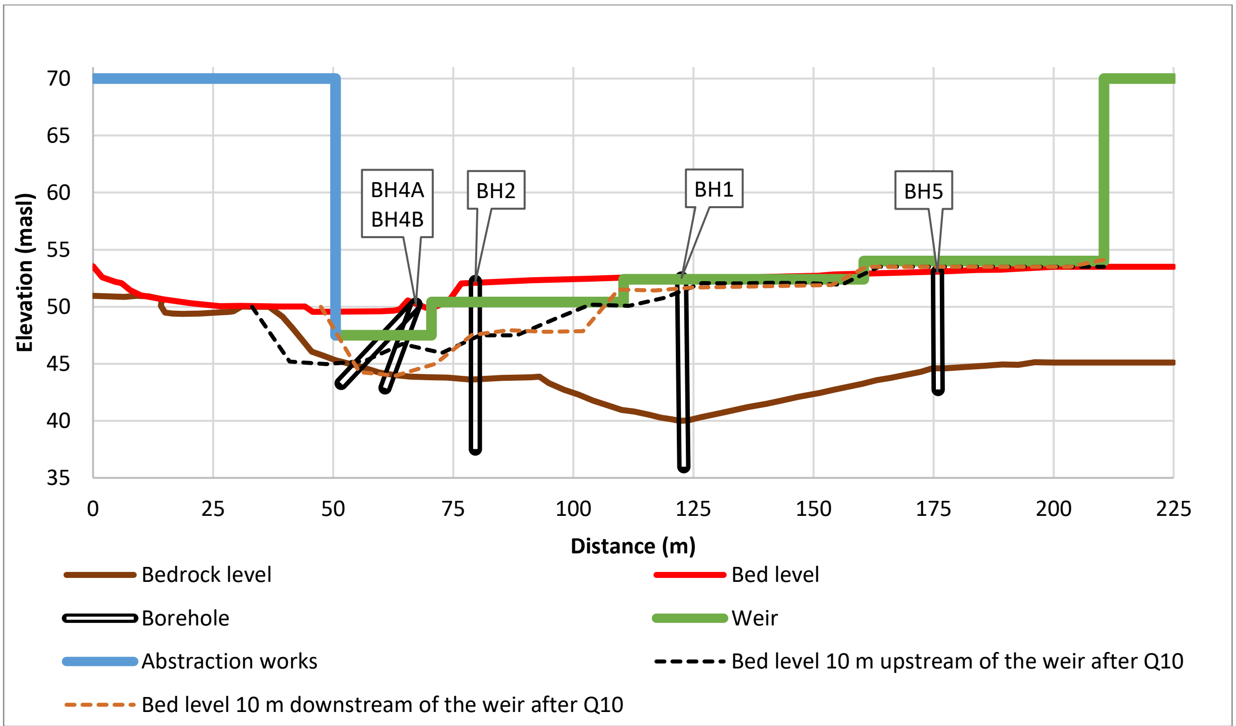


Figure 5.2-9: Cross-section of the bedrock levels at the proposed weir site viewed looking downstream with the simulated bed levels

c) Simulated water levels for different flows with the 10 year flood simulated bed level used as bathymetry (based on the 2012 survey data and with the abstraction works and weir constructed)

In this simulation, the simulated bed level at the end of the 10 year flood hydrograph was used as bathymetry in the model. Figure 5.2-10 shows the simulated water levels for different inflows. There is a slight drop in the local tailwater levels downstream of the weir under small flood conditions due to the scour caused by the Q10 flood. At a river discharge of 10 m^3/s the drop in tailwater level is about 0.3 m, which is important for the canoe chute and fishway designs. At low river flows the river is only about 1 m deep downstream of the weir with the bed level scoured to the bedrock level where the fishway-canoe chute is proposed at the low notch at the left bank; this was later considered in the design of the fishway-canoe chute and testing in the physical model of the chute.

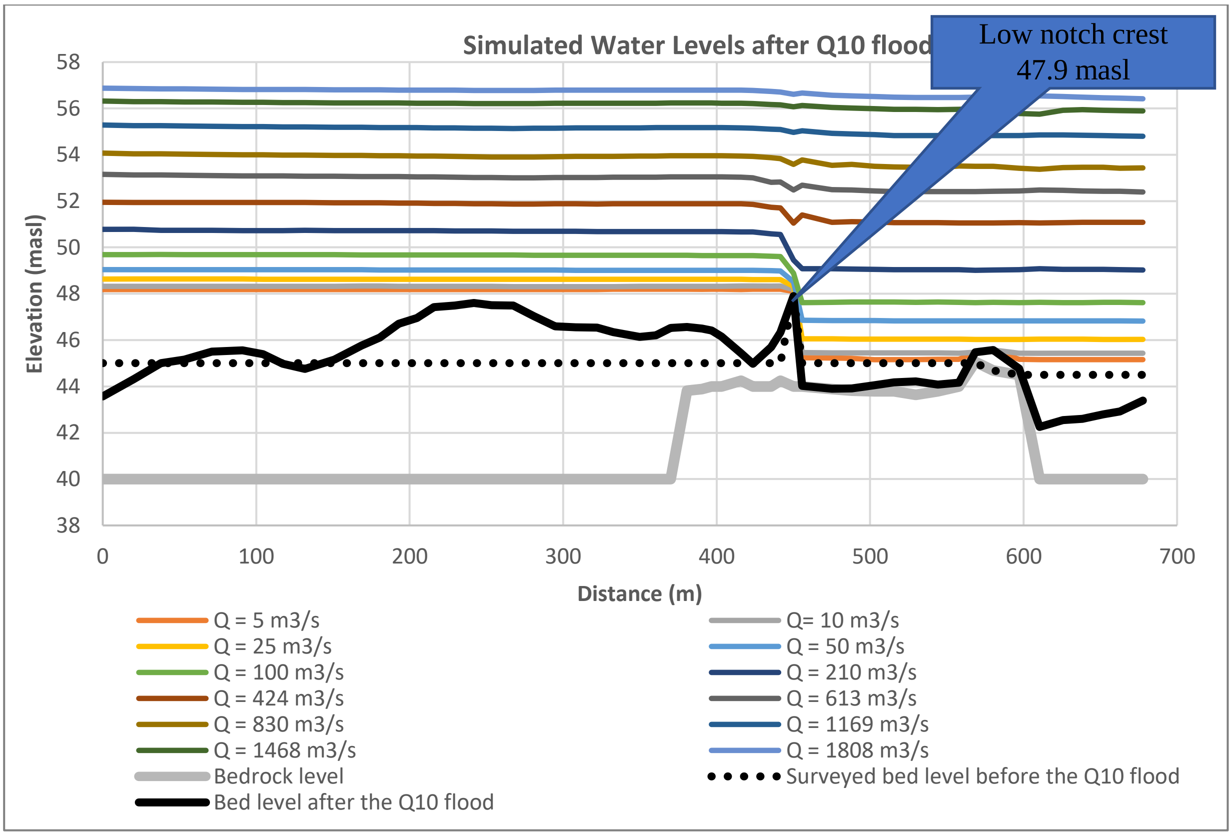


Figure 5.2-10: Simulated water levels along the main channel of the Berg River following the 10 year flood

The simulated flow depths and flow velocities of the above scenarios are shown in the figures in **Appendix D1**.

5.3.1 Hydrodynamic model setup based on the new 2021 survey data

The Mike 21C model was set up based on the new 2021 underwater and topographical survey data. The model bathymetry was generated using a curvilinear mesh with cell sizes of about 5 m in the flow direction and 5 m in the direction perpendicular to the flow. The Berg River reach included in the model is 3.7 km long along the main channel. Figure 5.3-1 shows the model bathymetry. Table 5.3.1 highlights different parameters used in the model setup.

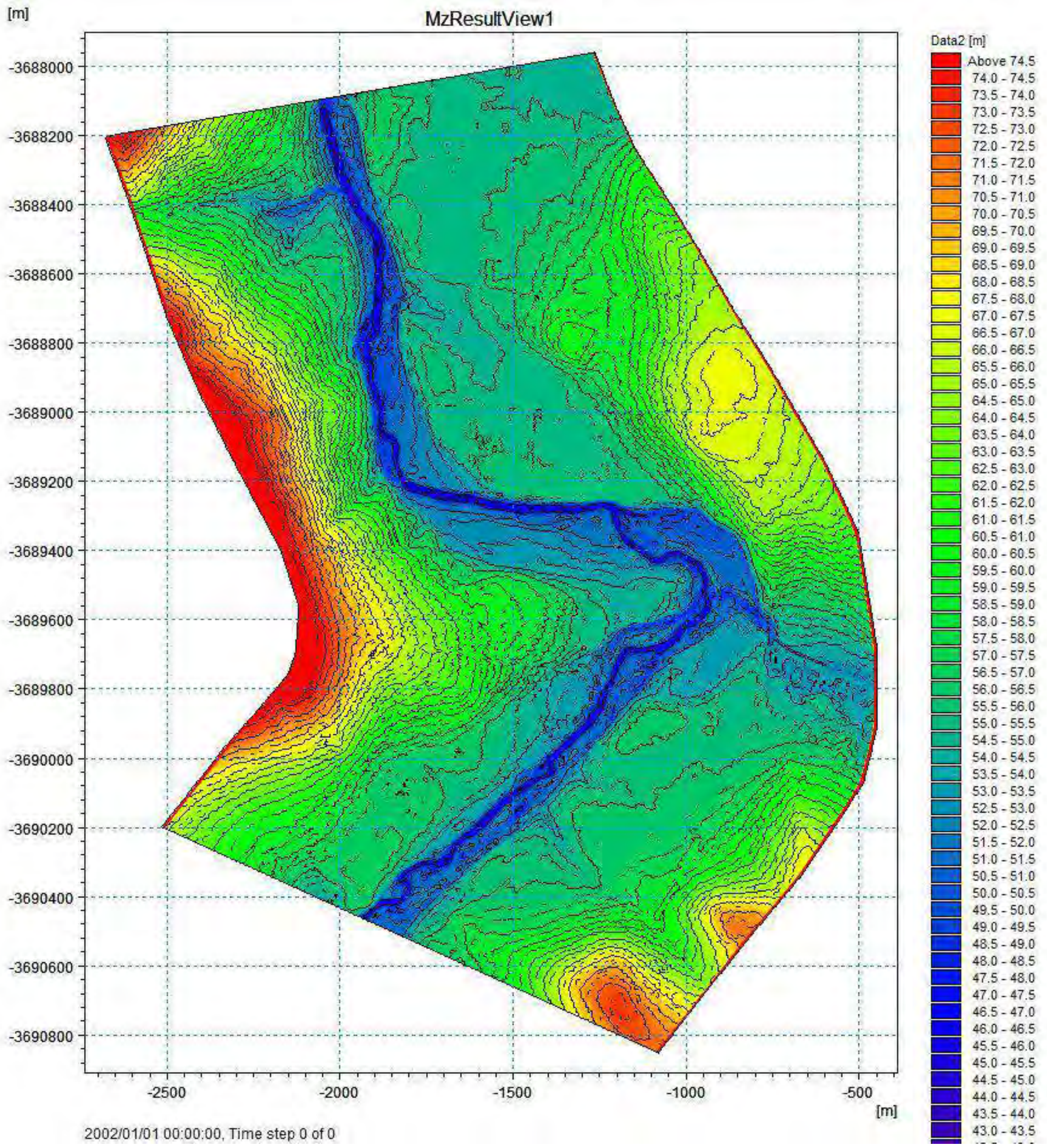


Figure 5.3-1: Bathymetry used in the Mike 21C model (masl)

Table 5.3.1: Hydrodynamic model setup parameters

Parameter and unit	Value
Curvilinear mesh size (m)	5 by 5
Bed roughness – Manning n ($s/m^{0.33}$)	0.045 (main channel); 0.06 (floodplain)
Simulation time step (s)	0.4
Eddy viscosity (m^2/s)	0.5
Integration type	Fully dynamic
Morphological time step	2
Sediments fraction size (fraction percentage)	0.83 mm (35%) 0.19 mm (35%) 0.063 mm (30%)
Sediment transport theory	Engelund & Hansen
Sediment specific gravity	2.56

The hydraulic roughness values of Manning n = 0.045 was used for the sand bedded main channel and n = 0.06 was used for the floodplains in the model. Bedrock geotechnical data (DWS, 2012) received was added in the model in order to get the profile of the bedrock elevation as well as the erodible bed thickness in the numerical model. Figure 5.2-2 in the previous section, shows the cross section of the bedrock elevation at the proposed weir site.

5.3.2 Hydrodynamic modelling scenarios and boundary conditions

The 2D hydrodynamic model was subsequently used to help optimize the hydraulic design of the weir and abstraction works faster as well as to design the physical model boundaries, in a hybrid approach with the 3D physical model. It is important to note that the numerical cannot replace the 3D physical model tests and results and the latter is more accurate and reliable than the numerical model.

The following hydraulic design scenarios were simulated (layouts of the various options are depicted in Section 9):

- Current scenario with no weir and no abstraction works.
- Option A: Proposed weir and abstraction works with berms (flood levees) along the river (modified version of the feasibility design shown in Section 5.2) (refer Figure 9-9).
- Option B: Proposed weir and abstraction works located 50 m downstream of Option A, similar in design to Option A, with longer berms along the river. (refer to Figure 9-10). These berms act as flood levees to protect the agricultural land against floods up to the Q50cc flood, and spill during large floods, but more importantly it guides the flood flow around the river bend to ensure self-scouring of the intake of the abstraction works.
- Option C: This option has a similar chainage along the river as Option B, but is located more to the left bank side on the main channel. (refer to Figure 9-11).
- Option B2: This option corresponds to the modified version of Option B, but keeping the feasibility weir design layout (without levees) and adding a longer left flank wall and a central guide wall. This option was selected as the best design considering the approved

Environmental Impact Assessment (EIA) based on the feasibility study layout with a weir and a right bank side berm. The Options B and B2 weir location is within a few meters where the feasibility study carried out their weir geotechnical borehole drilling (DWS, 2012).

For all the scenarios, the Q50cc year flood peak (1169 m^3/s) and Q100cc year flood peak (1468 m^3/s) were simulated as only hydrodynamic (no moving bed) and constant inflows. From these simulation results, the best weir design was selected (Option B2) and additional simulations including sediment transport and movable bed were conducted namely: routing of Q50cc and Q100cc hydrographs, 15 years long term simulation and 15 years long term plus 50 year and 100 year hydrograph simulations.

The downstream boundary water levels are specified based on normal flow depth calculation of a stage - discharge relationship and was also calibrated against low flow water levels obtained from the 2021 topographical survey (Figure 5.3-2). For floods above 150 m^3/s the tailwater level from the 2021 survey is about 1 m higher than the tailwater level predicted in by using the 2012 survey data. The 2021 survey data extends further downstream of the weir site and with the better underwater survey data added as well as with water level calibration, the 2021 discharge- water level tailwater relationship of Figure 5.3-2 is more reliable than the 2012 relationship.

Figure 5.3-2: Discharge-water level relationship at the downstream boundary of the numerical model compared with 2012 rating curve

Table 5.3-2 gives the constant discharges and the corresponding water levels used at the downstream boundary. Figure 5.3-3 shows the Q50cc year flood and Q100cc year flood hydrograph and Figure 5.3-4 the corresponding Q50cc year flood and Q100cc year flood water level time series at the downstream boundary.

Table 5.3-2: Low flow or flood peak discharges and corresponding water levels (2021 study)

Annual recurrence interval flood	Discharge/Flood peak (m^3/s)	Water level (masl)
<Q1cc	5	46.66
<Q1cc	10	47.09
<Q1cc	25	47.84
<Q1cc	50	48.71
Q1cc	100	49.98
Q2cc*	210	52.50
Q5cc	424	53.66
Q10cc	613	54.45
Q20cc	830	55.15
Q50cc	1169	55.86
Q100cc	1468	56.28
Q200cc	1808	56.69

Note: “cc” means future climate change impact included

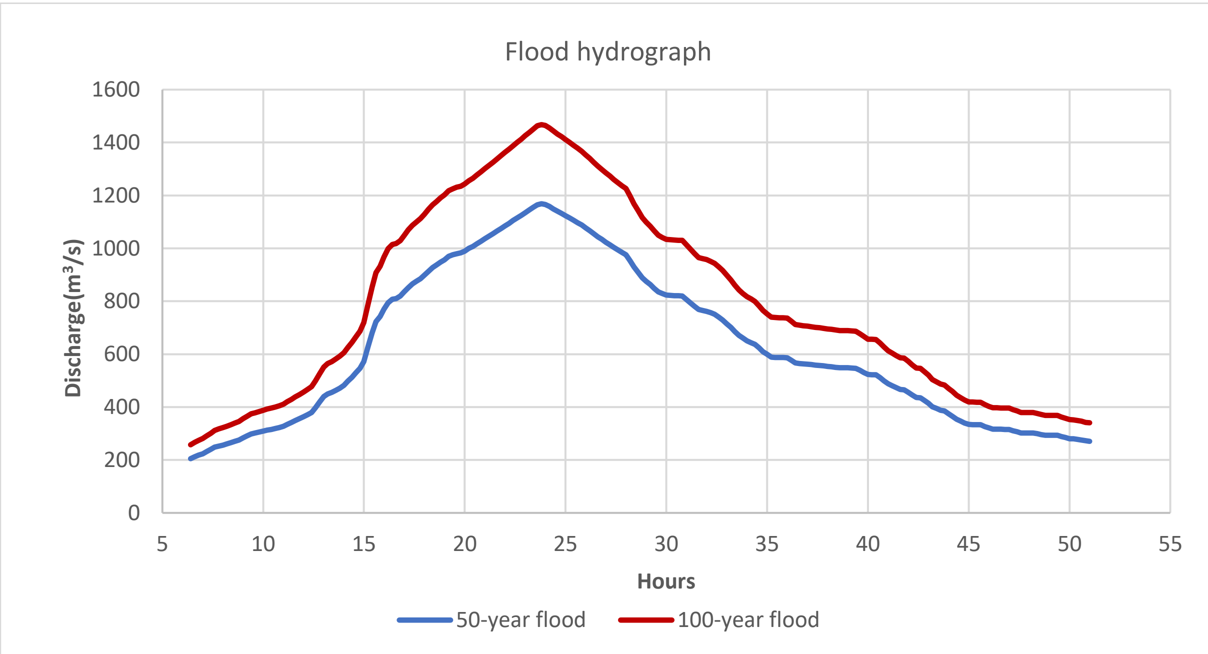


Figure 5.3-3: Q50cc year and Q100cc year flood hydrograph (2021 study)

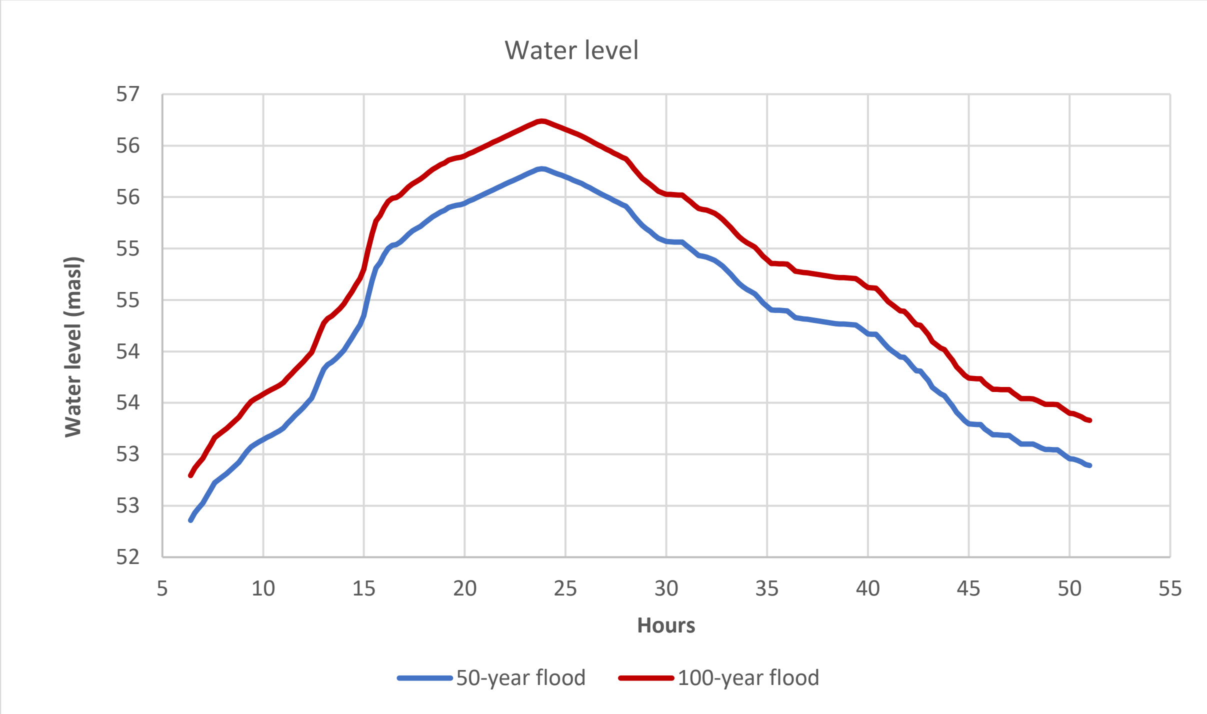


Figure 5.3-4: Downstream open boundary water levels during Q50cc year and Q100cc year flood (2021 study)

5.3.3 Simulation results

a) Current scenario with no weir and no abstraction works

Figures 5.3-5 to 5.3-8 show the simulated flow velocity and simulated water depth for the Q50cc and Q100cc flood peaks.

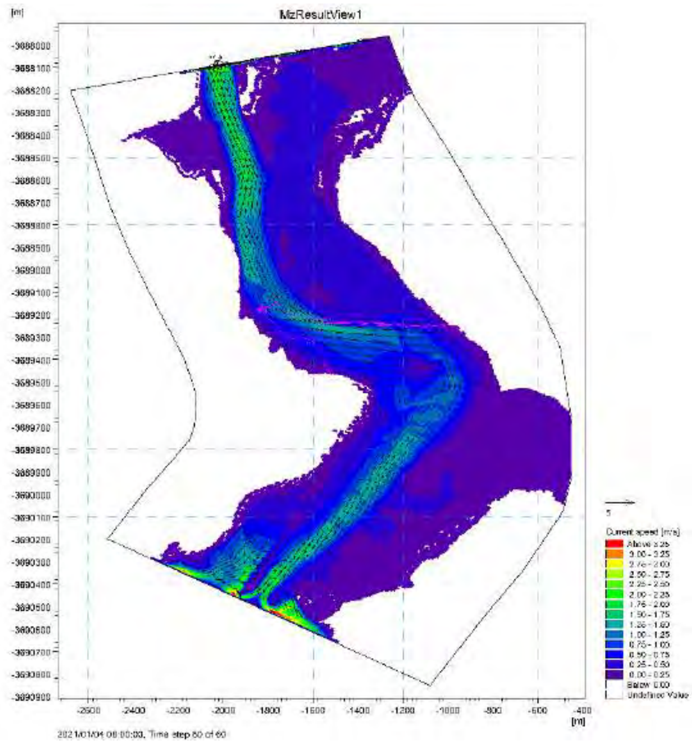


Figure 5.3-5: Flow velocity and velocity vectors for the Q50cc flood peak

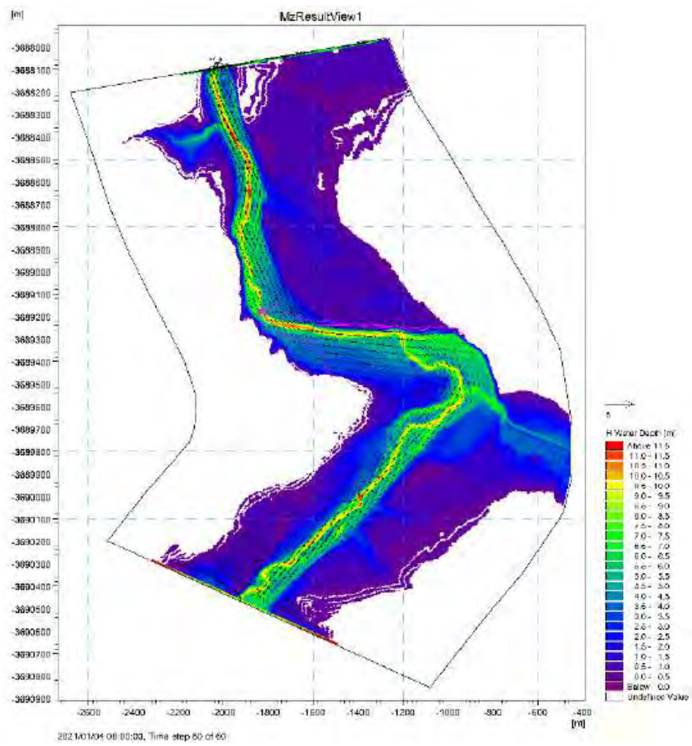


Figure 5.3-6: Water depth and velocity vectors for Q50cc flood peak

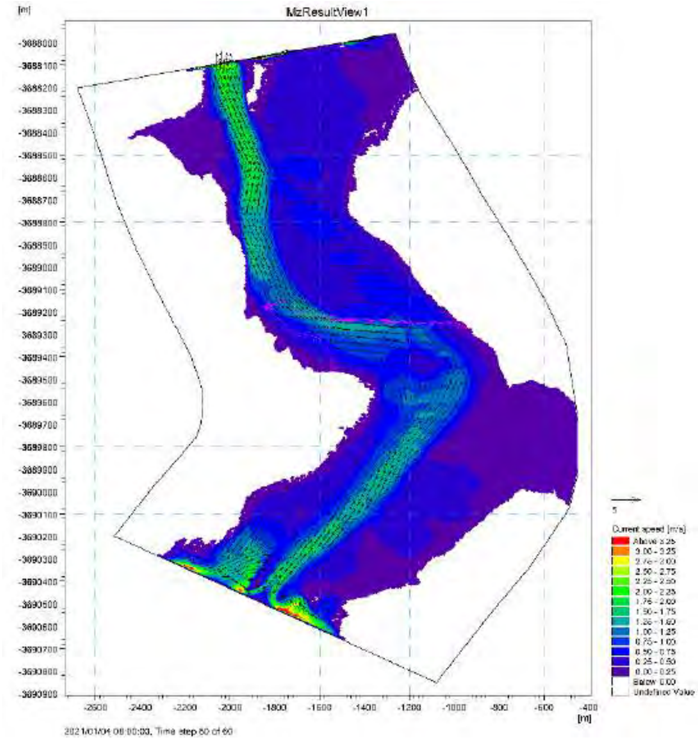


Figure 5.3-7: Flow velocity and velocity vectors for the Q100cc flood peak

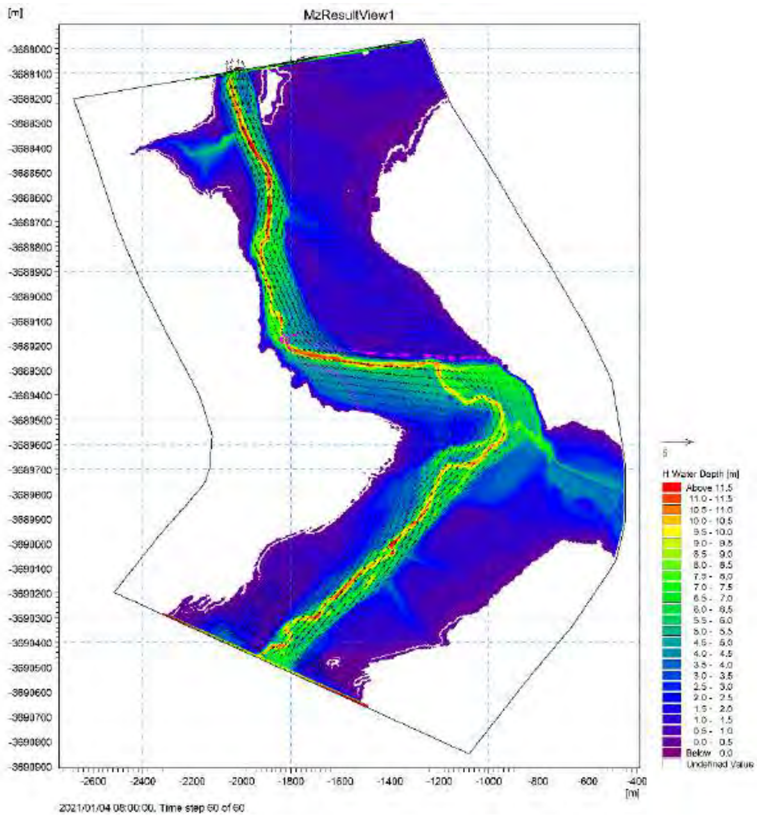


Figure 5.3-8: Water depth and velocity vectors for the Q100cc flood peak

b) Option A scenario: weir and abstraction works with berms (levees) along the river

Figures 5.3-9 to 5.3-12 show the simulated flow velocity and simulated water depth for Q50cc year and Q100cc year flood peak.

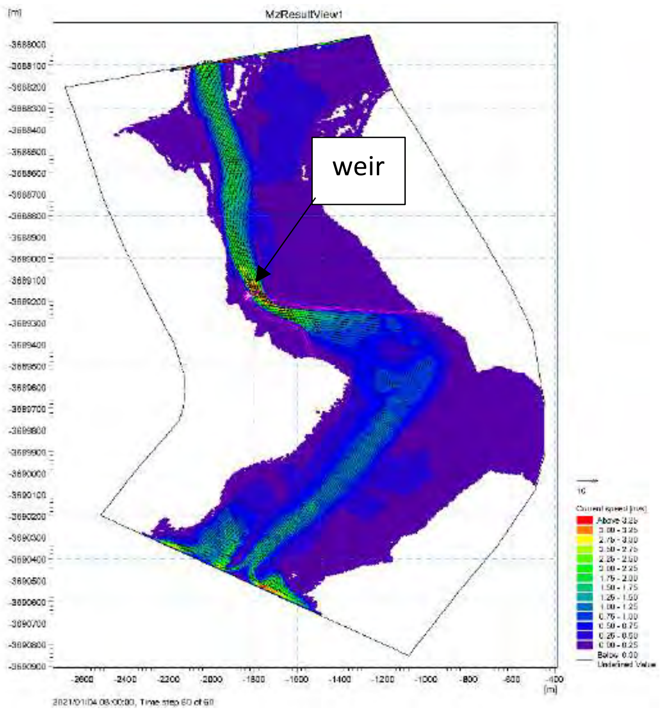


Figure 5.3-9: Flow velocity and velocity vectors for the Q50cc flood peak (Option A)

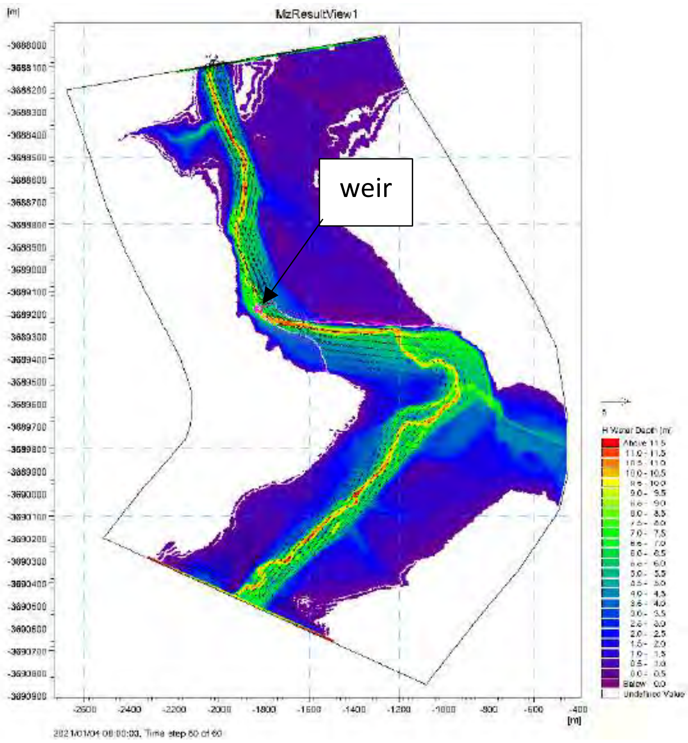


Figure 5.3-10: Water depth and velocity vectors for the Q50cc flood peak (Option A)

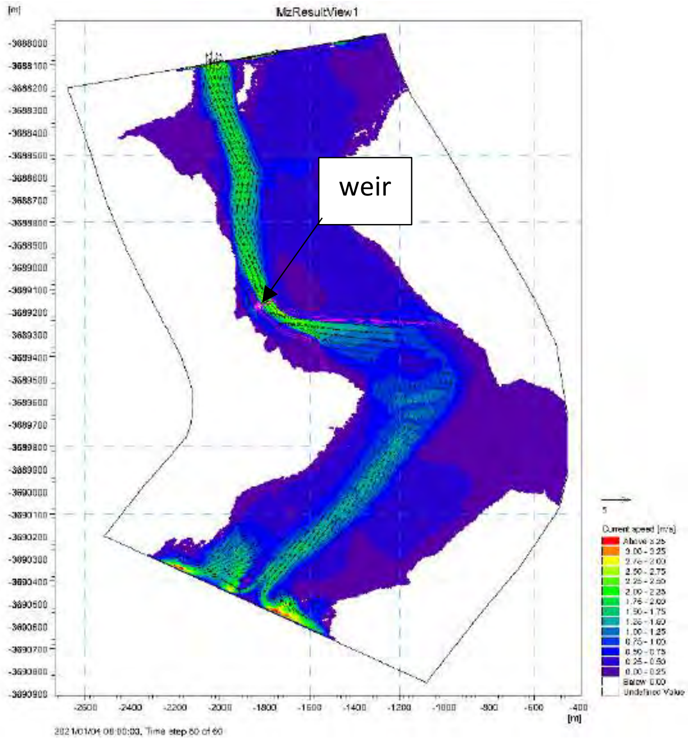


Figure 5.3-11: Flow velocity and velocity vectors for the Q100cc flood peak (Option A)

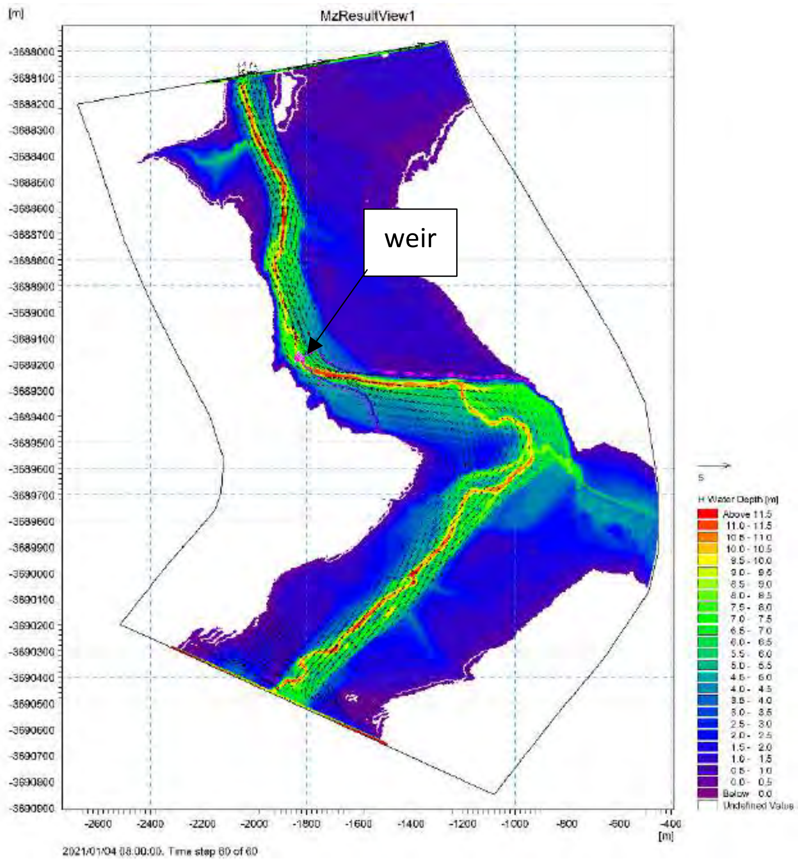


Figure 5.3-12: Water depth and velocity vectors for the Q100cc flood peak (Option A)

Figure 5.3-13 shows the simulated water level profiles along the main channel of the Berg River for different inflows with the initial bathymetry without a movable bed. All ARI flood peaks were simulated with the effects of climate change included.

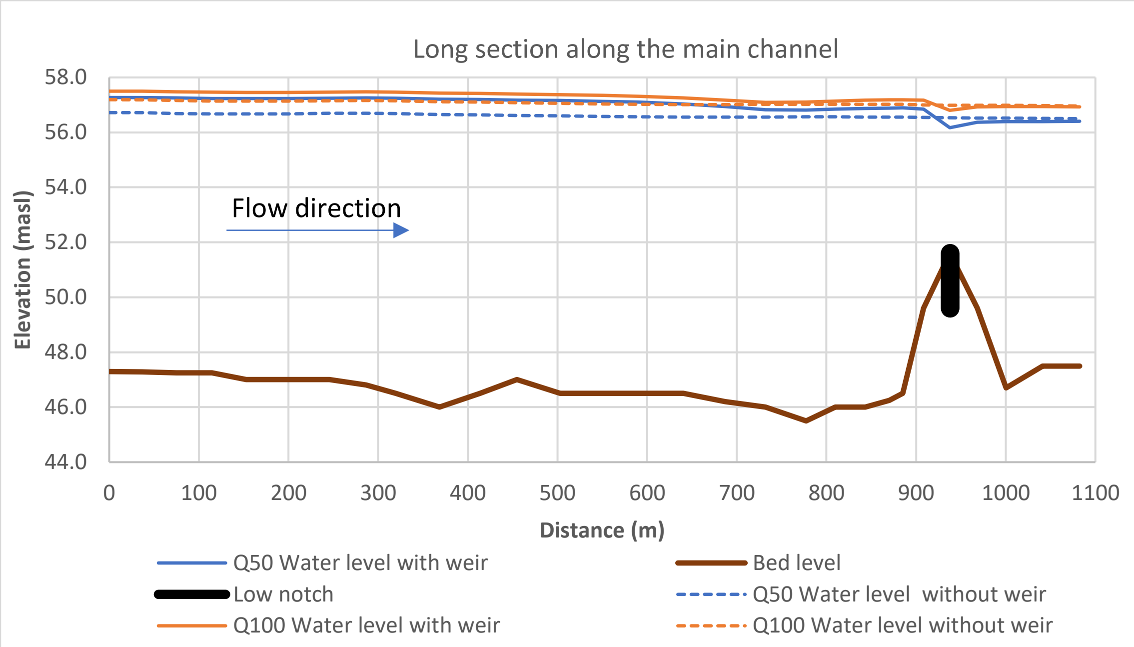


Figure 5.3-13: Simulated water levels along the main channel of the Berg River with the proposed weir and with the initial bathymetry without a movable bed (Option A)

c) Option B scenario: weir and abstraction works located 50 m downstream of Option A

Figures 5.3-14 to 5.3-17 show the simulated flow velocity and simulated water depths for the Q50cc and Q100cc flood peaks.

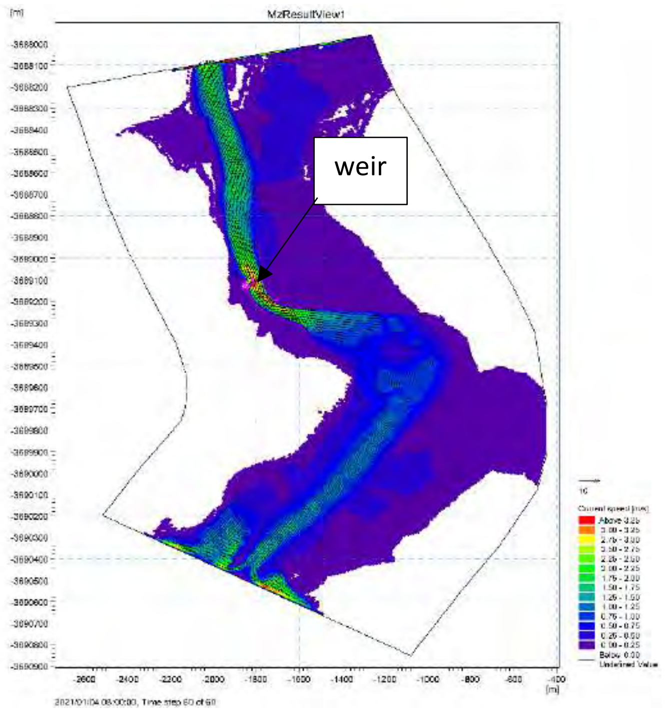


Figure 5.3-14: Flow velocity and velocity vectors for the Q50cc flood peak (Option B)

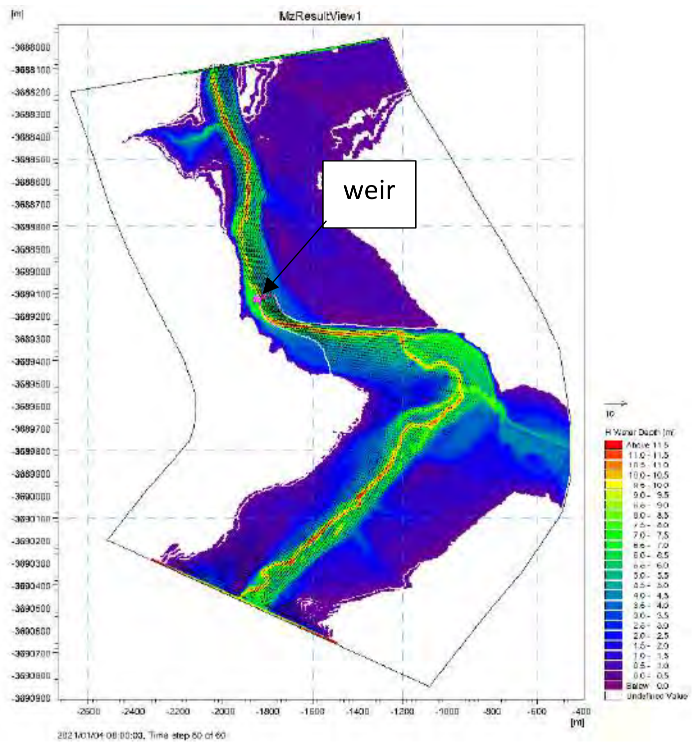


Figure 5.3-15: Water depth and velocity vectors for the Q50cc flood peak (Option B)

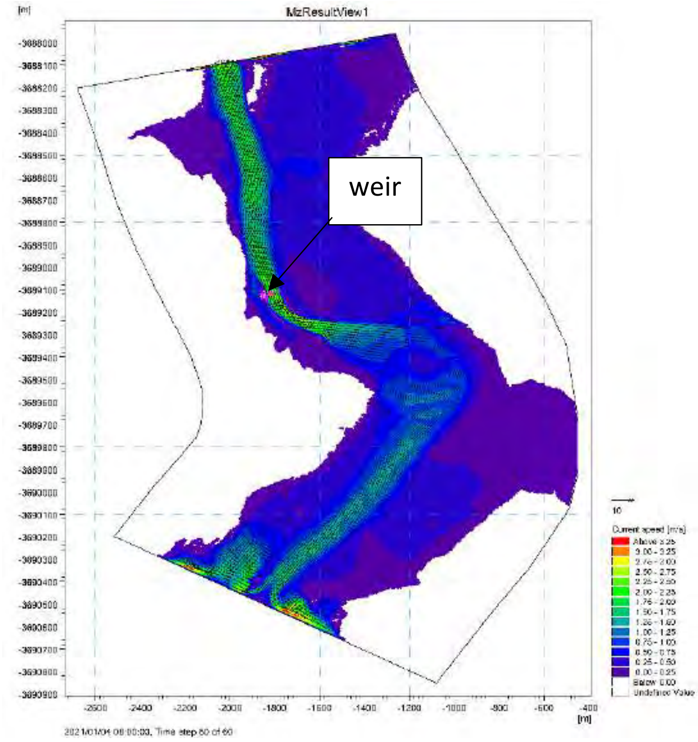


Figure 5.3-16: Flow velocity and velocity vectors for the Q100cc flood peak (Option B)

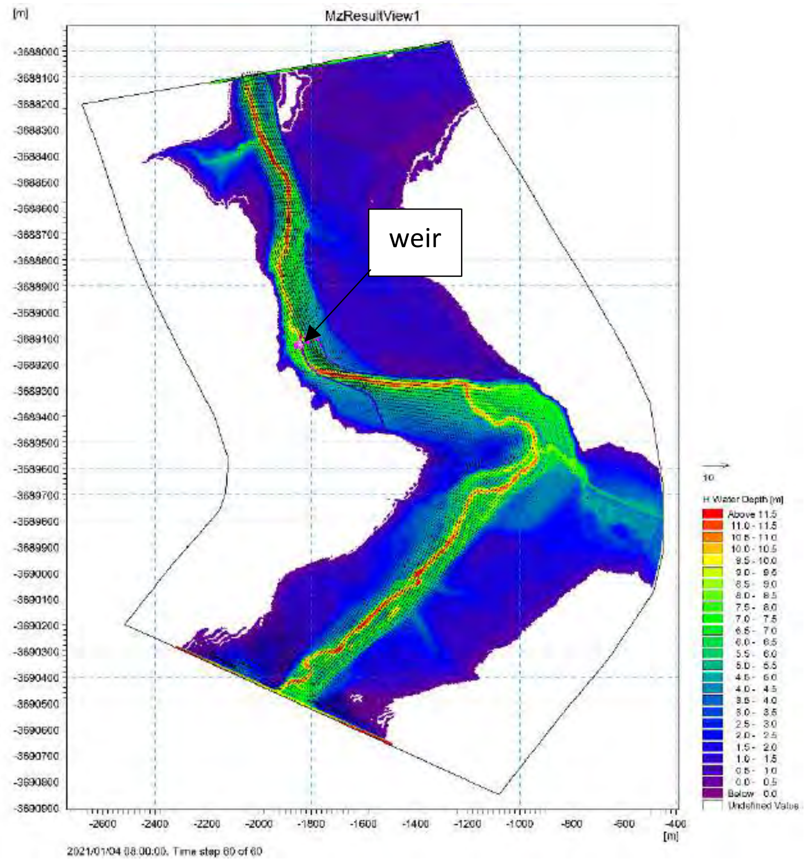


Figure 5.3-17: Water depth and velocity vectors for the Q100cc flood peak (Option B)

Figure 5.3-18 shows the simulated water level profiles along the main channel of the Berg River for different inflows with the initial bathymetry without a movable bed.

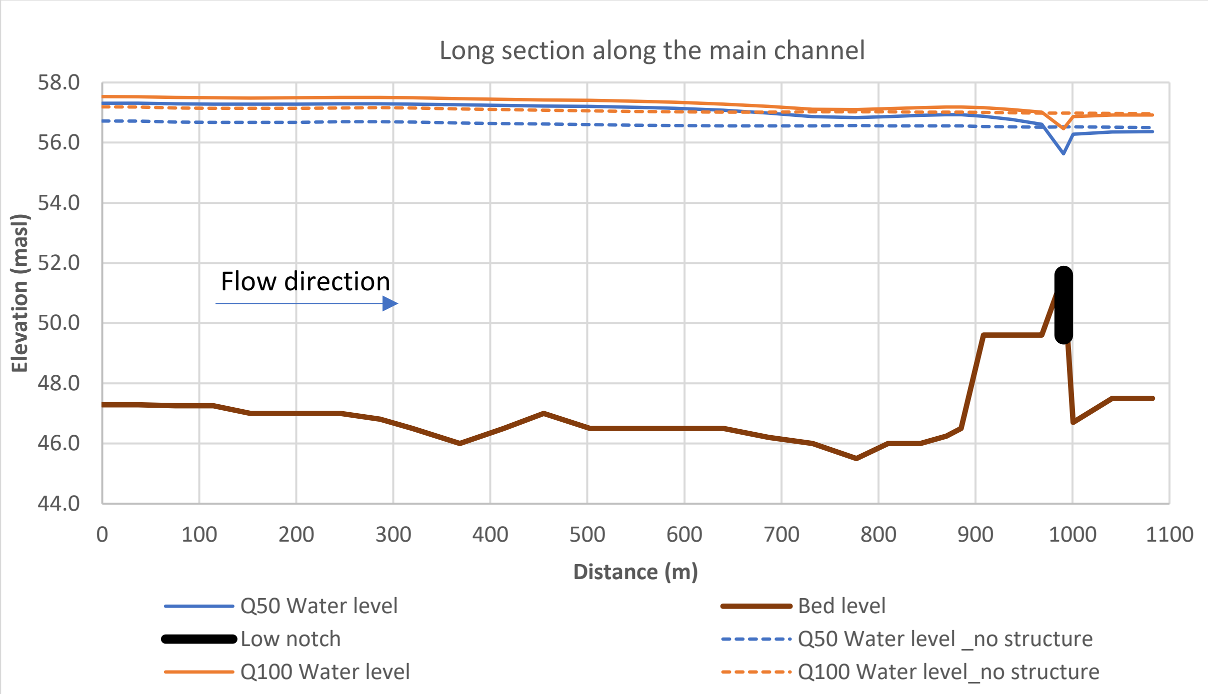


Figure 5.3-18: Simulated water levels along the main channel of the Berg River with the proposed weir and with the initial bathymetry without a movable bed (Option B)

d) Option C scenario: weir and abstraction works located 50 m downstream of Option A but is located more to the left bank side on the main channel

Figures 5.3-19 to 5.3-20 show the simulated flow velocity and simulated water depths for the Q50cc year flood peak.

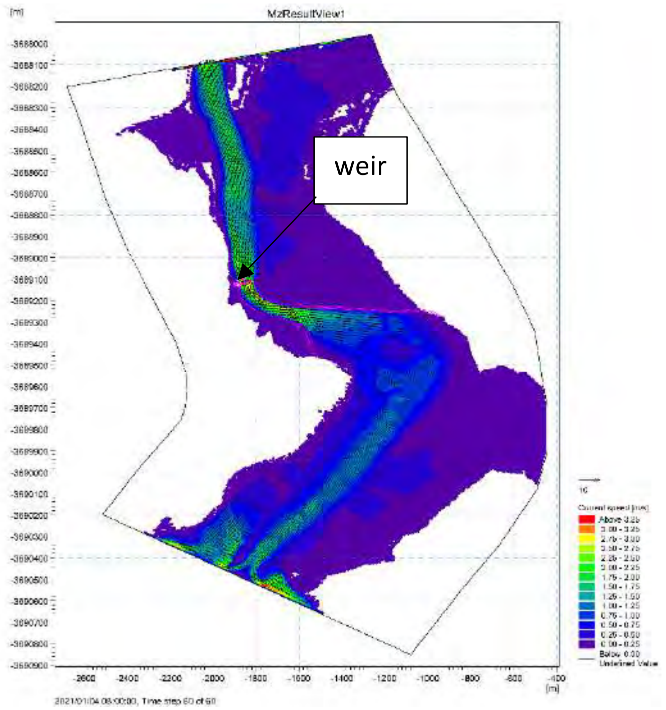


Figure 5.3-19: Flow velocity and velocity vectors for the Q50cc flood peak (Option C)

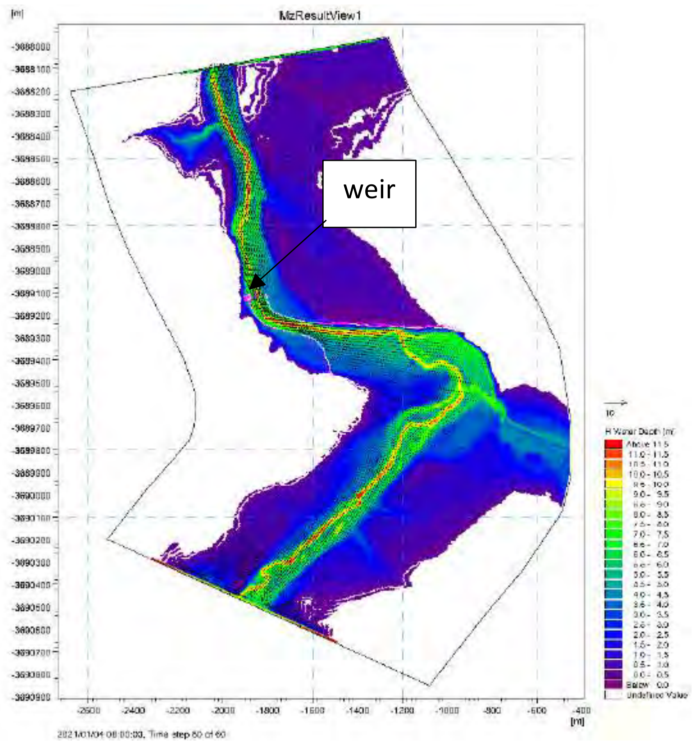


Figure 5.3-20: Water depth and velocity vectors for the Q50cc flood peak (Option C)

5.3.4 Simulation results for the final weir abstraction works design (Option B2)

Option B2 scenario is the modified version of Option B, but keeping the feasibility weir design and chainage along the river and adding a left flank wall and central guide wall.

a) Constant flow simulation results with no moving bed

Figures 5.3-21 to 5.3-26 show the simulated flow velocity and simulated water depth for the Q50cc, Q100cc flood peak and RMFcc.

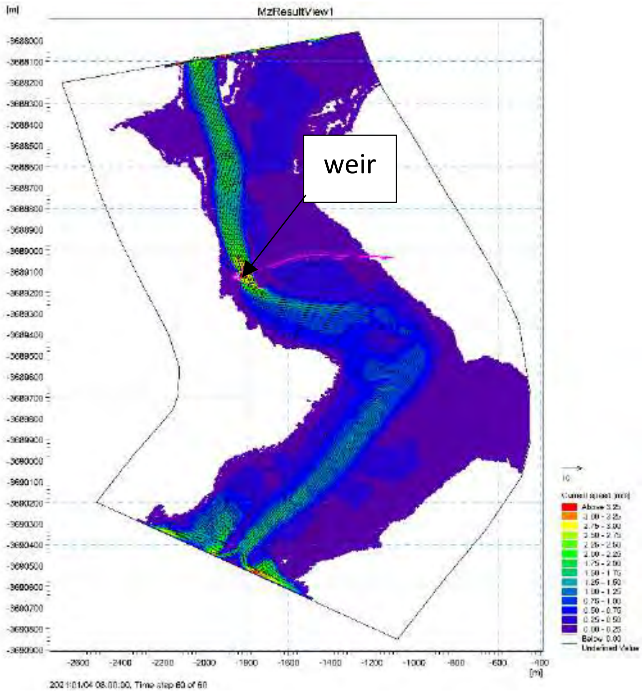


Figure 5.3-21: Flow velocity and velocity vectors for the Q50cc flood peak (Option B2)

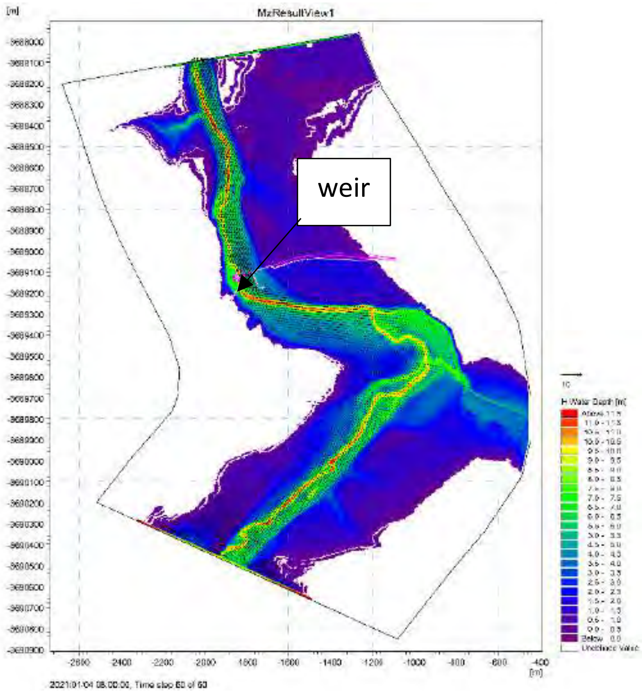


Figure 5.3-22: Water depth and velocity vectors for the Q50cc flood peak (Option B2)

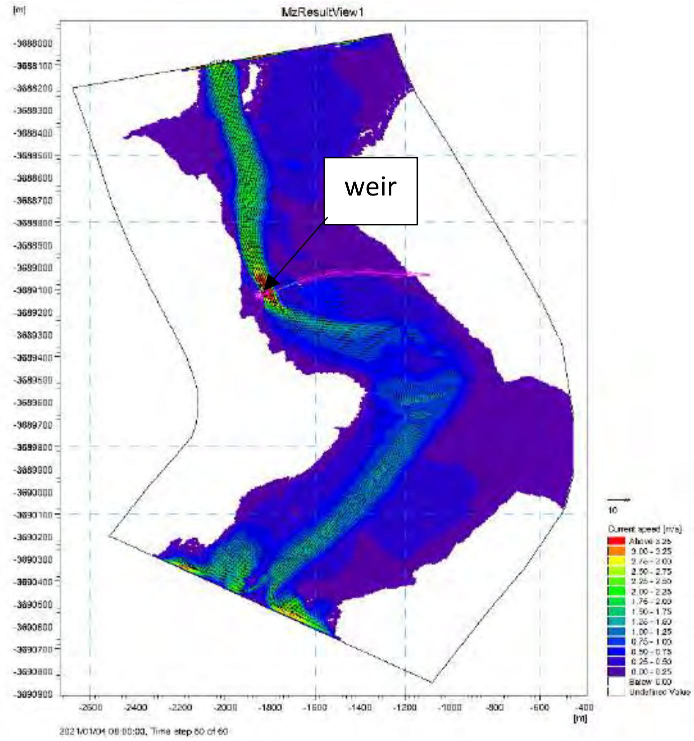


Figure 5.3-23: Flow velocity and velocity vectors for the Q100cc flood peak (Option B2)

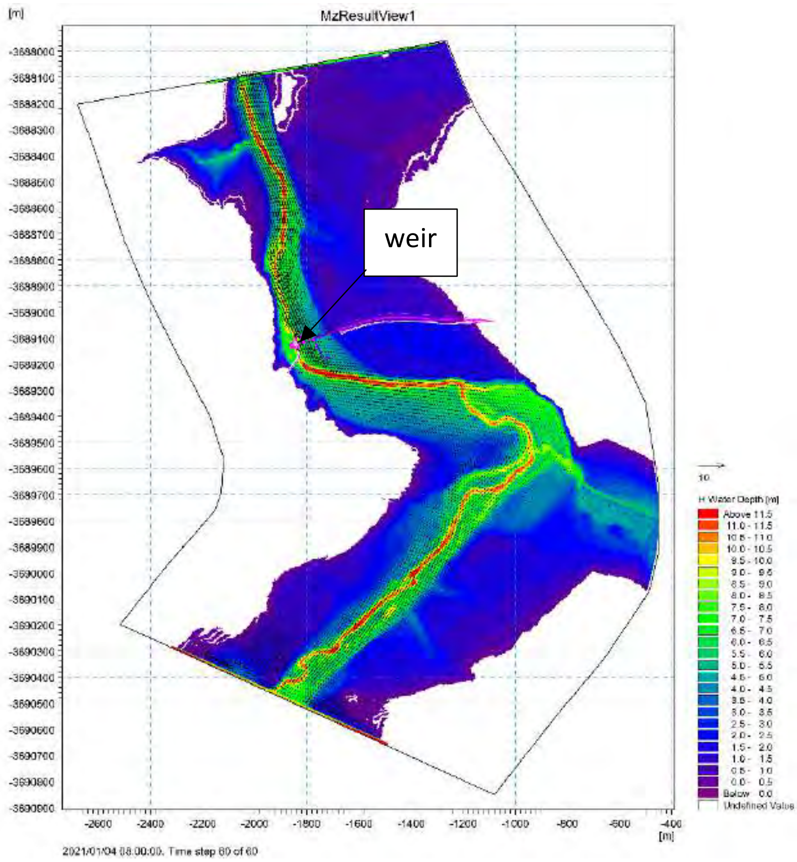


Figure 5.3-24: Water depth and velocity vectors for the Q100cc flood peak (Option B2)

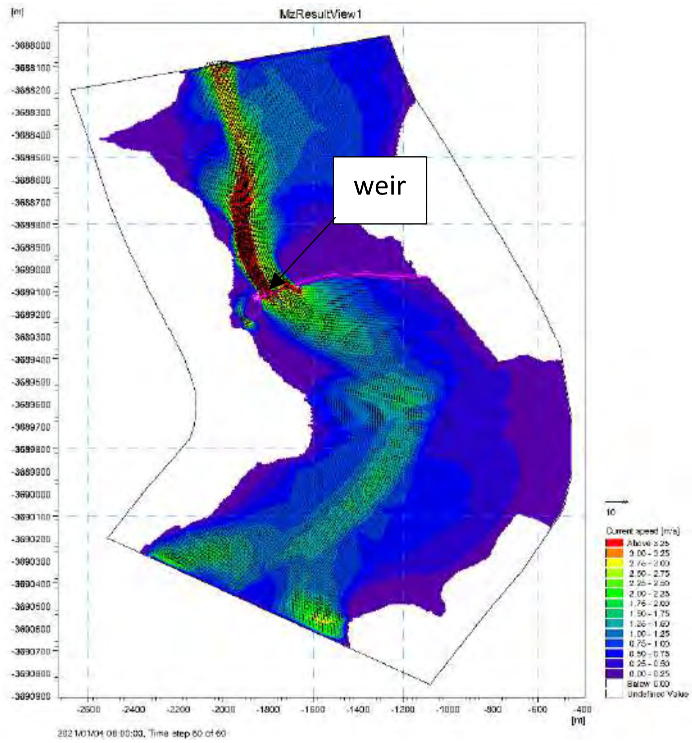


Figure 5.3-25: Flow velocity and velocity vectors for the RMFcc (Option B2)

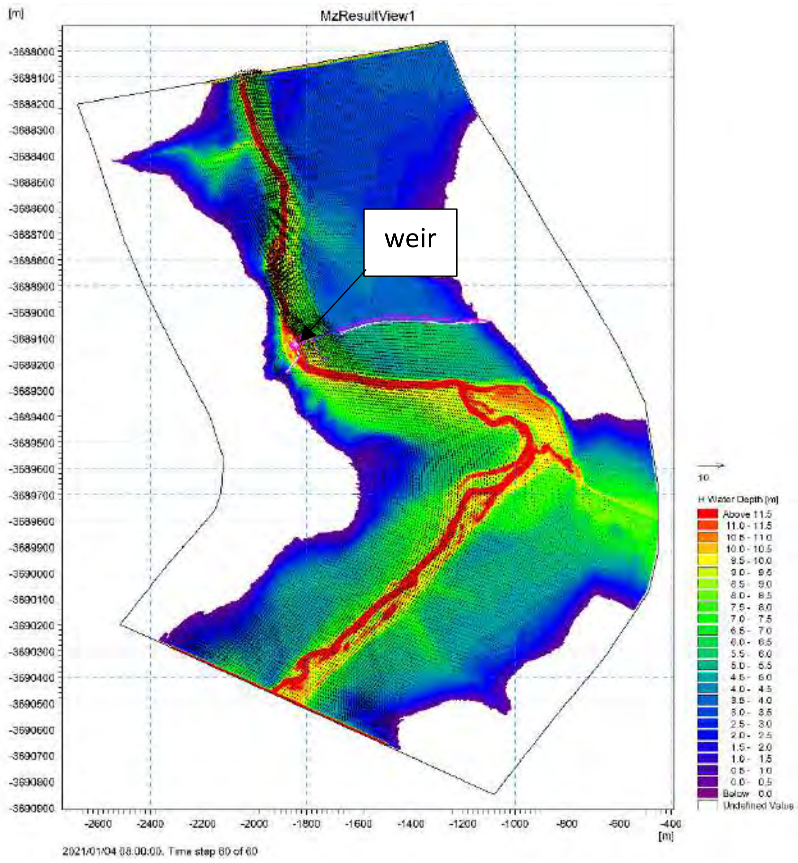


Figure 5.3-26: Water depth and velocity vectors for the RMFcc (Option B2)

b) Sediment transport simulation results for the Q50 year and Q100 year flood hydrograph with movable bed (current scenario with weir)

The bathymetry used in the model after including the selected weir and the abstraction works (Option B2) is shown in Figure 5.3-27. The simulation results for the current scenario for 50 year flood hydrograph routed in the model, with movable bed (bedrock based on DWS (2012) data) are shown in Figures 5.3-28 to 5.3-33. Similar simulation results for 100 year flood hydrograph are shown in Figures 5.3-34 to 5.3-39. Figures showing the full bathymetry are shown in **Appendix D2**. The central guide wall is effective to increase the flow velocities near the left bank side at the intake to above 2 m/s to ensure local scour near the intake, which will also ensure that the main channel remains near the left bank intake and that coarse bedload is transported towards the inside of the bend by the secondary currents, away from the intake.

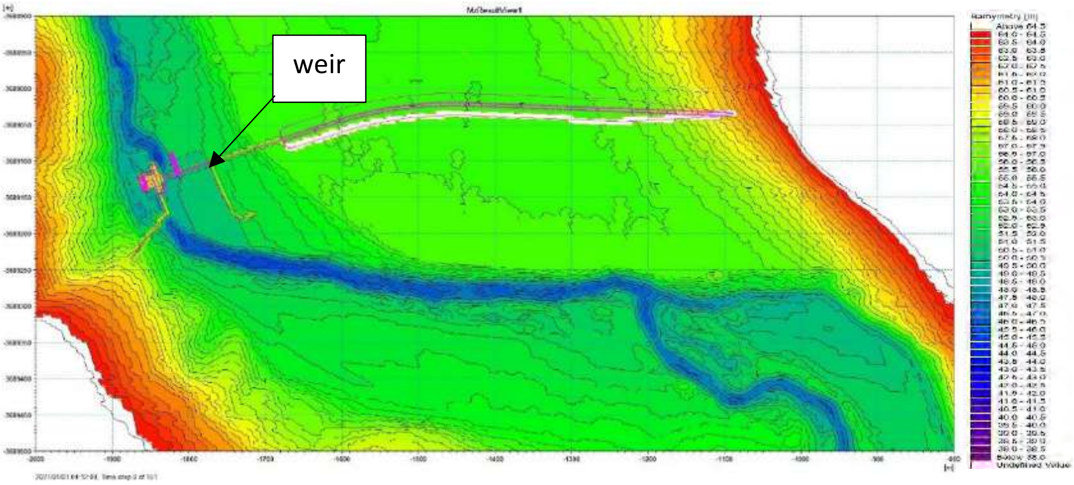


Figure 5.3-27: Bathymetry (2021 survey) used in the Mike 21C model (masl) with the proposed abstraction works and weir added (Option B2)

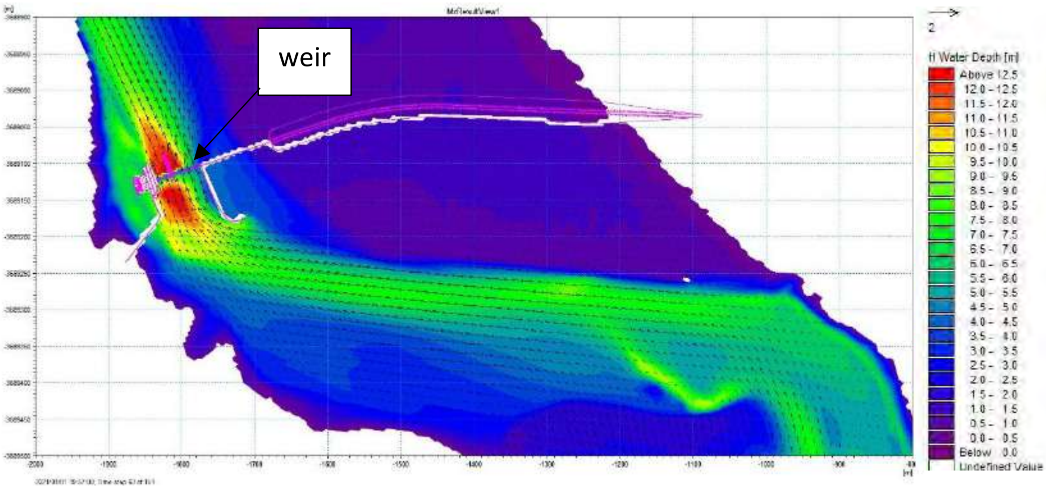


Figure 5.3-28: Simulated maximum flow depths with velocity vectors at the peak of the 50 year flood (current scenario)

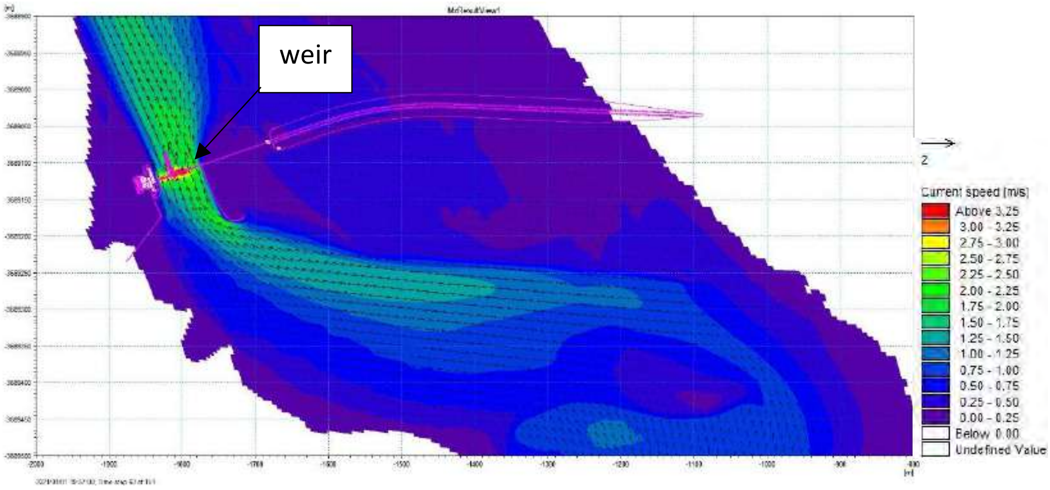


Figure 5.3-29: Simulated maximum flow velocities with velocity vectors at the peak of the 50 year flood (current scenario)

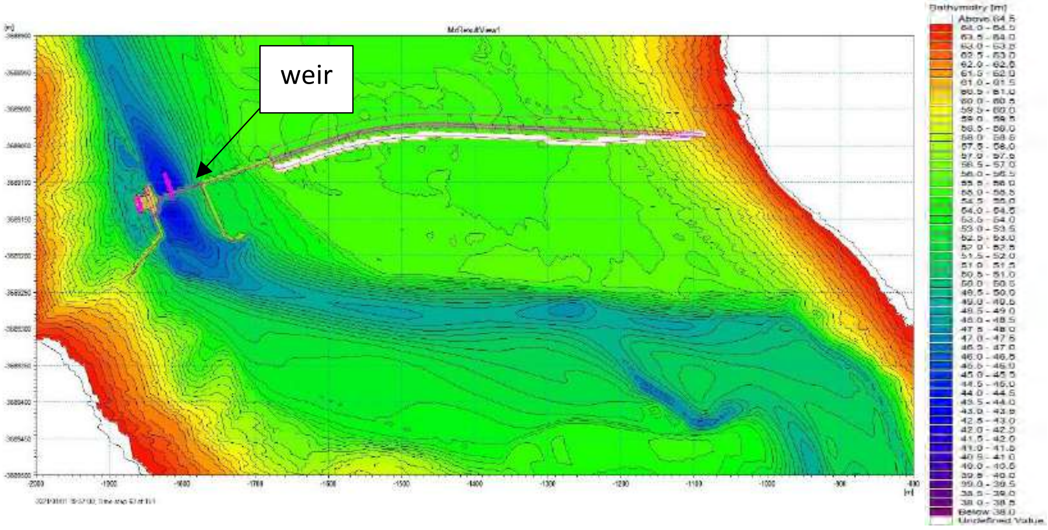


Figure 5.3-30: Simulated bed levels at the peak of the 50 year flood (current scenario)

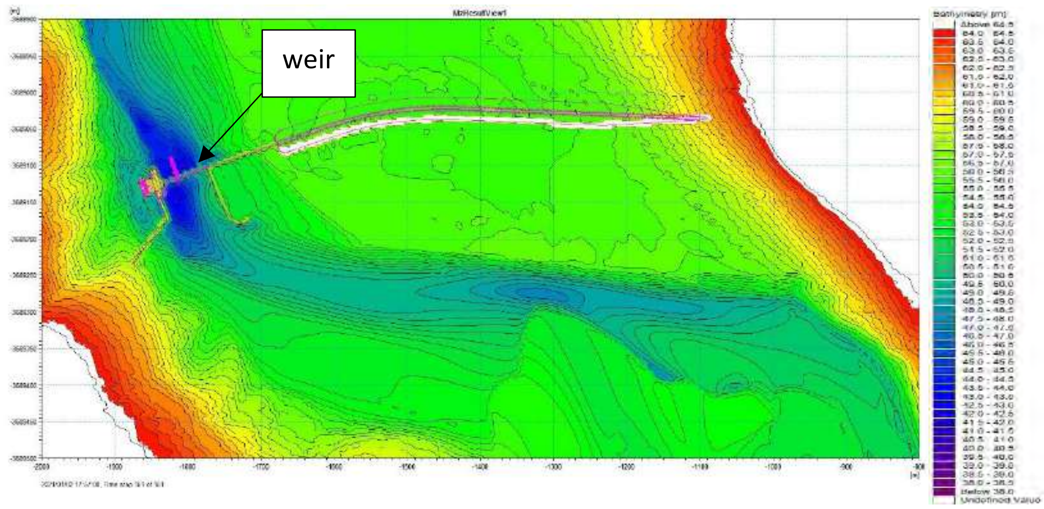


Figure 5.3-31: Simulated bed levels at the end of the 50 year flood (current scenario)

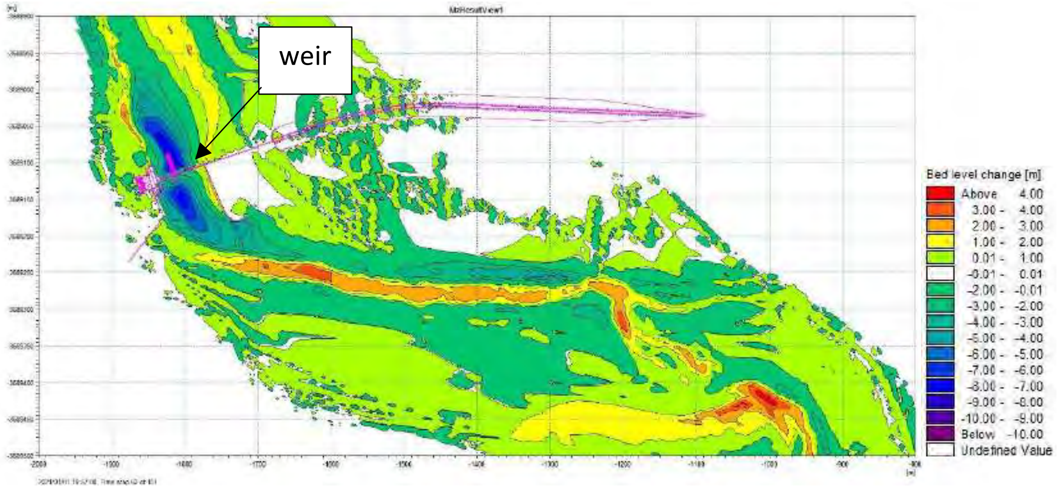


Figure 5.3-32: Simulated bed level change at the peak of the 50 year flood (current scenario)

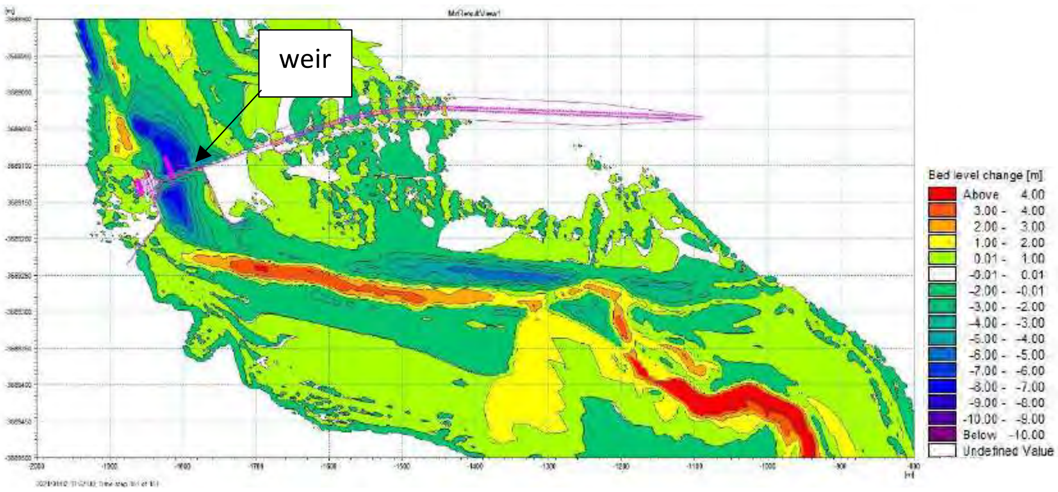


Figure 5.3-33: Simulated bed level change at the end of the 50 year flood (current scenario)

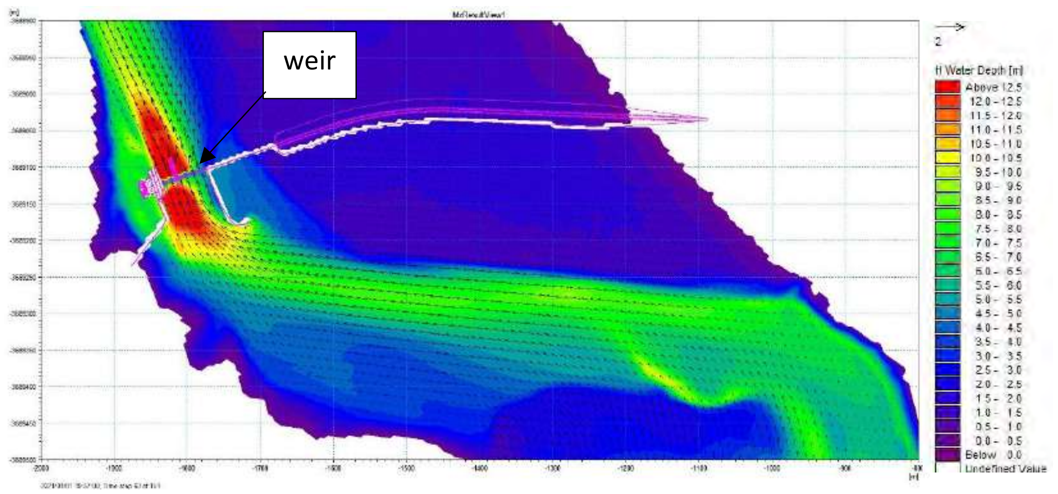


Figure 5.3-34: Simulated maximum flow depths with velocity vectors at the peak of the 100 year flood (current scenario)

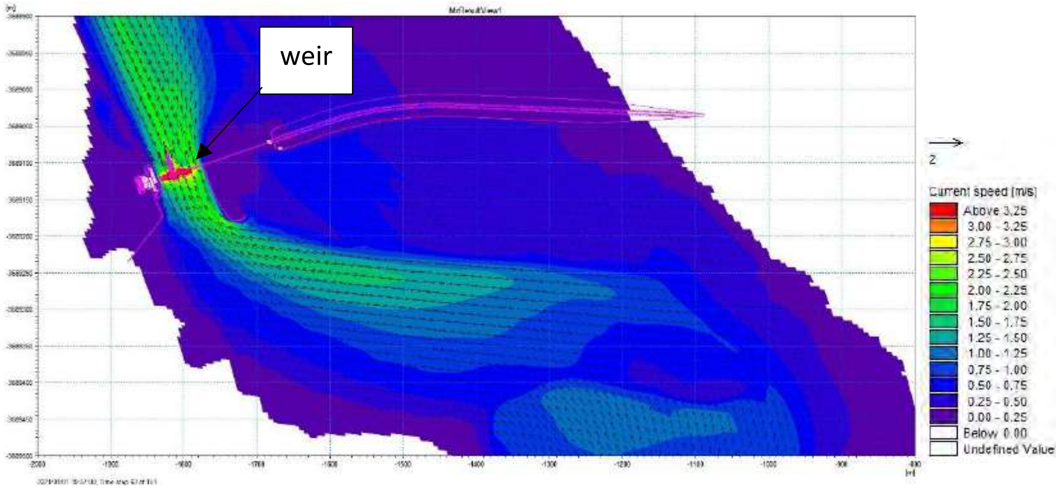


Figure 5.3-35: Simulated maximum flow velocities with velocity vectors at the peak of the 100 year flood (current scenario)

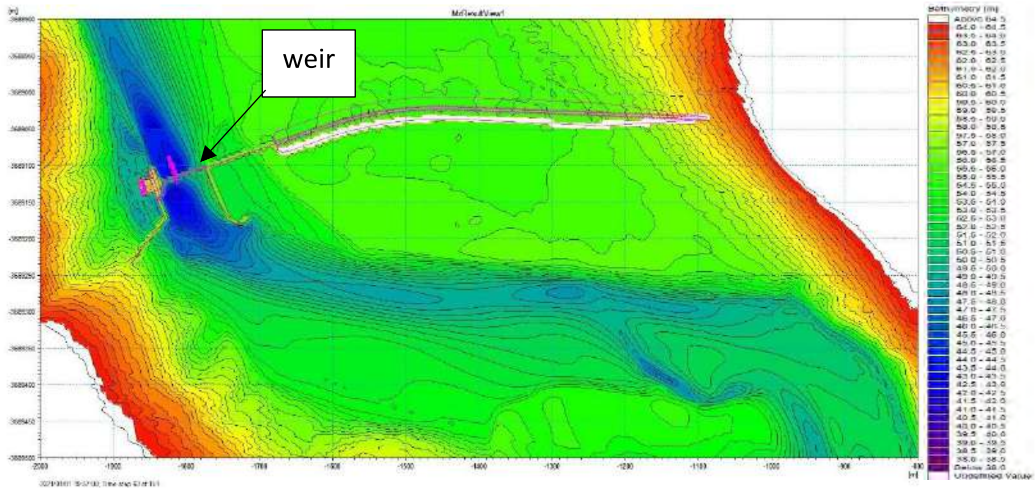


Figure 5.3-36: Simulated bed levels at the peak of the 100 year flood (current scenario)

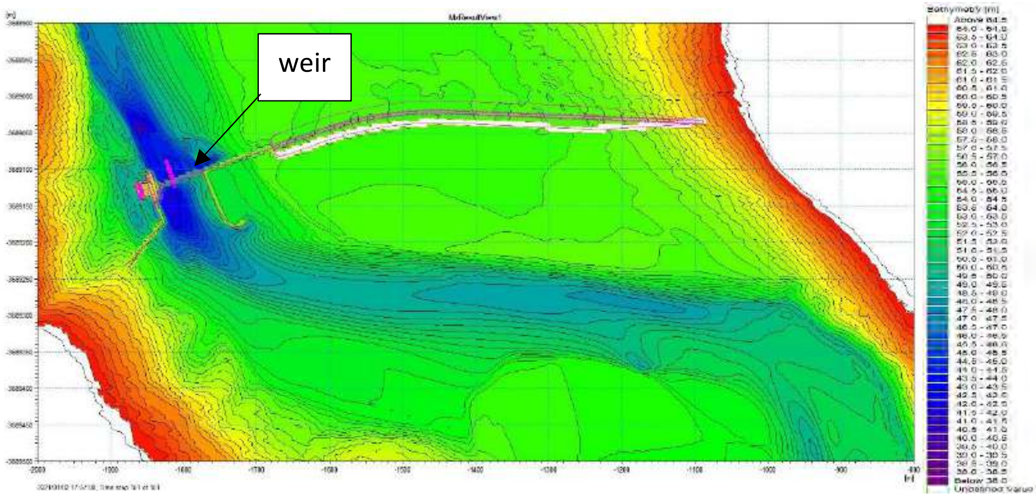


Figure 5.3-37: Simulated bed levels at the end of the 100 year flood (current scenario)

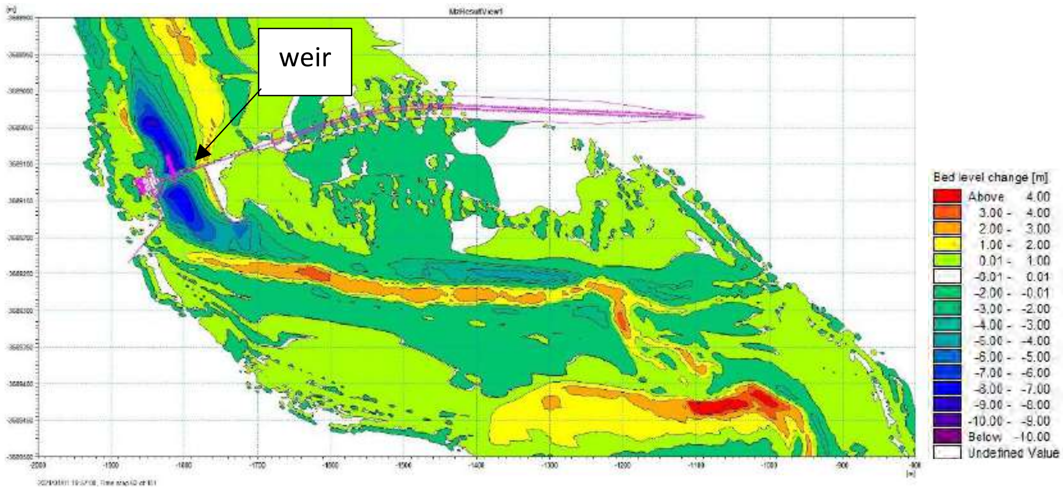


Figure 5.3-38: Simulated bed level change at the peak of the 100 year flood (current scenario)

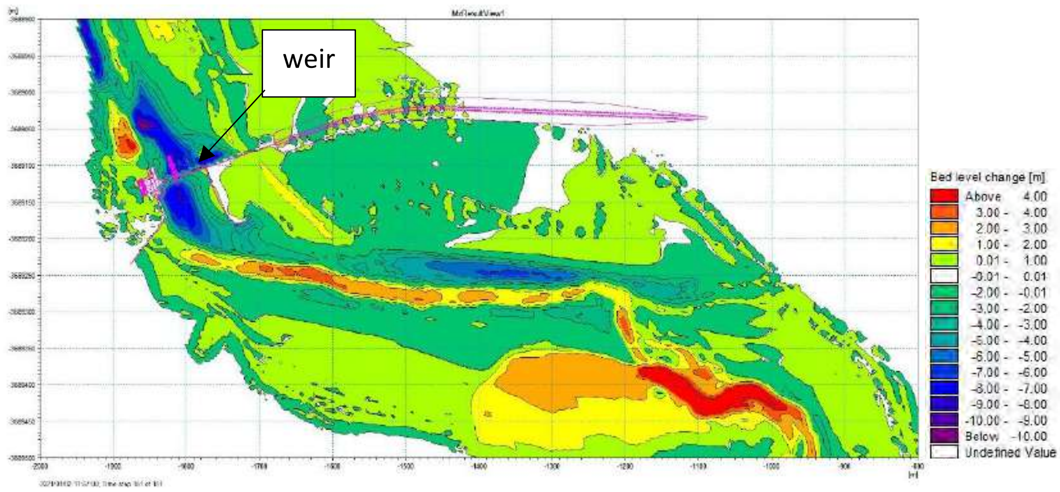


Figure 5.3-39: Simulated bed level change at the end of the 100 year flood (current scenario)

c) 15 years long term simulation results with movable bed

Model setup

The long term simulation with movable bed was conducted using 15 years historical observed flow data. The 15 year period was selected from the DWS flow database to be reliable data, and the extreme drought period since 2015 was excluded because this was not a normal flow period with low sediment loads. The Berg River Dam was completed in 2008 in the upper Berg River catchment and does have a flood attenuation impact at the BRVAS site. The DWS Berg River baseline study of 2006 estimated that the Berg River Dam will attenuate the Q20 flood at Hermon of 550 m^3/s for the pre-Berg River Dam to 385 m^3/s post-Berg River Dam (without environmental flood releases from the dam), which is a 24% flood peak decrease. The observed flow record of Figure 5.3-40 was however used without flood peak adjustments before 2008, because the Berg River Dam is actually designed with an environmental flood release outlet to counteract the flood attenuation caused by the dam. The record before year 2000 could not be used due to gaps and/or unreliable flood peaks, but the 15 year record used is long enough to simulate equilibrium sedimentation conditions upstream of the weir with its relatively small initial storage capacity.

Based on the calculated sediment yield of the catchment of the proposed weir and abstraction works location, a time series of sediment concentrations for the corresponding 15 years historical flow data was used as input at the upstream boundary of the model. The sediment fraction distribution was 85% for the cohesive sediment and 15% for the non-cohesive sediment. The different sediment sizes used for this simulation were 0.90 mm, 0.21 mm, 0.075 mm and 0.011 mm (cohesive). The discharge - sediment concentration relationship was established from observed data obtained during the 2003 to 2006 Berg River Baseline Monitoring Study of DWS, and subsequently adjusted slightly to account for the current sediment yield at the BRVAS site (refer to Section 12). Figure 5.3-40 shows the inflow time series and Figure 5.3-41 depicts the sediment concentration time series used in the hydrodynamic model.

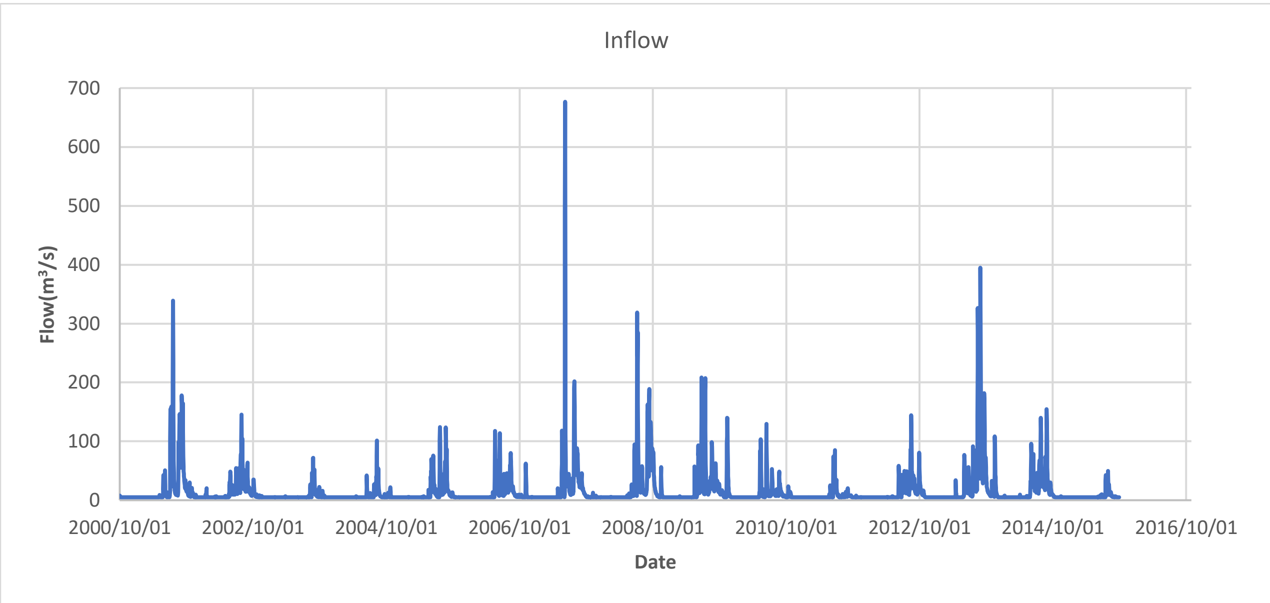


Figure 5.3-40: 15 year Berg River flow data

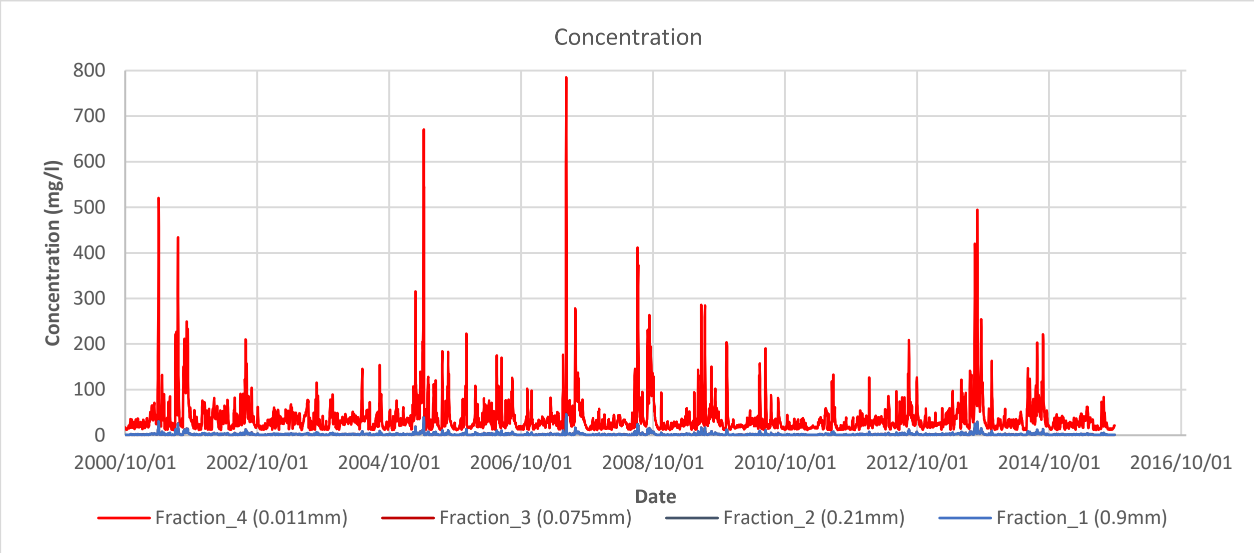


Figure 5.3-41: 15 year sediment concentration time series

Hydrodynamic model simulation results for hydraulic design option B2

Figures 5.3-42 to 5.3-44 show the simulated bed levels and bed level change after 15 years. The proposed intake zone remains sediment free, but the river upstream of the weir experiences sediment deposition which is acceptable as long as the intake remains clear. Note that Figure 5.3-43 indicates the simulated bathymetry of the riverbed after 15 years, and Figures 5.3-43 and 5.3-44 show the sediment deposition/scour, in meters, after 15 years. Although the simulated deposition upstream of the weir is high, the pump intake area is scoured due to the effect of secondary currents that exists on the outside bend of the main channel against the boulder trap. The river bed downstream of the proposed weir will experience scour due to the sedimentation upstream of the weir and the turbulence caused by the weir. Sediment transport under low flow conditions are reduced due to the reduction in flow velocity caused by the damming caused by the weir. The weir in turn is raised above the natural ground level and the combination of accelerating flow (critical flow over the weir crest) and falling water causes downstream scour. The scour is limited by the relatively shallow bedrock at 40 masl to 44 masl (DWS, 2012).

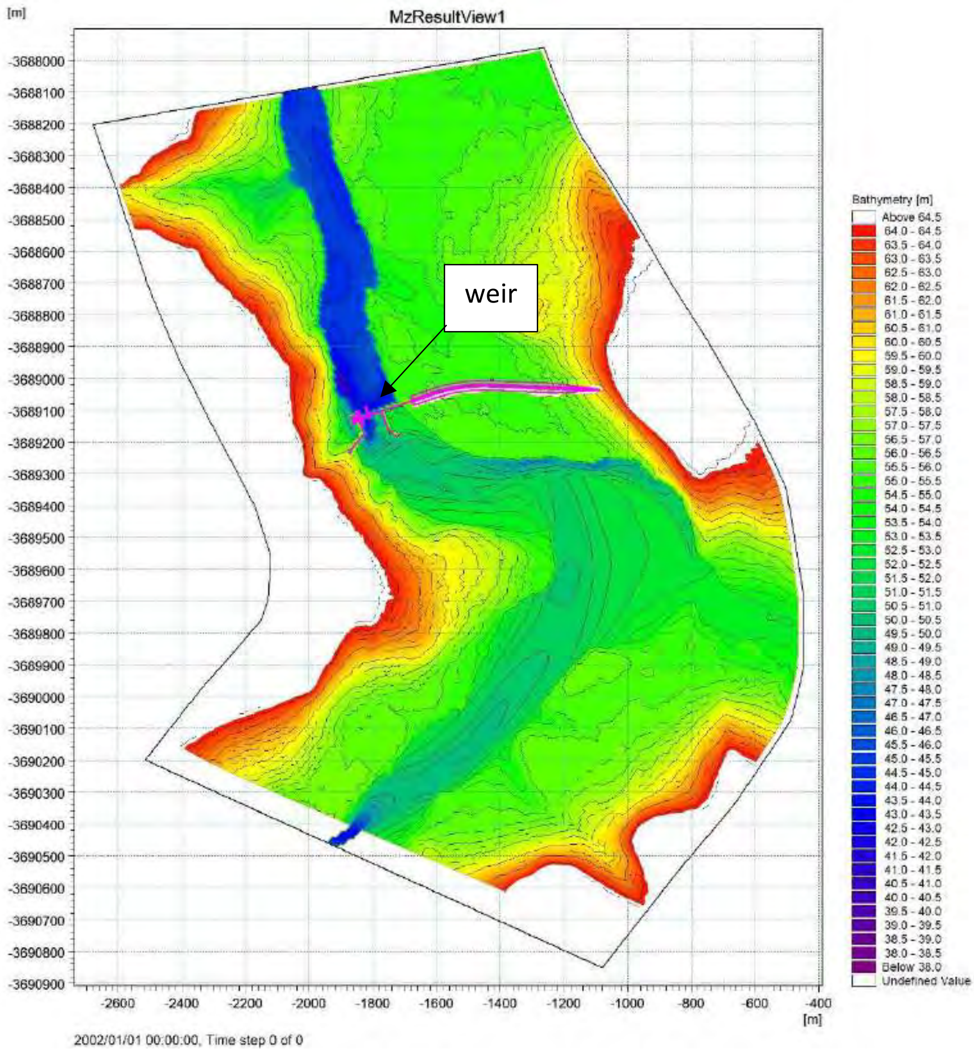


Figure 5.3-42: Simulated bed level after 15 years (masl)

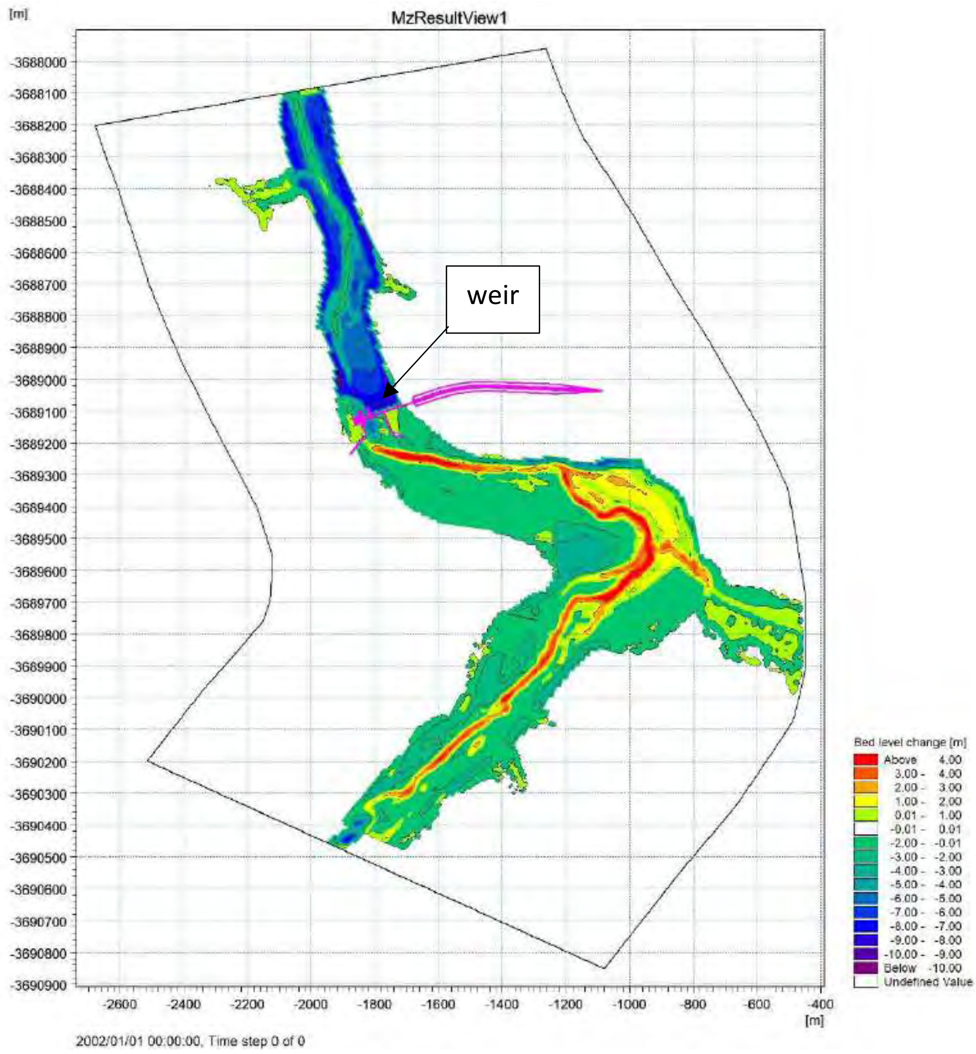


Figure 5.3-43: Simulated bed level change after 15 years

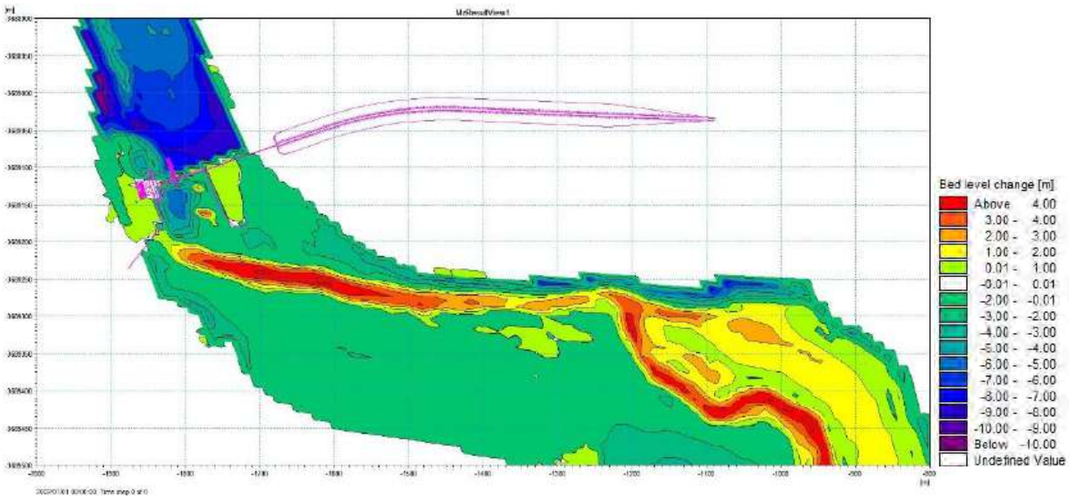


Figure 5.3-44: Close up view of the weir of the simulated bed level change after 15 years

The simulated total sediment concentrations and the sediment load near the flank wall upstream of the boulder trap are given in Figures 5.3-45 and 5.3-46. The data is used in the sediment mass balance calculations (Section 12).

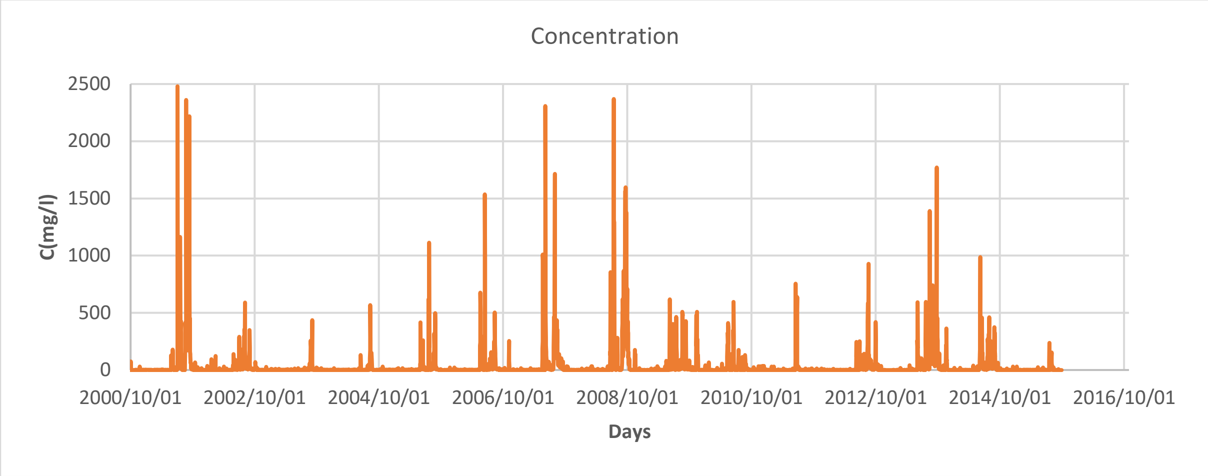


Figure 5.3-45: Simulated sediment concentrations upstream of the boulder trap

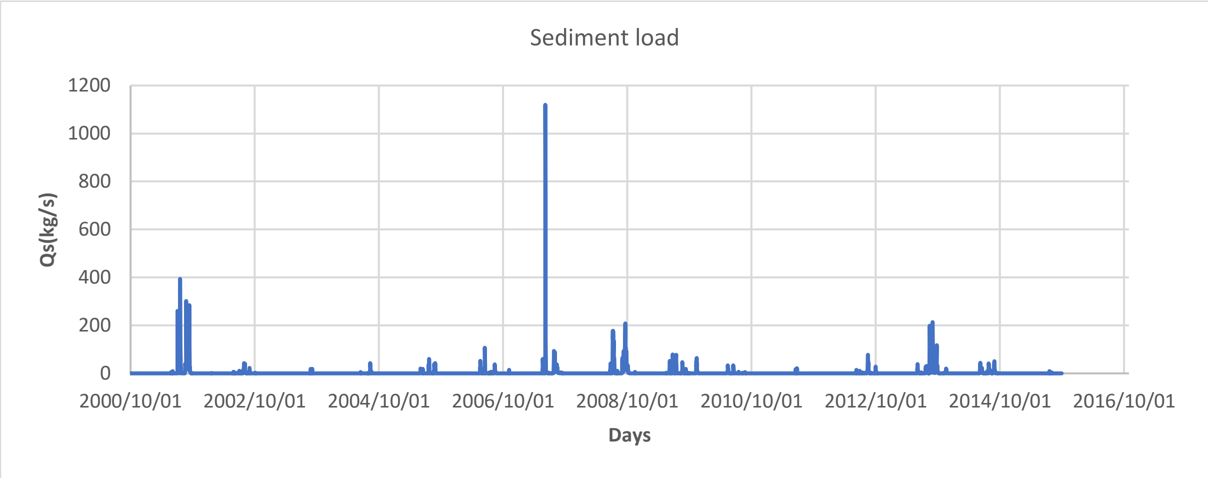


Figure 5.3-46: Simulated sediment loads upstream of the boulder trap

d) Sediment transport simulation results for the 50 year and 100 year flood hydrograph with movable bed (future scenario after 15 years of operation) and hydraulic design option B2

In this scenario, the 50 year flood and 100 year flood hydrographs were routed in the model using the final bed level obtained after the 15 years long term simulation as initial bathymetry. The initial bathymetry used in the model is shown in Figure 5.3- 47. The simulation results for the current scenario for the 50 year flood hydrograph routed in the model, with a movable bed are shown in Figures 5.3-48 to 5.3-53. Similar simulation results for the 100 year flood hydrograph are shown in Figures 5.3-54 to 5.3-59. Figures showing the full bathymetry are shown in **Appendix D2**.

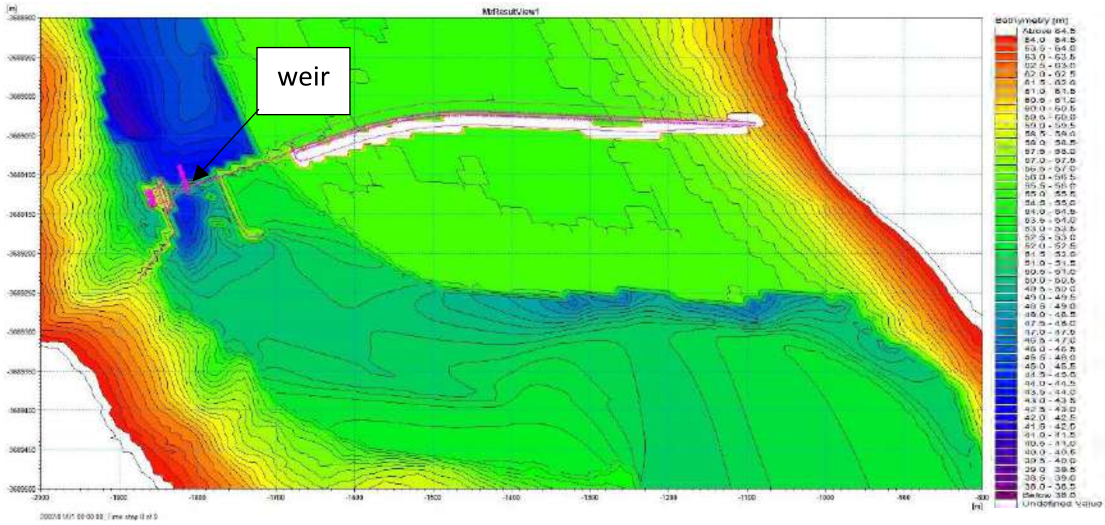


Figure 5.3-47: Initial bed level (masl) with the proposed abstraction works and weir added

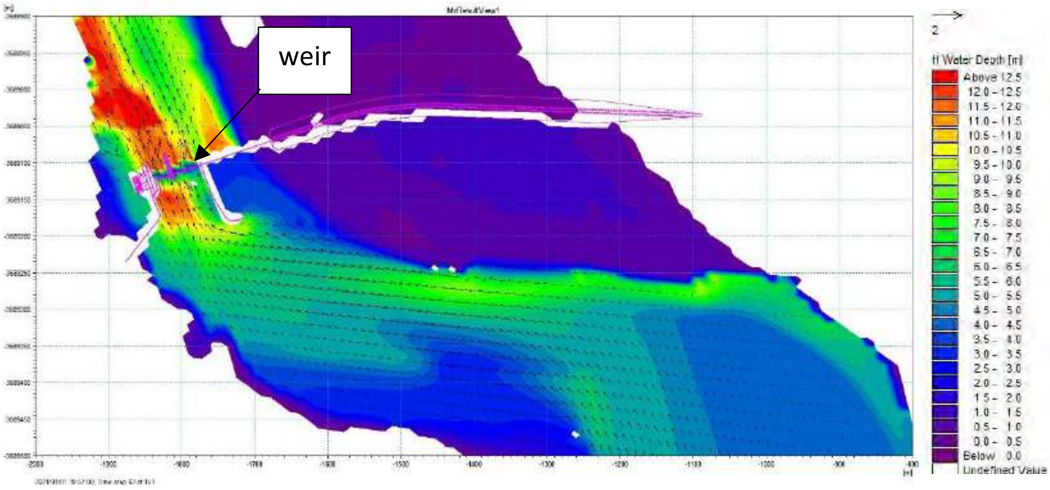


Figure 5.3-48: Simulated maximum flow depths with velocity vectors at the peak of the 50 year flood (future scenario)

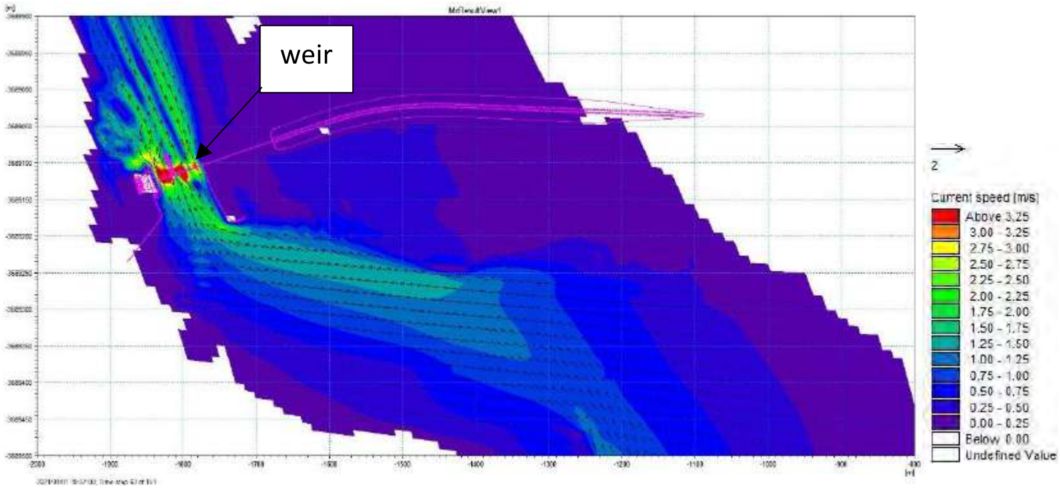


Figure 5.3-49: Simulated maximum flow velocities with velocity vectors at the peak of the 50 year flood (future scenario)

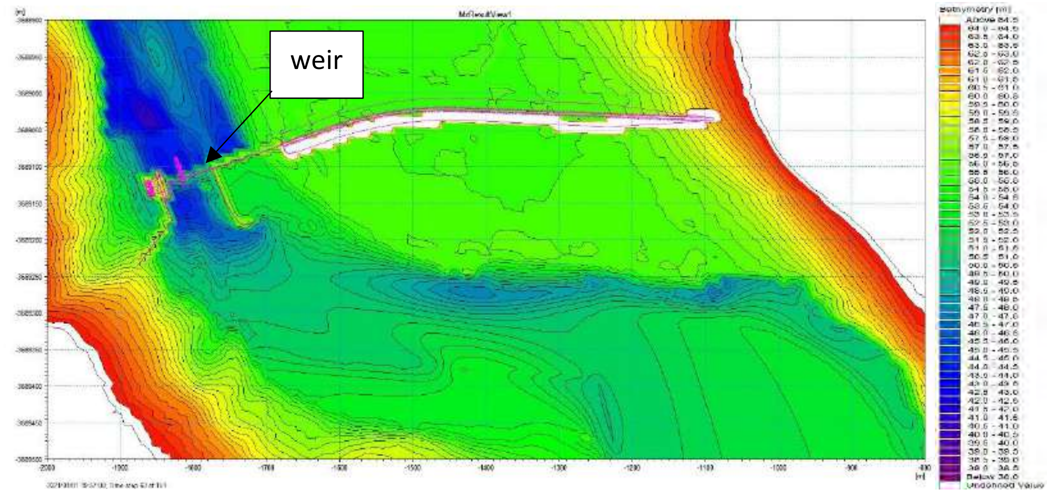


Figure 5.3-50: Simulated bed levels at the peak of the 50 year flood (future scenario)

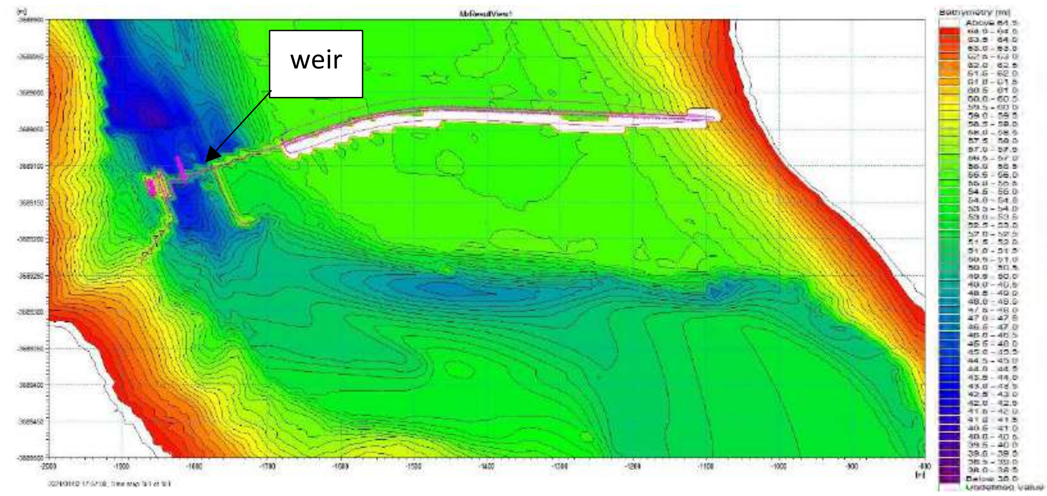


Figure 5.3-51: Simulated bed levels at the end of the 50 year flood (future scenario)

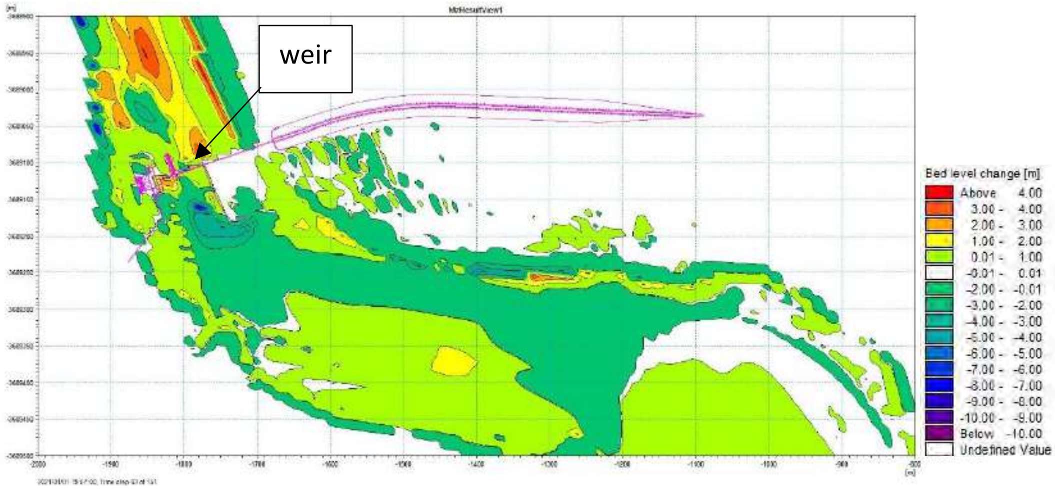


Figure 5.3-52: Simulated bed level change at the peak of the 50 year flood (future scenario)

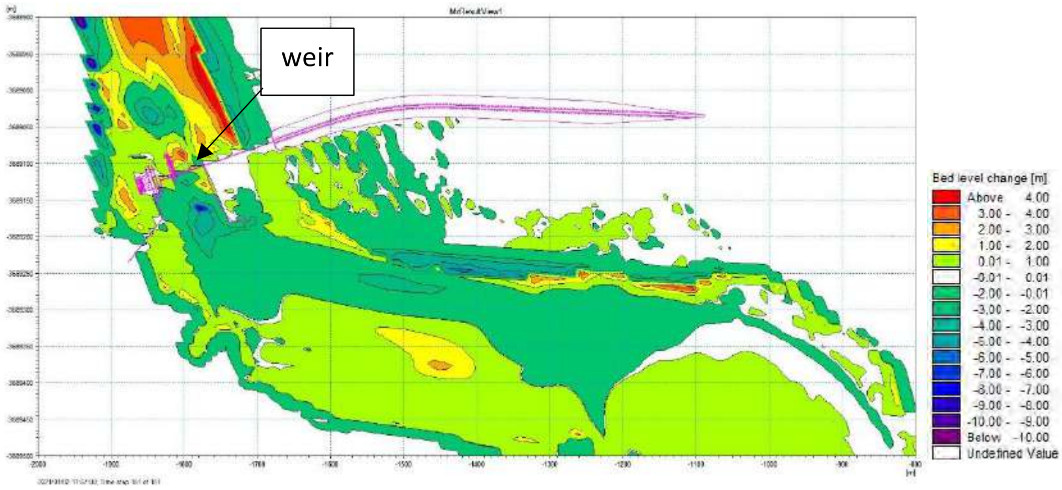


Figure 5.3-53: Simulated bed level change at the end of the 50 year flood (future scenario)

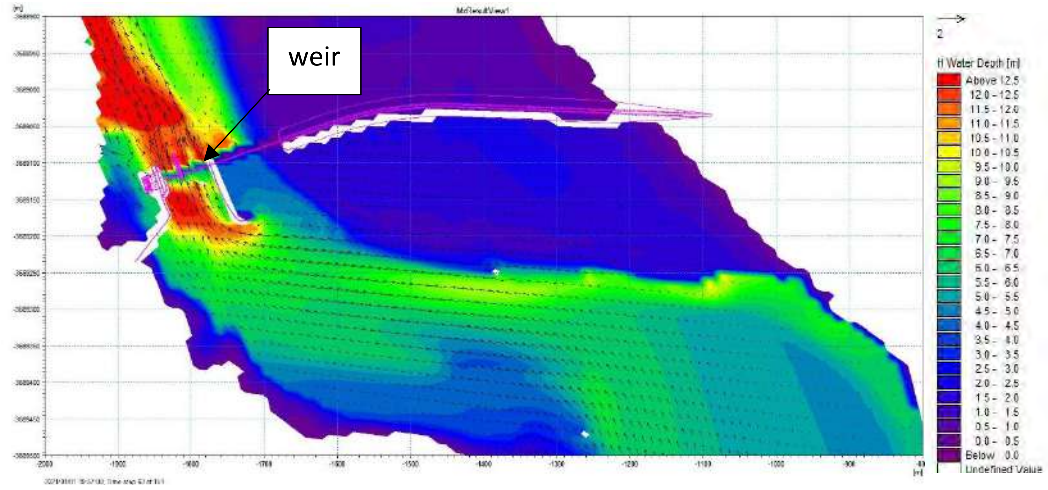


Figure 5.3-54: Simulated maximum flow depths with velocity vectors at the peak of the 100 year flood (future scenario)

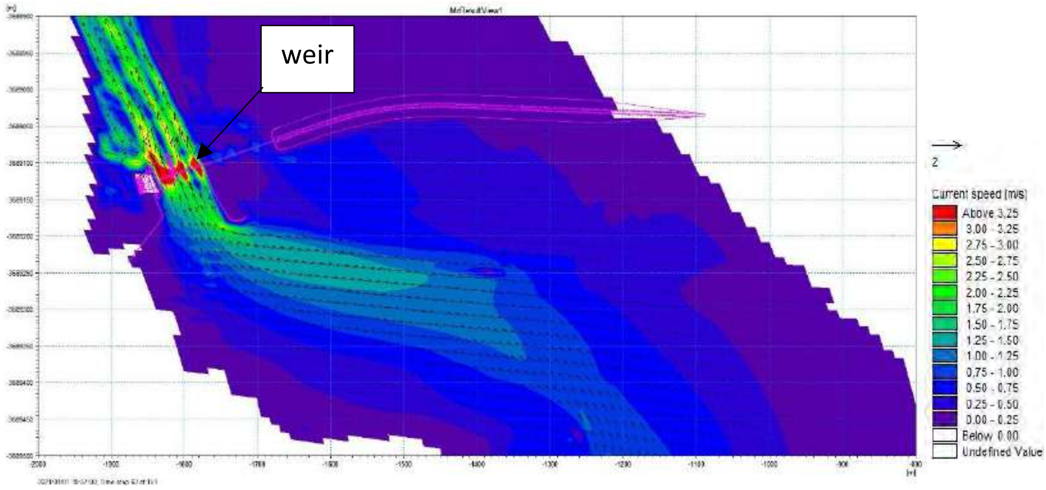


Figure 5.3-55: Simulated maximum flow velocities with velocity vectors at the peak of the 100 year flood (future scenario)

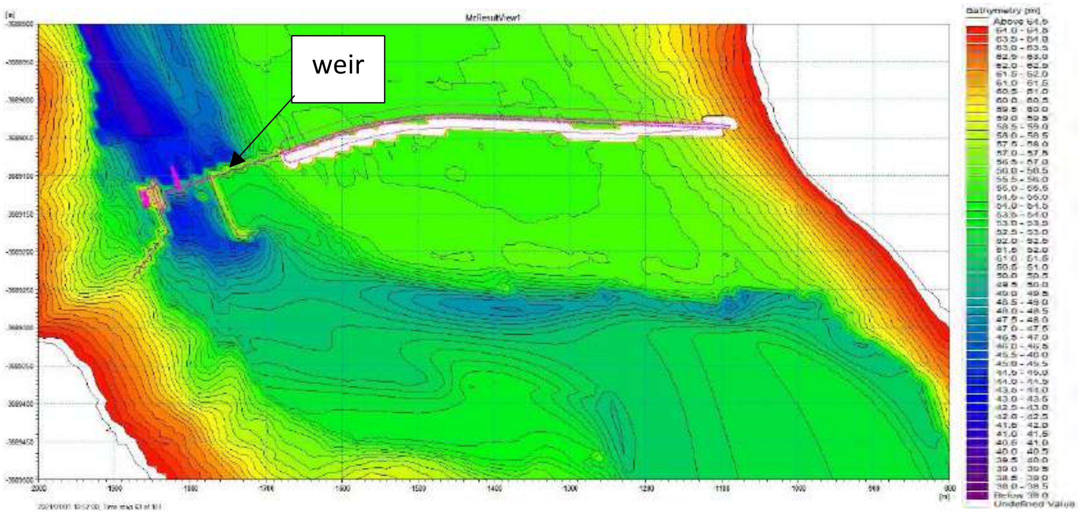


Figure 5.3-56: Simulated bed levels at the peak of the 100 year flood (future scenario)

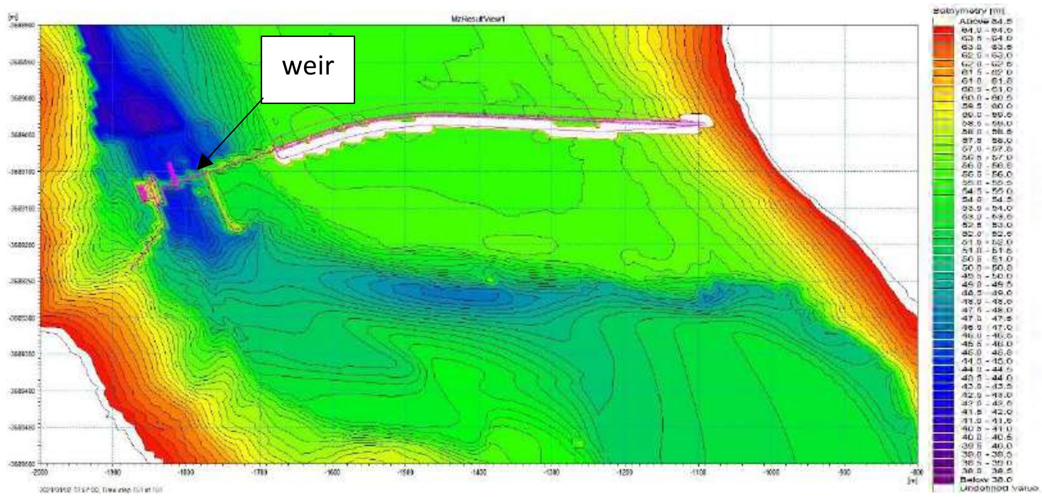


Figure 5.3-57: Simulated bed levels at the end of the 100 year flood (future scenario)

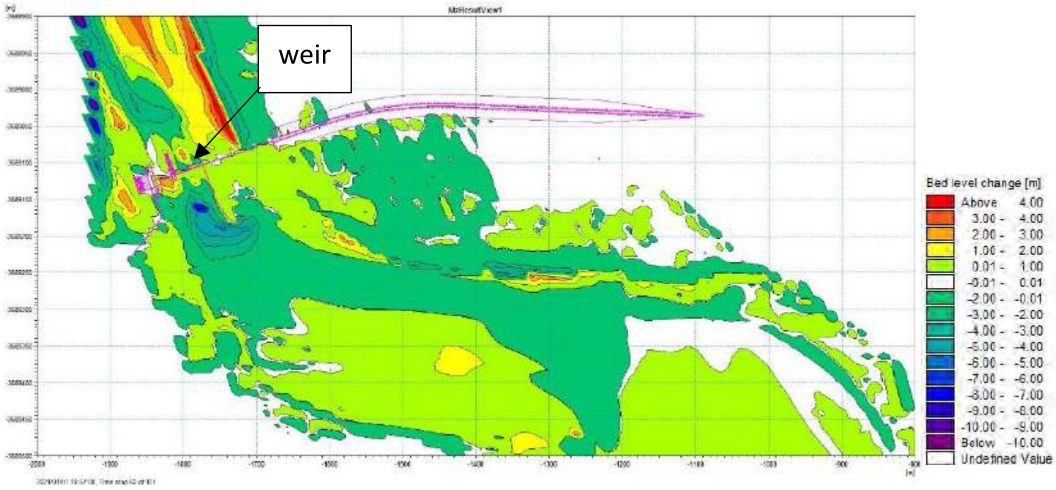


Figure 5.3-58: Simulated bed level change at the peak of the 100 year flood (future scenario)

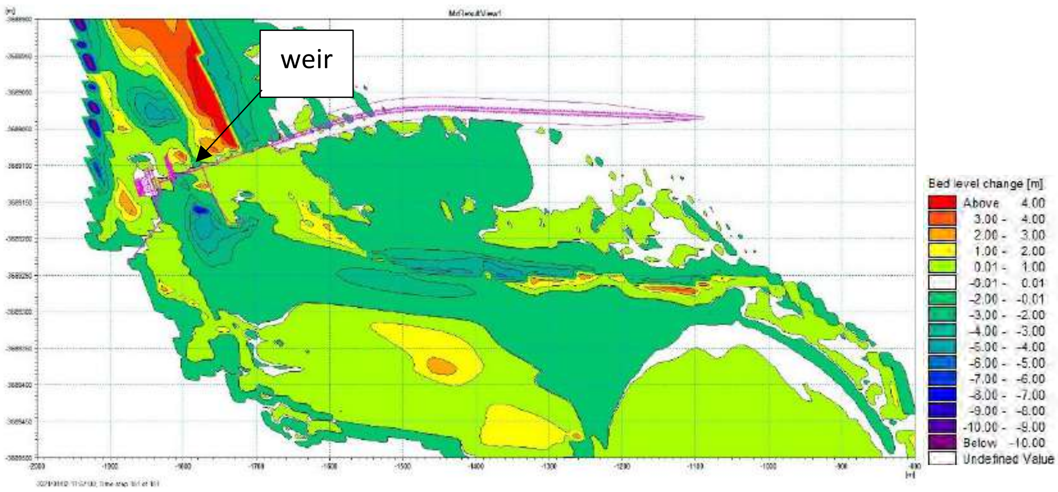


Figure 5.3-59: Simulated bed level change at the end of the 50 year flood (future scenario)

The simulated total sediment concentration and the sediment load near the flank wall upstream of the boulder trap are given in Figures 5.3-60 and 5.3-61.

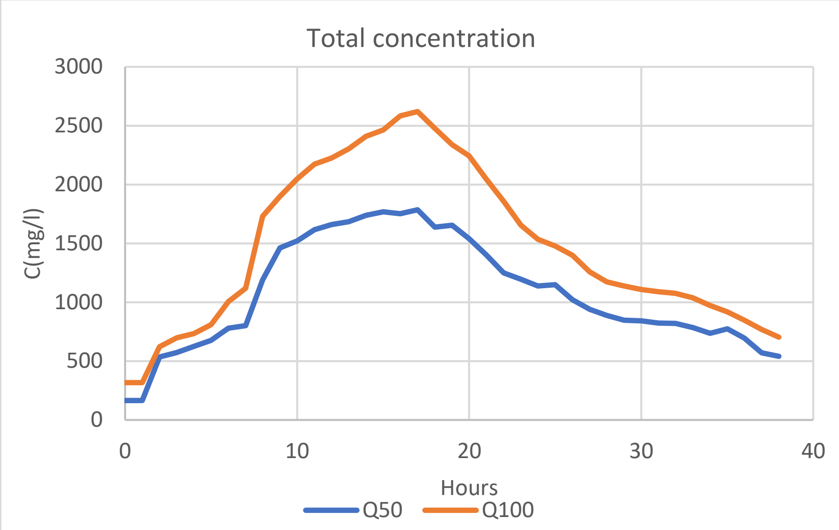


Figure 5.3-60: Simulated sediment concentration upstream of the boulder trap

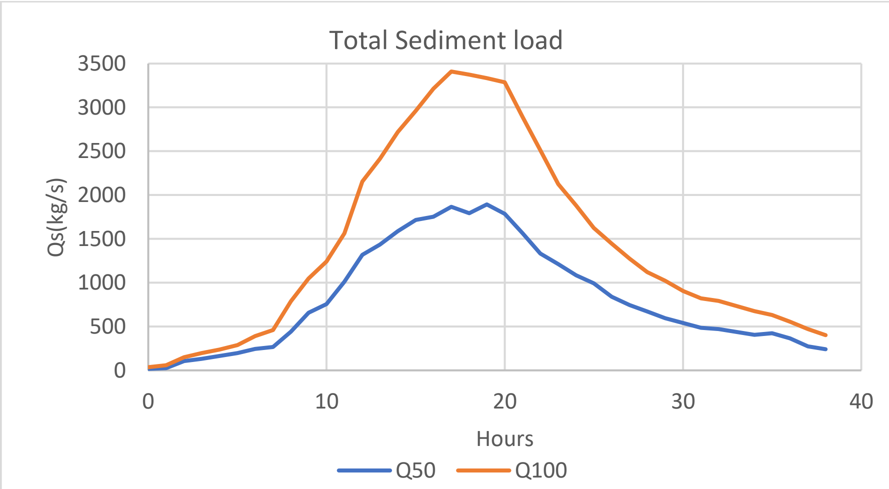


Figure 5.3-61: Simulated sediment load upstream of the boulder trap

Long sections of the simulated maximum water levels as well as the simulated bed levels for all the scenarios are given in Figure 5.3-62. The "initial bed level" in the legend indicated in the figure is the 2021 surveyed bed levels. Also note that in this simulation the low notch of the Crump weir elevation is at 51.6 masl (the final proposed elevation), while long sections shown earlier in the report were typically based on previous hydraulic designs which had lower weir crest levels). Some of the key findings are:

- The weir will cause sedimentation upstream of the weir in the order of 3 to 6 m higher than the current river main channel bed levels.
- The low notch of the weir is almost completely drowned during the Q50cc and Q100cc floods, with limited additional damming caused by the weir.
- Upstream near the weir the Q100cc flood after 15 years of sedimentation causes less than 0.3 m additional damming compared to the current surveyed bed scenario with a weir, even when considering future sediment deposition in the river reach upstream of the weir. These differences were considered in the floodline determination (Section 9.8).

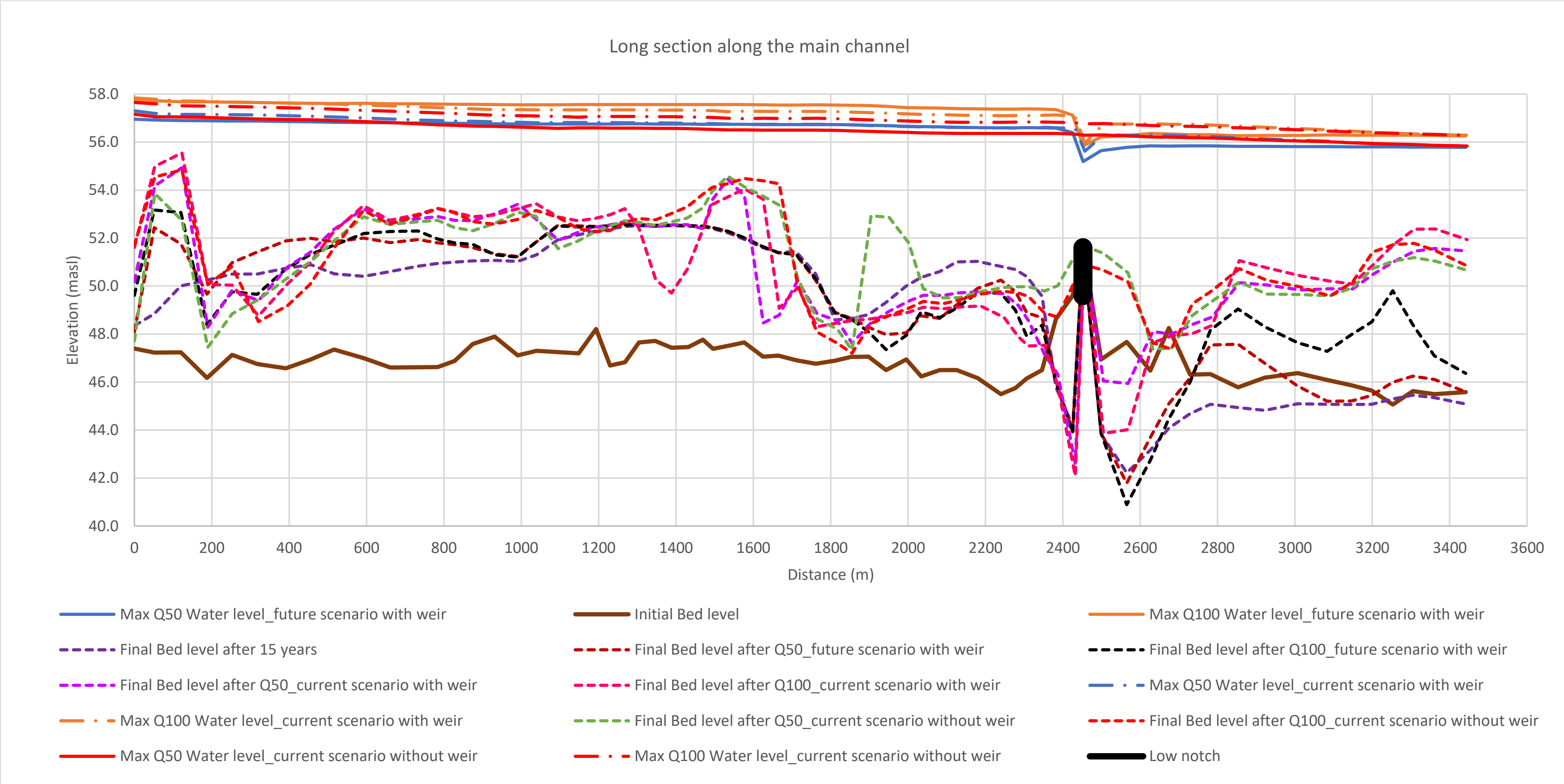


Figure 5.3-62: Simulated maximum water levels and bed levels indicated in a long- section along the main river channel

6. Design of the physical model

This section addresses the following model design aspects of the 1:40 undistorted scaled physical hydraulic model according to the Froude model scale laws:

- Scaled factors relevant to the 1:40 Froude scaled model.
- Overall model dimensions and laboratory setup.
- Detailed dimensions of the abstraction works that were built in the physical model. These dimensions evolved from the initial study concept (Section 3), additional field data, numerical modelling (Section 5) and initial physical model testing.

6.1 Scale factors relevant for the 1:40 Froude scaled model

Table 6-1 below presents the scale factors for different parameters relevant in the physical model and are applicable to convert prototype parameters to model parameters and vice versa.

Table 6-1: Scale factors for a 1:40 scale Froude model

Parameter	Dimensions	Scale factors		Comment
Density	kg/m³	1		If model and prototype densities are the same
Length	m	40		
Surface area	m²	40^2	1,600	
Volume	m³	40^3	64,000	
Time	s	40^0.5	6.32	
Velocity	m/s	40^0.5	6.32	
Acceleration	m/s²	1.00		
Discharge	m³/s or l/s	40^(5/2)	10,119.29	
Force	N	40^3	64,000	
Pressure	N/m²or Pa	40.00		
Reynolds Number	-	40^(3/2)	252.98	

6.2 Model dimensions and laboratory setup

The prototype spatial area covered by the model (i.e. the model boundary) is shown in Figure 6-1. The overall aerial dimensions of the physical laboratory model were 30 m by 27 m with inflow stilling basin on the upstream boundary and control gates at the downstream boundary. The model was constructed in the Hydraulic Laboratory at Stellenbosch University.

The entire model was contained in a brick wall basin. The natural topography and excavated chute were constructed with a sand cement mixture and the abstraction works and weir were made of marine ply, with intricate components that were 3D printed. Figure 6-2 shows the templates that were used based on topographical survey data to shape the model contours. The photographs of Figures 6-3 and 6-4 show how the physical model topography was then modified to incorporate the new surveyed levels as well as the BRVAS abstraction works and weir structure.

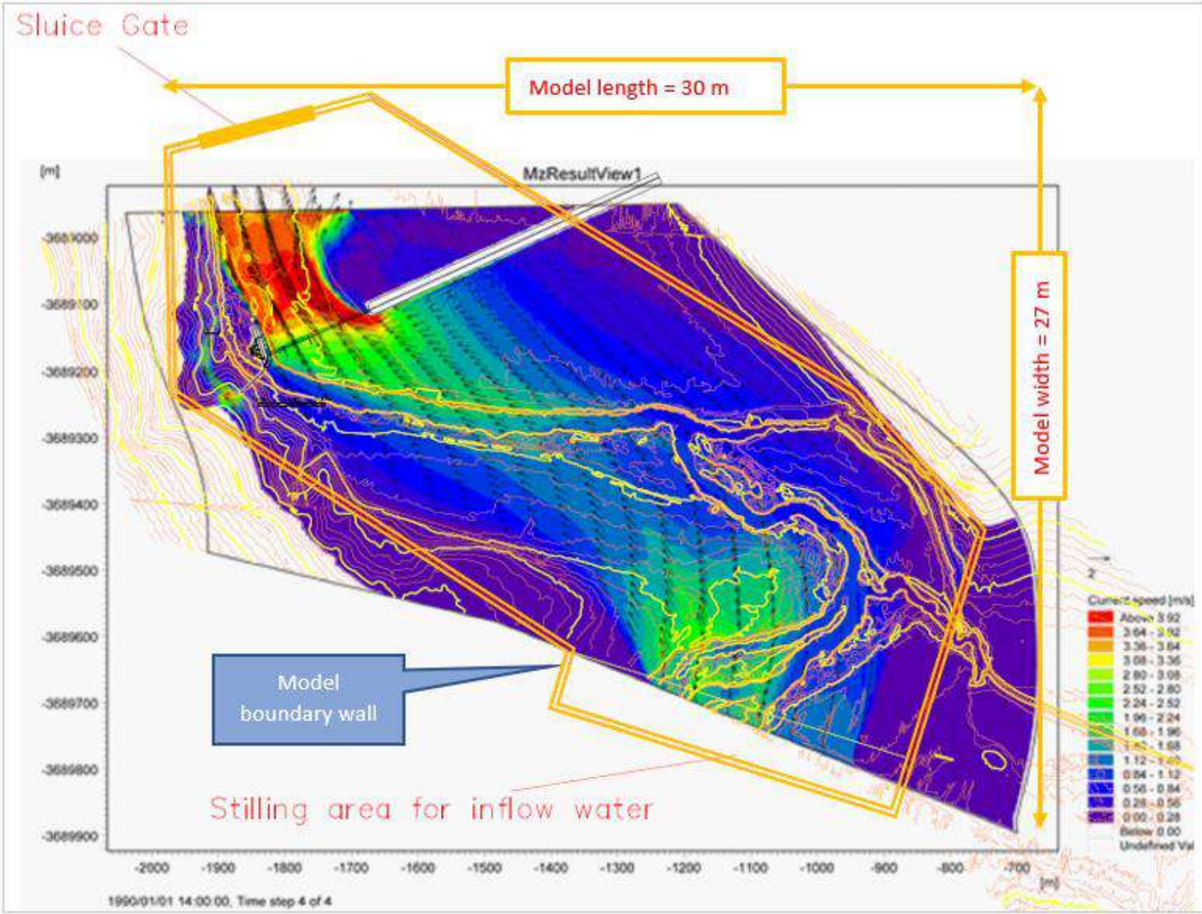


Figure 6-1: Physical model boundaries with 3D numerical model velocity distribution for RMF

Figure 6-2: Construction of the physical model topography of the Berg River and its floodplains

Figure 6-3: Modification of the physical model topography by incorporating new templates that are representative of the new 2021 surveyed levels

Figure 6-4: Construction of the proposed river abstraction works and weir in the physical model

The extent of the physical model included a river reach of 1 750 m of which 1 500 m was upstream of the abstraction works and 250 m downstream of it to ensure the flow patterns at the proposed site were not affected by any boundary effects. Baffle blocks and guide walls were used to streamline the inflow at the upstream boundary based on the hydrodynamic model simulations (as shown in Figure 6-5). The model was also wide enough to include high ground levels to contain the 50-year flood. Given the very wide floodplains of the Berg River site, any sections that would theoretically be flooded beyond the model's high ground boundary would not affect the flow patterns or flood elevations.

Fixed bed tests were later complimented with movable bed tests to evaluate the local sediment deposition and erosion patterns at the abstraction works and weir. For the movable bed tests, the river bed was topped with concrete to the expected rock elevations and covered with sediment scaled to be representative of the in situ riverbed sediment (Section 4.2) as shown in Figure 6-5. Coarser aggregate was used next to the movable bed to further optimize the required bed excavation levels. The movable bed tests are discussed in Section 9.5.

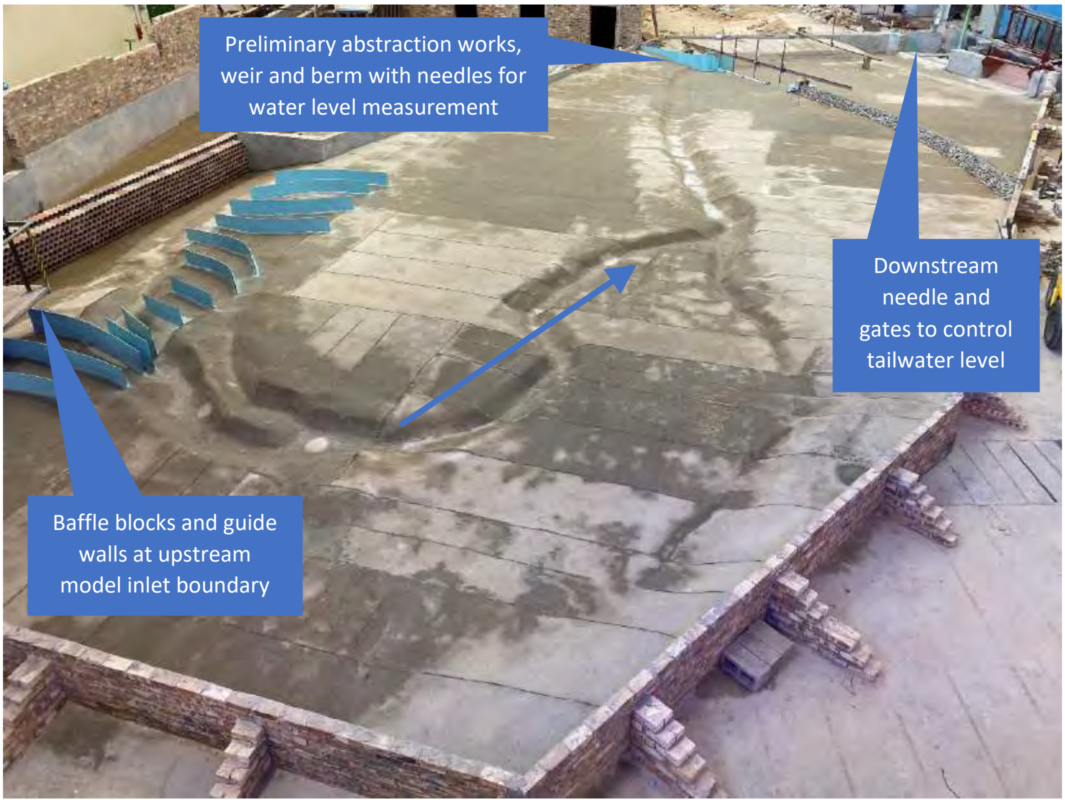


Figure 6-5: Fixed bed tests of the preliminary abstraction works and berm design

Figure 6-6: Movable bed tests of the final abstraction works design viewed from downstream

6.3 Dimensions of the intake model structure

As indicated earlier, the dimensions of the intake model structure to be built into the model, evolved from the initial study concept (Section 3), initial and subsequent numerical modelling and subsequent numerical modelling (Section 5), and additional field data.

Some of the dimensioned drawings for the model intake structure are shown in Figures 6-7 to 6-9 and in more detail in **Appendix A**. Provisional duty pumps and motive pumps to drive the jet pumps were selected to enable the appropriate sizing of the pump bays and hoppers and shown in **Appendix B**. The pump bays and hoppers were sized for a pump discharge of 6 m^3/s and will also work effectively for the 4 m^3/s. Some of the key features of the hydraulic design are:

- Abstraction works designed for total peak duty pump discharge of 6 m^3/s.
- 6 x 1 m^3/s duty and 2 standby pumps are proposed.
- The pumps in the model were simulated in a dry well mode. However, if submersible pumps are selected, a wet well mode is also possible.
- The pump installation could be phased if required. However, all duty and standby pumps are provided for in the pumphouse structure civil works.
- There are two hoppers with one jet pump in each.
- The bottom slopes of the hoppers are steep (1H:2V) to allow cohesive sediment to slide towards the jet pumps.
- The two hoppers are separated by a dividing wall between them equipped with gates which are normally closed.
- The two motive pumps for driving the two jet pumps are located next to the duty pump bays in the hopper compartments.
- The intake structure is located on the left bank side of the river.
- On the river side of the high curved wall of the intake structure and upstream of the weir a boulder trap with a 1:15 floor slope is provided.
- Next to the boulder trap, on the left bank side, a gravel trap with two canals are provided.
- The high intake wall between the boulder and gravel trap has a submerged opening without screens; the soffit of the opening is at the weir low notch crest level to keep floating debris out during floods.
- The submerged opening between the gravel trap and the hoppers are equipped with trashracks and designed for a flow velocity through the unblocked screen of 0.3 m/s. The trashracks can be raised for cleaning; vertical gates are installed at the trashrack openings which should be closed when the trashracks are raised or when the abstraction works is not operational.
- The trashrack should have vertical and horizontal bars with 50 mm x 50 mm openings. The flat bars should have a cross section dimensions of 10 mm thick x 50 mm long and could be streamlined. The openings are determined by the jet pump requirements.
- A Crump weir is proposed without energy dissipation, with the weir fixed on the bedrock. The bedrock is shallow at the left bank, but as deep as about 10.5 m at the right bank floodplain (Figure 5.2-2). DWS was consulted to review the weir crest levels, notch lengths, design head, dividing walls and flank wall designs for improved flow measurement accuracy.
- A combined fishway-canoe chute at the weir was designed with input and review by fishway specialist, Dr A Bok, as well as canoeists. The Campsdrift weir canoe chute on the Dusi River was used as basis for the design. The proposed design was also tested in a flume at a scale of 1:15 before being constructed in the 3D 1:40 scale physical model. The design of the fishway-canoe chute is discussed in Section 11. DWS Hydrology, flow gauging station division,

recommended that the fishway-canoe chute should be located between the first and second notch of the proposed Crump weir for accurate flow measurement, but there are also safety benefits not to have the fishway-canoe chute at the abstraction works near the left bank side of the river.

- The combined fishway-canoe chute was placed between the low Crump notch (51.6 masl) and the higher Crump notch (51.9 masl) away from the boulder trap and pumpstation intake structure. Placing the fishway-canoe chute between the low and high notches also reduces the number of dividing walls on the weir which are obstacles in the flow path that can accumulate debris.

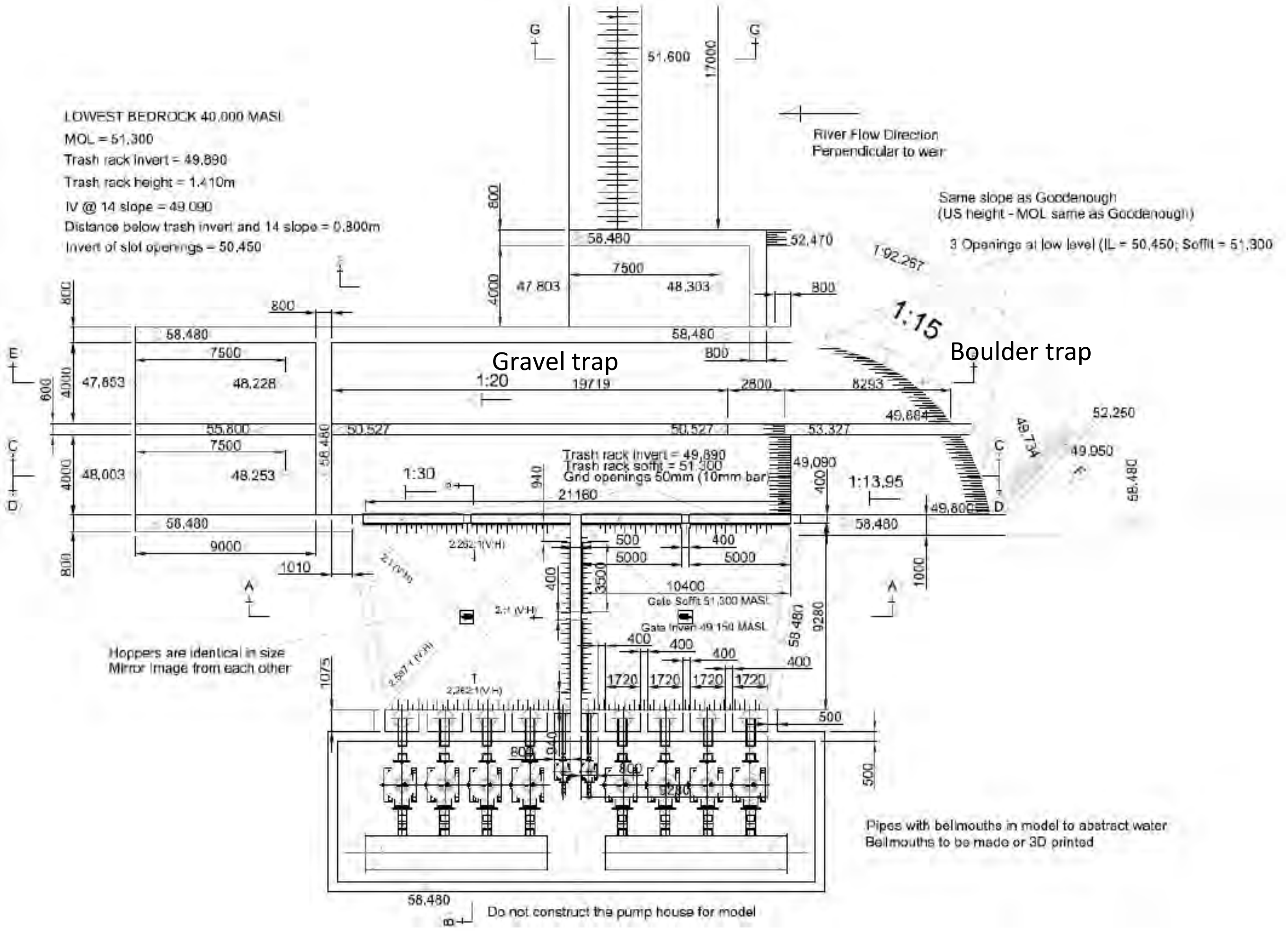


Figure 6-7: Plan layout of intake structure on left riverbank including the boulder trap and two gravel trap canals

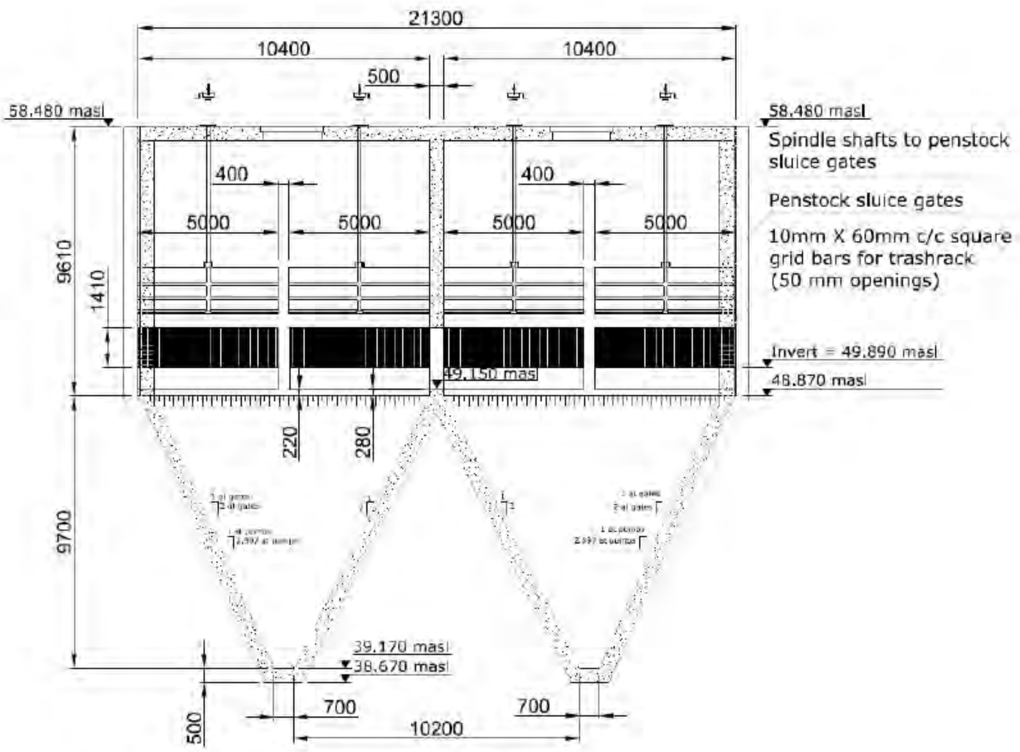


Figure 6-8: Section AA through the hoppers indicating trash racks

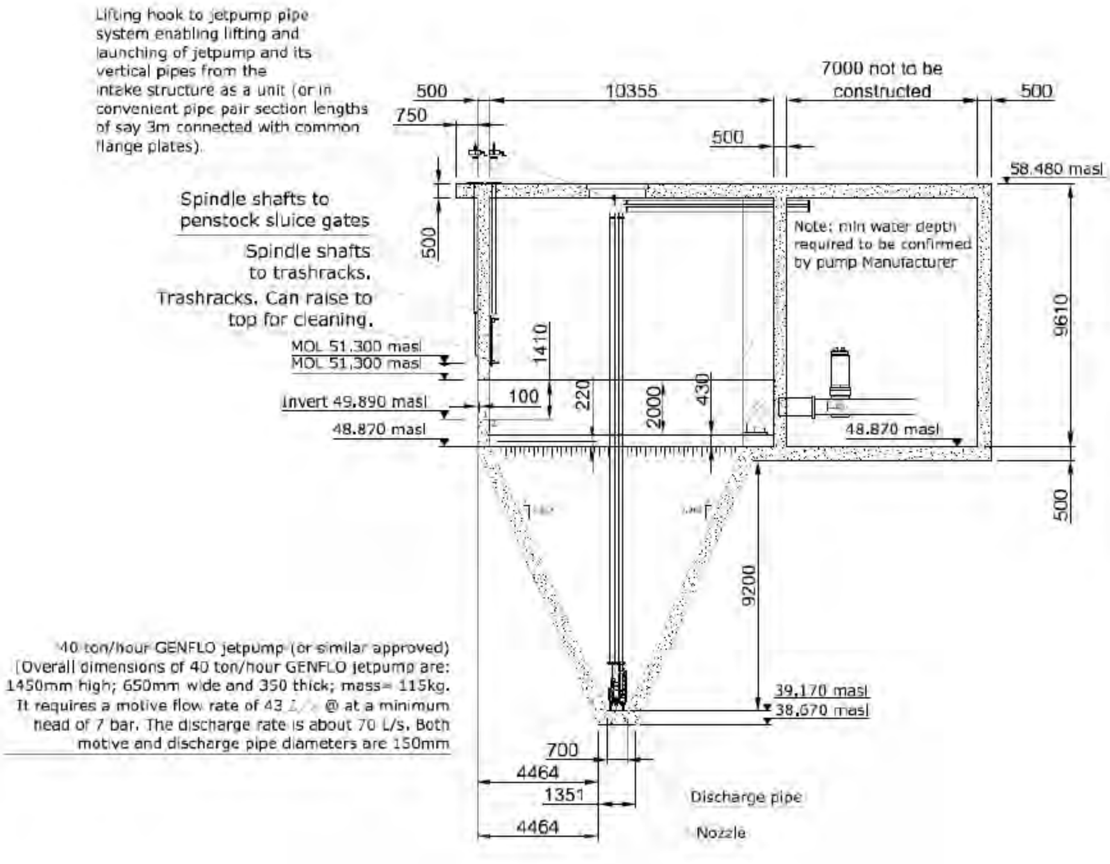


Figure 6-9: Section BB through the hoppers indicating duty and jet pump in hopper

7. Physical model test conditions and parameters for measurement

The flow conditions used in the model tests were for upstream river water levels between the lowest weir crest level of 51.3 masl and the 100-year flood level in steps of approximately 0.5 m to establish a stage-discharge or H-Q relationship for the weir. The water level of the RMFcc flood upstream of the weir was also measured in the model.

As indicated under the scope of the study (Section 2.2) the observations in the physical model included:

a) The following design aspects were optimized in the physical model:
   - The weir type (Crump as opposed to Ogee profile).
   - Weir length and crest heights of the overtopping sections as well as the right bank berm's non-overtopping crest height.
   - Damming on the upstream side of the weir and energy dissipation if needed on the downstream side of the weir.
   - Sediment deposition/erosion upstream and local scour downstream of the weir by providing for movable bed areas and bedrock in the relevant area of the model.

b) Optimum orientation of the proposed abstraction works relative to the weir was determined to ensure effective self-cleaning of the boulder trap during floods.

c) Flow patterns in the abstraction works at peak pumping rate under normal operating conditions were observed and recorded.

d) Evaluation of the sediment control and flushing efficiency of the sediment traps at the abstraction works (a low weir and high tailwater levels could lead to poor sediment flushing during small floods).

e) Design and testing of a proposed combined fishway-canoe chute (1:15 scale model for optimisation purposes followed by 1:40 scale model of the optimised chute) including evaluation of its optimal location.

f) Measurements of the water levels at the right bank berm and design and test suitable erosion protection as required.

g) Evaluate flood levels and flood lines and the required top of the abstraction works elevation.

h) Establishment of the H-Q relation of the weir.

i) Determining the hydraulic head for different river flow rates across the weir available for effective flushing of the different components of the abstraction works.

j) Recommendations for the temporary works design.

Tailwater levels for the downstream boundary of the physical model were obtained from hydrodynamic modelling of the updated survey. The tailwater level was controlled at the following coordinates X: -1879.413 Y: -3688996.597 approximately 135 m downstream of the proposed BRVAS abstraction works site. The tailwater levels for the different recurrence intervals are summarized in Table 7-1.

Table 7-1: Simulated tailwater levels (TWL) for different discharges (Q) and recurrence intervals (RI) based on the updated 2021 survey

RI (years)					**2**	**5**	**10**	**20**	**50**	**100**	**200**	**RMF**
Q (m^3/s)	0	10	50	100	210	424	613	830	1169	1468	1808	4494
TWL (masl)	46.75	48.36	49.68	50.52	51.71	53.23	54.13	54.94	55.91	56.37	56.74	58.56

8. Laboratory instrumentation and measuring methods

Recording methods and instrumentation that were used in the model study to record relevant parameters such as flow rate and water levels are presented in this section.

8.1 Flow rate

Flow rate was controlled with constant head supply tanks and isolating valves on the supply pipe to the model which were recorded with an electro-magnetic flow meter. Water levels and flow rates were continuously monitored during the tests and sufficient time was allowed for flow and approach water levels to stabilize before measurements were recorded for a specific discharge rate.

8.2 Water level

To establish the weir-head the approach water levels were recorded at needle locations upstream of each weir notch (approximately four times the design head upstream of the weir as required by DWS for prototype river discharge measurement). The water levels were recorded with needle point gauges with an estimated accuracy of ±1 mm in the model. The levels in a stagnant area near the right bank berm and tail water level in the river were also recorded with needle gauges.

8.3 Visual observations

Photographic and video recordings were used to capture flow phenomena such as possible flow patterns and instabilities at the abstraction works, weir, and in the river. Visual recordings were made of the sediment flushing of the traps. Dye was used to visualize the flow patterns at the weir and structure and to obtain an approximation for the flow velocities.

8.4 Model recording/testing procedure

To establish the performance of the abstraction works and weir, the following procedure was followed:

a) For each test condition the desired flow rate and tailwater (i.e. boundary conditions) recording instruments were set and flow through the model was monitored to establish when the flow had stabilised before the required model parameters were recorded. Flow rates were checked during and after all recordings to ensure a constant flow during the recording period.
b) After the flow of a specific test condition had stabilized all the relevant recordings listed under Section 7 were performed.
c) In the case of movable bed tests, the model was run until the movable bed changes had stabilised after which the model was slowly drained and the changes then surveyed.

9. Physical model test results and discussion

9.1 Preliminary tests for the current scenario

As indicated previously, the physical model was initially constructed based on the 2012 survey data (with limited underwater survey data). Tests were carried out initially without the proposed weir and abstraction works to evaluate the flow patterns near the site. It was clear that the site is not ideal because of the relatively low flow velocities due to the low river slope and relatively wide floodplain flow occurs for floods larger than the 2 year flood (see Figures 9-1 and 9-2). From the tests the abstraction site was selected in the model near the bend in the river where the bedrock is known to be shallow, the left bank is relatively steep (enabling the shortening of the left bank flank wall length), and the main channel is well defined without tree blockages locally.

Figure 9-1: Photograph of the physical model of the natural BRVAS site for the Q2cc (210 m^3/s)

Figure 9-2: Photograph of the physical model of the natural BRVAS for the Q10cc (613 m^3/s)

9.2 Fixed bed tests with the initially proposed weir and abstraction works

The final new 2021 topographical survey and underwater survey data was received on 21 June 2021 and the physical model main channel was completely reconstructed when the model weir and abstraction works were constructed in the model. The layout of the weir and abstraction works based on the 2012 feasibility design concept is shown in Figure 9-3.

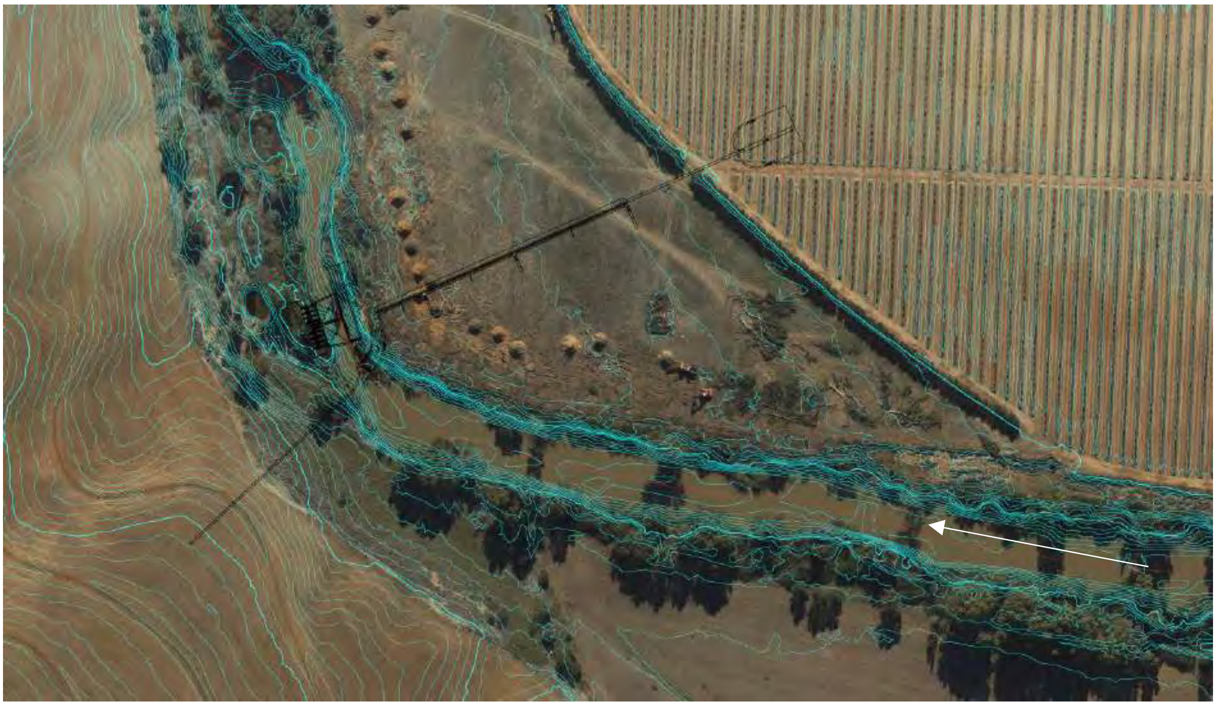


Figure 9-3: Plan layout of the proposed BRVAS abstraction works and weir shown on the 2021 topographical survey and based on the 2012 feasibility study design concept

The Crump weir was designed to improve flow measurement accuracy based on the DWS guidelines following a meeting between SU and DWS on 9 June 2021 and the key design considerations were:

a) DWS considered the long term flow record of the DWS Hermon flow gauging station located upstream of the BRVAS site and in order to measure 80% of the MAR at the BRVAS site two notches of 20 m (left bank side) and 40 m in length are required, but a third was added of 50 m length on the right bank side to limit damming upstream during extreme floods. A fourth notch on the right bank side could be 50 m long but could be a broad crested weir, while the three other notches closer to the left bank are truncated Crump weirs.
b) The three left bank side notches of the Crump weir should have crest elevations which differ by only 0.3 m for accurate flow measurement. The selected crest levels of these weirs from left to right were 50.1 masl, 50.4 masl and 50.7 masl. The fourth weir notch on the right bank side which is not used for flow measurement has a crest level of 54.0 masl. With these weir crest levels the floodplain upstream and locally downstream of the weir has to be excavated to open the weir crest for flow measurement since the NGL is at about 52 masl near the second and third notches.
c) The weir height at the low notch above the river bed level of 46.5 masl (2021 survey) is 3.6 m. The observed low flow water level during the May 2021 underwater survey of the river at the weir site was 47.7 masl at 1.2 m^3/s; the drop from the low notch crest elevation to the relatively low river flow water level downstream is therefore 2.4 m. The observed water level was also used to calibrate the numerical hydrodynamic model and it was found that the 2021

survey based tailwater levels are as much as 3 m higher during low flows and small floods than what was previously estimated. This can be attributed to a longer survey carried out during 2021 downstream of the abstraction site, a detailed underwater survey and the damming effects of tree blockages in the river.

d) The Crump weir has a design head of 1.2 m. The truncated weir length in the flow direction is therefore 3.6 m.
e) Dividing walls are located between the notches of the weir. These concrete walls are 7.2 m long upstream of the weir crest, 1.0 m thick, with upstream ends 45 degrees to the vertical to limit floating debris accumulation. The skew wall turns into a vertical wall at the same elevation as the 1:2 (V:H) Crump weir lowest elevation upstream. The dividing walls improve flow measurement accuracy and helps to scour the sediment from upstream of the weir during floods.
f) The weir should be founded on solid rock. Based on the geotechnical information available (DWS, 2012), the left bank bedrock elevation is at 44 masl, while on the right bank side the deepest bedrock level is at 40 masl.
g) The berm on the right bank side of the floodplain was made sufficiently high in the model to prevent overtopping by the Safety Evaluation Flood (SEF) = RMFcc.

Model tests were carried out from low flows to floods with the weir (as described above) and abstraction works and the key findings are:

- The relatively high tailwater levels and the long length of the weir of 160 m caused relatively small flow velocities (< 1 m/s) near the left bank side at the intake, which prevents self-scouring of the intake zone.
- The highest flow velocities during floods were also observed near the right bank side of the weir and not near the left bank side at the abstraction works. This was due to the upstream bend in the river and the wide floodplain flow towards the right bank berm.
- The gravel and boulder trap could only be flushed at relatively small river discharges up to 210 m^3/s, before becoming submerged during the 2-year flood due to the relatively high tailwater levels that made it impossible to flush by opening the gates. This is demonstrated in Figure 9-4 by the sediment remaining in the 2 gravel trap canals after attempting to flush during the 2-year flood.

Figures 9-5 to 9-8 show some photographs of the model tests of the above model setup.

Figure 9-4: Flushing limit of the two gravel trap canals after the Q2cc (210 m^3/s)

Figure 9-5: 1:40 scale model weir with 4 notches, abstraction works and berm during the Q5cc flood (424 m^3/s) as viewed from the right bank side

Figure 9-6: Weir with 4 notches, abstraction works and berm during the Q5cc flood (424 m^3/s) as viewed from downstream

Figure 9-7: Weir with 4 notches, abstraction works and berm during the RMF (4 494 m^3/s) as viewed from the right bank side

Figure 9-8: Weir with 4 notches, abstraction works and berm during the RMF (4 494 m^3/s) as viewed from downstream

9.3 Fixed bed tests – Option A

Based on the findings in Section 9.2, the following design modifications were tested in the laboratory:

a) The originally right bank floodplain berm was removed and replaced with a long right bank side near river berm (levee) to guide the wide floodplain flow towards the left bank intake. This is the same approach followed at the Berg River Dam Supplement Scheme completed in 2008 near the Drakenstein Prison, where the riprap berms were designed for the 50-year flood, allowing larger floods to spill over the berms and flow away freely past the abstraction weir as under current conditions and thereby limiting the extreme flood damming on properties. By the time floods > the Q50cc flood spills over the berms the flood backwater effect from downstream of the weir has raised the water levels so much at the berms that the outer floodplains are inundated and the berm spillage will fall into the inundated floodplain.
b) Near the weir the left and right bank side berms have to transition into vertical concrete walls where the flow velocities are high and to better guide the flood flow around the bend. The walls lengths will be optimized during the movable bed tests.
c) A shorter left bank side berm was also added in the model to force the bend effect with higher flow velocities near the intake for self-scouring.
d) The weir was shortened to 60 m length in total with a 20 m low notch and 40 m right bank side notch. The shorter weir increased the self-scouring flow velocities at the intake while the Q50cc flood additional damming compared to the current scenario was limited.
e) The weir crest was raised to improve the submergence problems, which caused ineffective small flood flushing of the sediment traps. The raised crest levels are 51.6 masl and 51.9 masl for the 20 m and 40 m notches respectively. At the design head of the weir the discharge

capacity is 123 m^3/s and the weir submergence is not affecting the discharge measurement. With the raised weir levels the right bank side of the floodplain has to be excavated about 1 m deep to elevation 51.0 masl to open the weir crest for flow measurement.

The plan layout of the above modifications (**Option A**) is shown in Figure 9-9**.** The model tests with the above design confirmed a significant improvement in the secondary currents and self-scouring of the intake, with relatively high flow velocities near the left bank berm during the floods up to the Q50cc.

Figure 9-9: Option A plan layout of the weir, abstraction works and berms/walls

Two additional hydraulic design options were also evaluated:

- **Option B** which is located 50 m downstream of option A (option A weir site is the feasibility study hydraulic design weir site), similar in design to option A, with a longer bend created by concrete walls/berms to improve the secondary currents further (Figure 9-10). This is also the weir site which was investigated during the feasibility study's geotechnical investigation. Option B had to be considered because TCTA considered the Option A design with berms along the river a fatal flaw for the project due to the expected delay of 2 years for the environmental approval of Option A.
- **Option C** which has a similar chainage along the river as option B, but is located more to the left bank side on the main channel (Figure 9-11). At this site the fishway-canoe chute outlet is also more aligned with the downstream river channel (the proposed combined fishway-canoe chute is located between the low Crump notch and the second Crump weir notch, 17 m from the left bank side of the weir, as recommended by DWS for improved flow measurement.

Of the three sites option B was found hydraulically to be the best site, with high flow velocities near the intake and the secondary currents developed well for self-scouring of the intake. This was expected since option B has a longer bend to develop the secondary currents better compared to option A. Option C is not nearly as good as either options A or B, with flow velocities < 1 m/s near the left bank side wall during the Q50cc flood. Photographs of the model study tests with bricks used as berms along the river for option A as discussed above are shown in Figure 9-12 to 9-15.

Figure 9-10: Option B plan layout of the weir, abstraction works and berms/walls

Figure 9-11: Option C plan layout of the weir, abstraction works and berms/walls

Figure 9-12: Option A model study test to optimize the berm locations at Q2cc (210 m^3/s) and a weir length of 60 m viewed from downstream

Figure 9-13: Photograph of option A physical model study showing optimized berm locations and weir design for the Q50cc (1 169 m^3/s) viewed from downstream

Figure 9-14: Photograph of option A physical model study with improved flow patterns for the new weir and berm layout during the Q10cc (613 m³/s) viewed from upstream

Figure 9-15: Option A model study test showing the self-scour abilities of the improved berm and weir design (without opening the boulder gate) during the Q10cc flood (613 m³/s) viewed from upstream

9.4 Fixed bed tests – Option B2

The berms and walls that were proposed and tested in Section 9.3 were not part of the approved EIA for the abstraction works. Obtaining approval for the berms/walls would have resulted in a 2-year delay in the project. Option B was adapted to conform to the existing approved EIA which consists of the abstraction works and 60 m long weir at the 2012 feasibility weir site of the geotechnical investigation with an additional higher broad-crested weir, 100 m in length, on the right bank side of the floodplain. Additionally, two high walls were added to guide the flood discharge around the bend towards the abstraction works.

The layout of the second scenario for option B is shown in Figure 9-16 with the additional guide walls and a cross-section of the weir and abstraction works is shown in Figure 9-17. The left bank wall is a local modification of the 2012 design which was also within 32 m of the main channel and is fixed on the shallow bedrock in the area. The right bank guide wall is on the floodplain and was placed more than 32 m away from the main channel of the river.

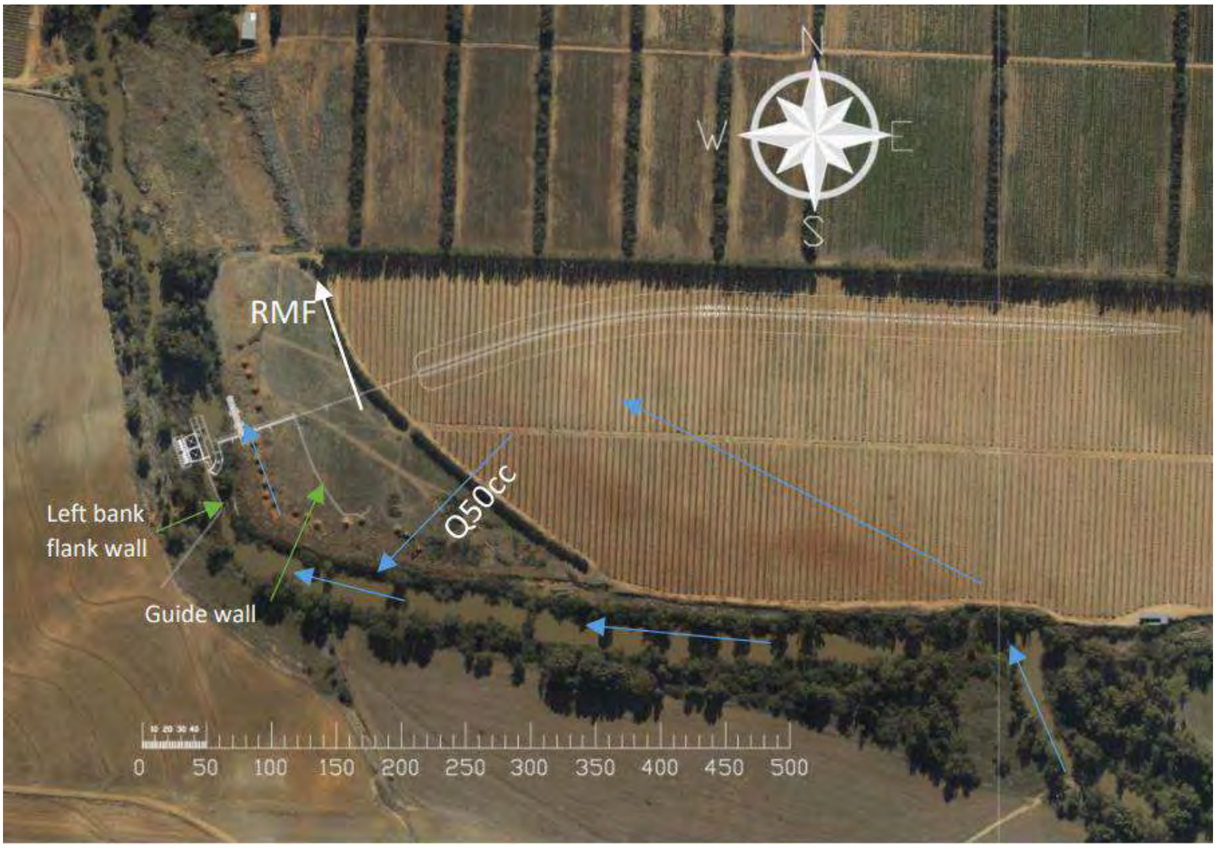


Figure 9-16: Option B2 plan layout of the weir, abstraction works, berm and proposed dividing wall (right bank) and flank wall (left bank)

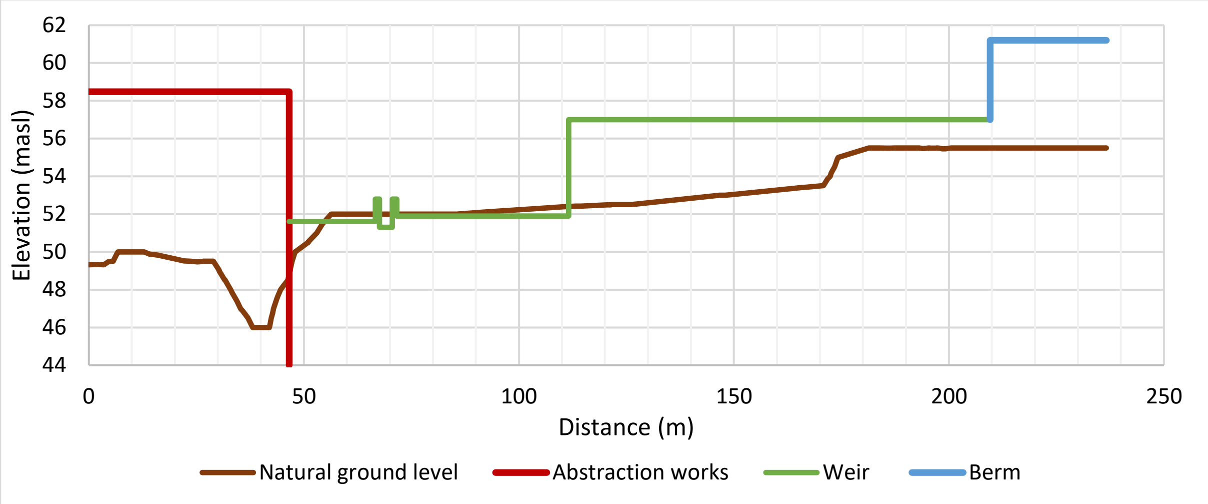


Figure 9-17 Option B2 cross-section of the weir, abstraction works and berm

The design considerations for option B2 were as follows:

a) Adhering to the approved EIA by placing the weir and abstraction works in the location determined in the 2012 feasibility study and by adding the necessary guide walls more than 32 m away from the main channel of the river, where possible.
b) Sufficient self-scouring of the intake during medium to large floods was ensured by the addition of the left bank flank wall and right bank dividing wall to maximize the bend effect and secondary currents.
c) Minimal increase in damming above the Q50cc flood with the addition of the higher, 100 m broad-crest weir at the Q50cc flood level.
d) Right bank protection during large floods by the berm up to the RMF flood level, in the event where the RMFcc flood peak is exceeded the water will spill over the berm onto the inundated floodplain downstream.

Option B2 was tested with a fixed bed as shown in Figures 9-18 to 9-25, the changes in the design were as follows:

- 17 m low notch Crump-weir at 51.6 masl.
- 3 m fishway-canoe chute between the low notch and right bank notch of the Crump weir at an elevation of 51.3 masl to ensure that low flow passes through the fishway-canoe chute first before starting to spill over the low notch.
- The fishway-canoe chute is also capable of measuring flow with a Crump weir crest.
- 40 m right bank Crump-weir at 51.9 masl.
- The design head for the flow measuring structures are 52.8 masl (design discharge = 123 m^3/s).
- The dividing walls between the low-notch, fishway-canoe chute and right bank notch has an elevation of 52.8 masl.
- 100 m broad-crested weir and right bank guide wall at the Q50cc level of 57.0 masl.
- Intake soffit of the abstraction works = 51.3 masl.
- Left bank flank wall to be cut to the Q100cc observed maximum flood level.
- Right bank berm to be cut to the observed maximum RMFcc flood level.

Observations from the fixed bed tests indicated that option B2 is a good compromise to conform to the existing EIA. The key findings from the fixed bed tests for option B2 were as follows:

- The flow against the abstraction works shows self-scouring currents for flows above the Q10cc (613 m³/s).
- With the raised weir and abstraction works the gravel and boulder traps can be effectively flushed up to the Q50cc (1169 m³/s) year flood.
- Observed flow velocities between the right bank guide wall and abstraction works are high for the Q50cc (1169 m³/s), in the order of 2.2 m/s which will facilitate self-scouring of the intake.
- The guide wall on the right bank is needed to guide the flow toward the abstraction works, as shown in Figure 9-24, the flood plain is relatively low and starts flowing at Q2cc (210 m³/s). Without the guide wall the flow velocity against the left bank would be low preventing secondary currents from scouring the intake area.
- The broad crest weir and guide wall did not spill for Q50cc (1169 m³/s) at 57.0 masl.
- The abstraction works and left flank wall top of the structure elevation were determined to be 58.5 masl including 0.5 m freeboard. This is the maximum water level observed against the structure for the Q100cc (1468 m³/s). Above the Q100cc water will be able to spill over the structure and around the left flank wall to minimize the additional damming caused by the abstraction works, weir and berm.
- The elevation of the right bank berm crest was determined as 61.2 masl from the physical model tests for the RMFcc flood; above this level the berm will spill onto the downstream floodplains which are inundated with the downstream water level reaching the berm. This elevation corresponds with the RMFcc which is considered the Safety Evaluation Flood for the berm to limit risk based on SANCOLD freeboard guidelines and as recommended by AEJV (2021).

The fixed bed tests were effectively able to optimize the design of the abstraction works. Note that the dimensions and layout of the design were updated in this study and are presented in Section 6 and **Appendix A**. A brief comparison of the 2012 and 2021 designs is also given in Table 9-1.

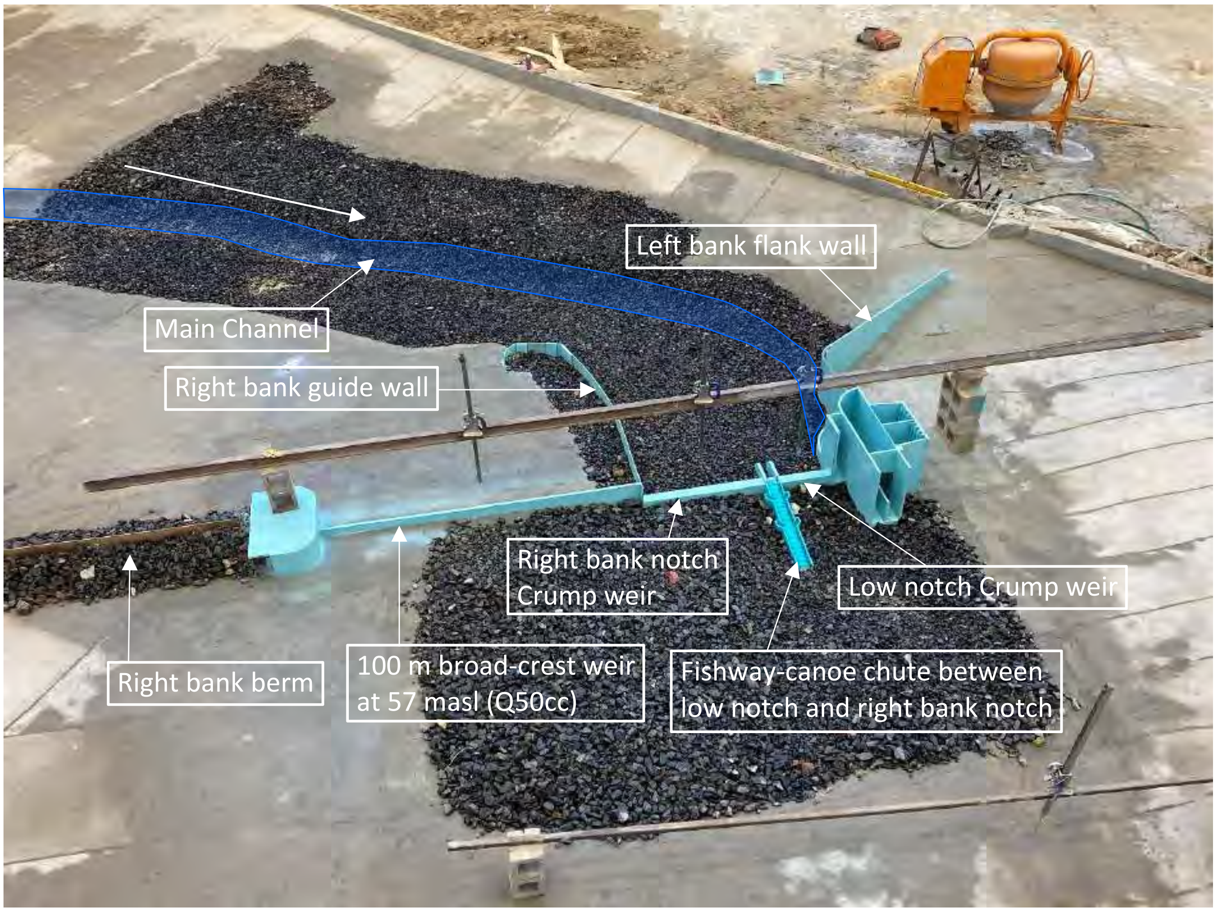


Figure 9-18: Fixed bed tests, Option B2, gravel used to obtain surveyed bed level before the movable bed sediment is introduced into the model

Figure 9-19: Q50cc (1169 m³/s) guide wall diverts flow towards the 60 m weir and abstraction works

Figure 9-20: Flow is guided by the right bank guide wall and flows over the weir perpendicular to the weir Q50cc

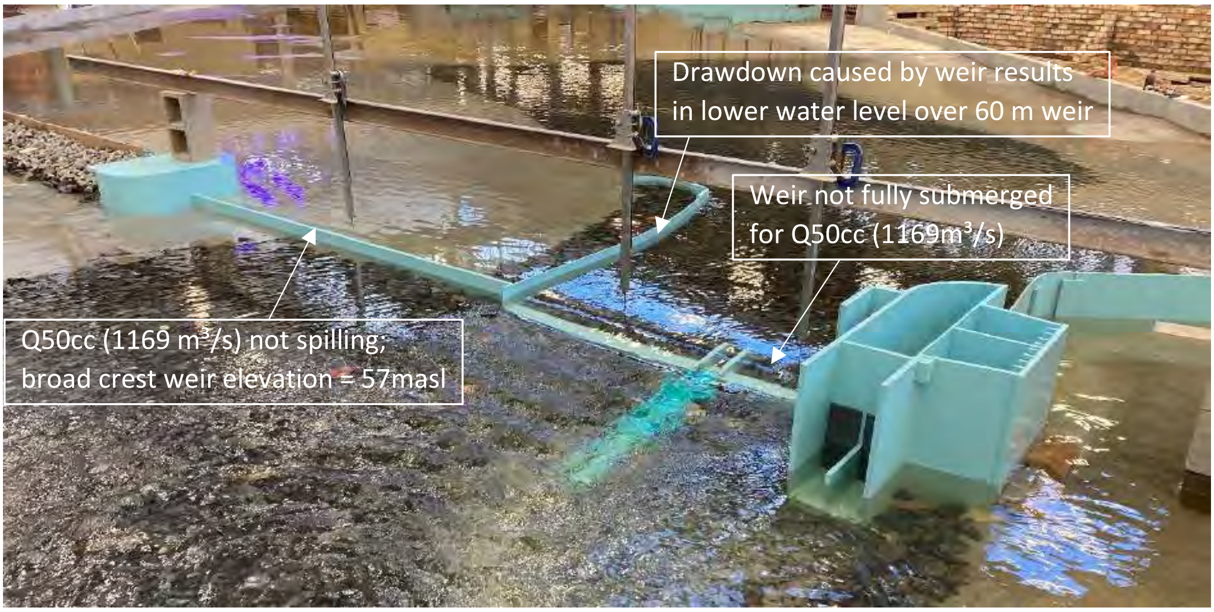


Figure 9-21: The 100 m broad crest weir and guide wall is not spilling for the Q50cc (1169 m³/s) flood peak

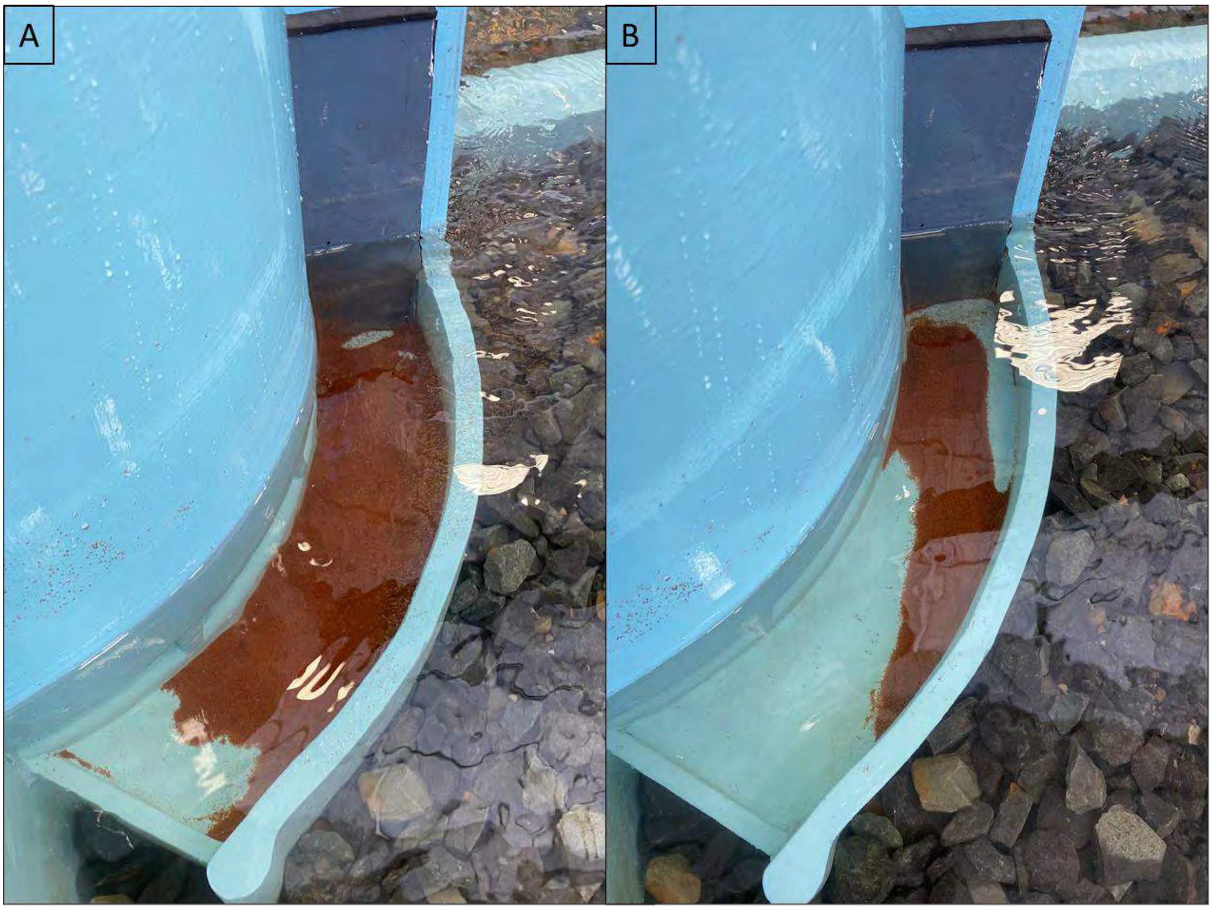


Figure 9-20: Evidence of self-scouring of the boulder trap at Q5cc (424 m³/s) confirms active secondary current at abstraction works intake

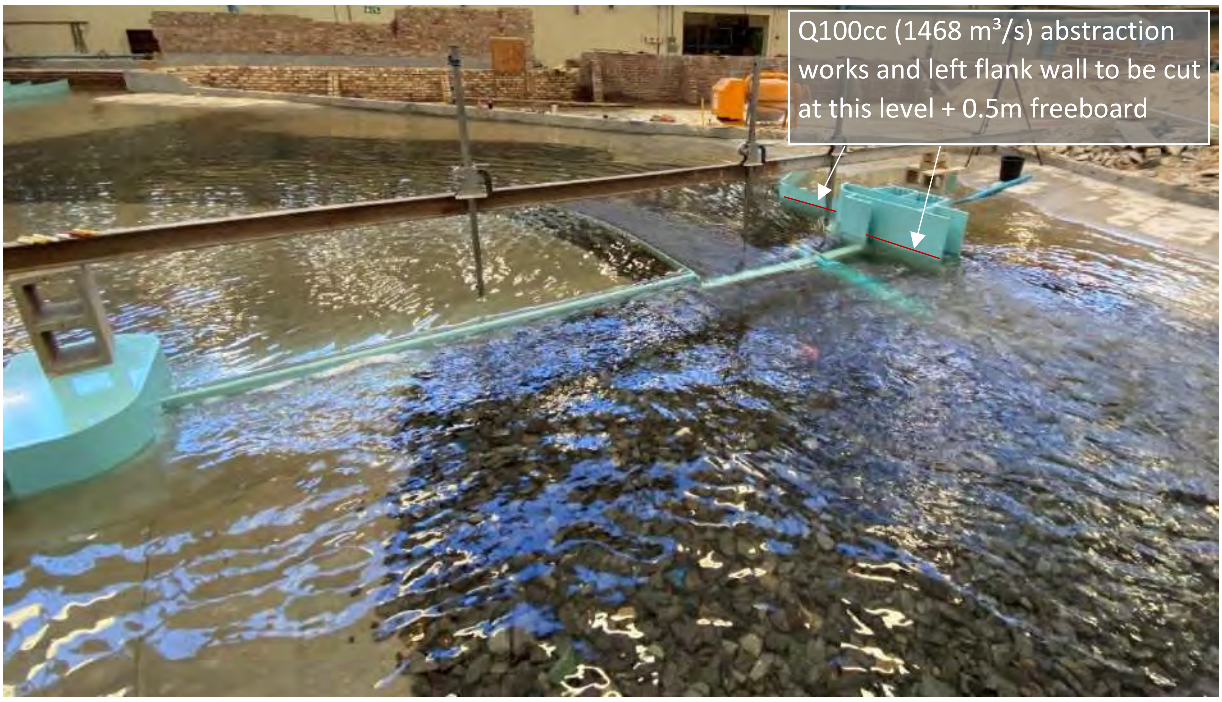


Figure 9-21: Q100cc (1468 m³/s) flow spills over the broad crest weir and over guide wall from right to left into the main channel

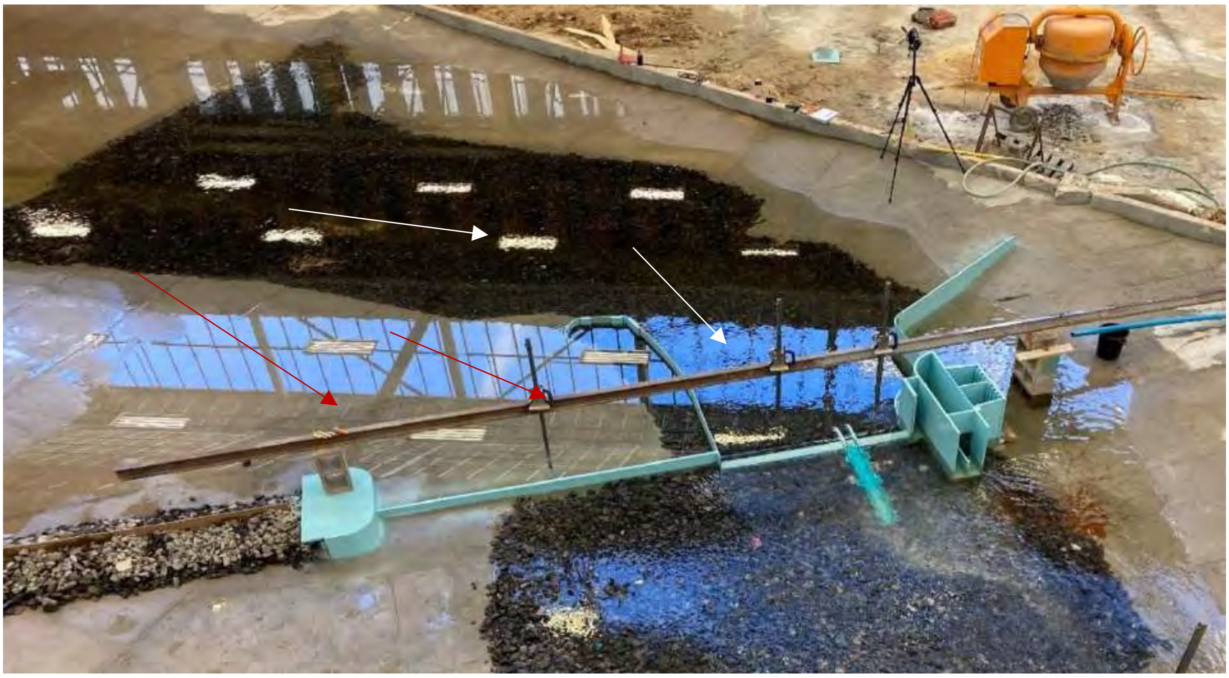


Figure 9-22: Q5cc (424 m³/s) guide wall is required to prevent flow from flowing over floodplain (red arrows), bypassing the abstraction works, guide wall redirects flow (white arrows) and aligns the flow with the abstraction works and weir

Figure 9-23: Gravel traps are flushed at Q10cc (613 m³/s). Figure A shows gravel traps before flushing and Figure B shows gravel trap during flushing with both sluice gates open

Table 9-1: Summary of dimensions of preliminary abstraction works design (ASP, 2012) compared to updated dimensions from this 2021 study

Design summary	Units	ASP (2012)	Updated in this study (2021)
Crump weir			
Low notch length	m	20	3 (Crump fishway-canoe chute)
2nd notch length	m	40	17 (low notch Crump)
3rd notch length	m	50	40 (high notch Crump)
4th notch length (broad crested)	m	50	100 (broad crested)
Low notch crest elevation (Fishway-canoe chute)	masl	47.9 (MOL)	51.3
2nd notch crest elevation	masl	50.4	51.6 (MOL)
3rd notch crest elevation	masl	52.4	51.9
4th notch crest elevation	masl	54	57.0
Lowest river bed elevation at site	masl	44.92	47.0
Low notch height above river bed	m	2.98	4.3
Discharge capacity low notch only, where Ha = Design head for each notch required for flow measurement based on DWS guidelines	m^3/s	71	0.98 for fishway (Ha = 0.3 m) 8.30 for low notch (Ha = 0.6 m) 123 for all notches (Ha = 1.5 m)
Abstraction inlet between boulder and gravel traps			
Opening length	m	15.4	13.8
Opening height	m	0.85	0.85
Opening invert level	masl	47.113	50.450
Trashrack			
Trashrack length	m	30	20
Trashrack minimum height required	m	0.774	1.410
Trashrack invert level	masl	47.127	49.890
Pump bays and hoppers			
Number of pump bays	#	4	8 (+2 for motive jet pumps)
Width of pump bays	m	2.6	1.72 (0.8 for motive jet pumps)
Number of duty pumps	#	3	6
Number of standby pumps	#	1	2
Total duty pump capacity	m^3/s	6	6 (maximum)
Total standby pump capacity	m^3/s	2	2
Number of hoppers	#	4	2
Max width of hoppers	m	7.5	10.4
Hopper invert levels	masl	35.123	39.170
Min volume required (100Q)	m^3	600	600
Max volume required (200Q)	m^3	1200	1200
Volume provided	m^3	970	870
Sediment traps			
Boulder trap (x1) width	m	4	4
Gravel trap (x2) width	m	4	4
Radial gate opening height (R = 6 m)	m	4	4
Min floor level downstream of traps	masl	45.223	47.803
Structure			
Top of structure i.e. Q100cc	masl	56.514	58.480 (includes 0.5 m freeboard)
Right bank side berm crest designed for RMF = SED without freeboard	masl	61.0	61.2

9.5 Movable bed tests – Option B2

Moveable bed tests were subsequently carried out for option B2. The purpose of the movable bed tests was to evaluate the scour patterns during different floods to establish whether the abstraction works will be able to scour the intakes, to ensure that the main channel does not move away from the left bank and to determine the downstream effects on bed movement with the abstraction works and weir in place. A section upstream of the structure was excavated to 4 m below the natural ground level obtained from the 2021 survey and downstream of the structure was excavated to the bedrock level and filled back up with sediment representative of the in situ sediment based on the grading analysis described in Section 4.2. Figure 9.26 shows the area filled with the moveable bed material (crushed graded peach pips).

Figure 9-24: Movable bed setup with sediment shaped to the 2021 survey of the river

The movable bed tests were carried out in sequence from 10 m³/s up to 1468 m³/s (Q100cc). Figures 9-27 to 9-35 show the observations from the tests. The following were the key findings from the tests:

- Scour of the toe of the weir to bedrock happens at low flows (Q = 10 m³/s – lowest tested) and a sand bar forms just downstream of the scour hole as shown in Figure-9-27. The fishway-canoe chute scours downstream which is beneficial for both fish and canoeists to navigate the chute safely.
- At 50 m³/s local scour around the dividing walls of the Crump weir was observed (Figure 9-28). The scour will ensure that the fishway-canoe chute approach remains accessible.
- Significant local scour and general sediment movement was visible for the Q2cc (210 m³/s), the scour is mainly focussed on the left-bank side of the weir and toward the fishway-canoe chute. Scour at a low recurrence interval flood is beneficial for the potential scouring of the intake near the boulder trap.
- For the movable bed tests, self-scour of the boulder trap was not as effective as shown in the tests with the fixed bed (Figure 9-29). Figure 9-30 shows self-scour for the Q5cc (424 m³/s), but the scour shown just upstream of the boulder trap increases the flow depth in the vicinity which in turn reduces the velocity and secondary currents. The boulder trap should still self-scour at this flood peak, but it can also be flushed effectively at this flow.
- A deflection caused by the abstraction works is visible on the Crump weir in Figure 9-31, this deflection may cause fluctuations in the flow measured over the low notch, but this is at a flood above the discharge table limit of the weir. Furthermore, deposition on the inside bend, near the right-hand side guide wall, may affect the flow measurements.

- The scour patterns of the sediment upstream of the weir was parallel to the intake of the abstraction works, this indicates that the flow exhibits secondary currents on the outside of the bend which promotes scour on the left bank.
- Scour around the upstream curve of the right bank guide wall have been observed as shown in Figure 9-33. The guide wall will need to be sufficiently protected against scour in this area or constructed on bedrock, similar to the weir and abstraction works.

Figure 9-25: At low flow the toe of the weir is scoured to bedrock with deposition just downstream of the scour hole formed (Q = 10 m³/s)

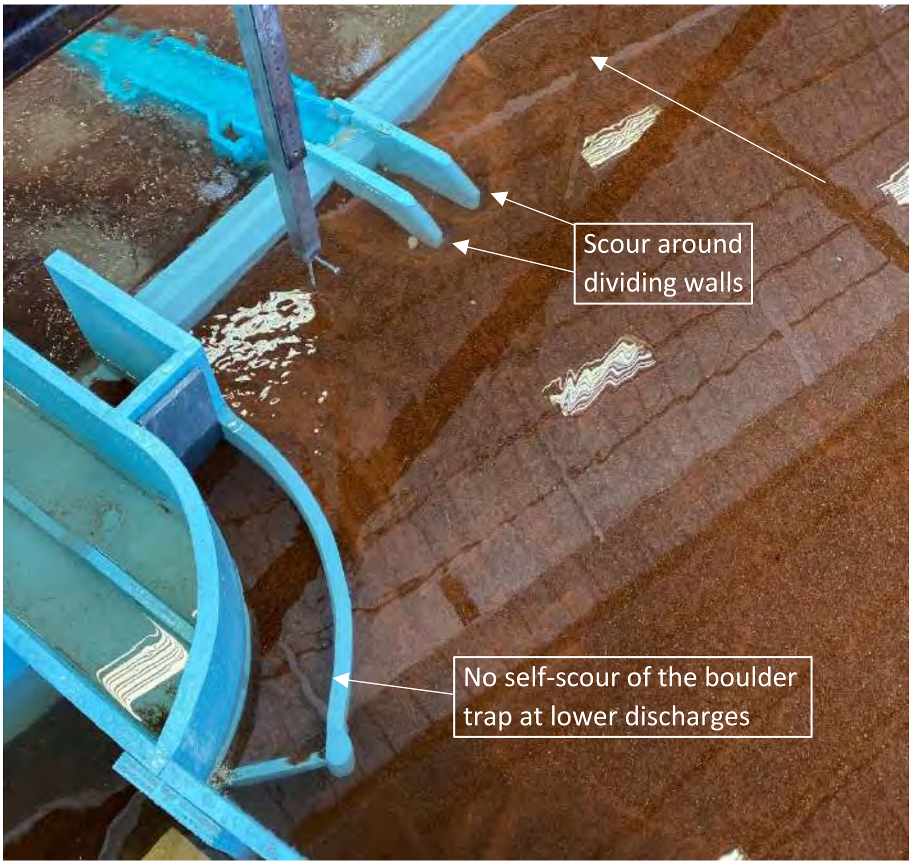


Figure 9-26: Scour around dividing walls of the Crump weir forming at Q = 50 m³/s

Figure 9-27: Upstream sediment transport at Q2cc (210 m³/s) and significant scour upstream, against the weir

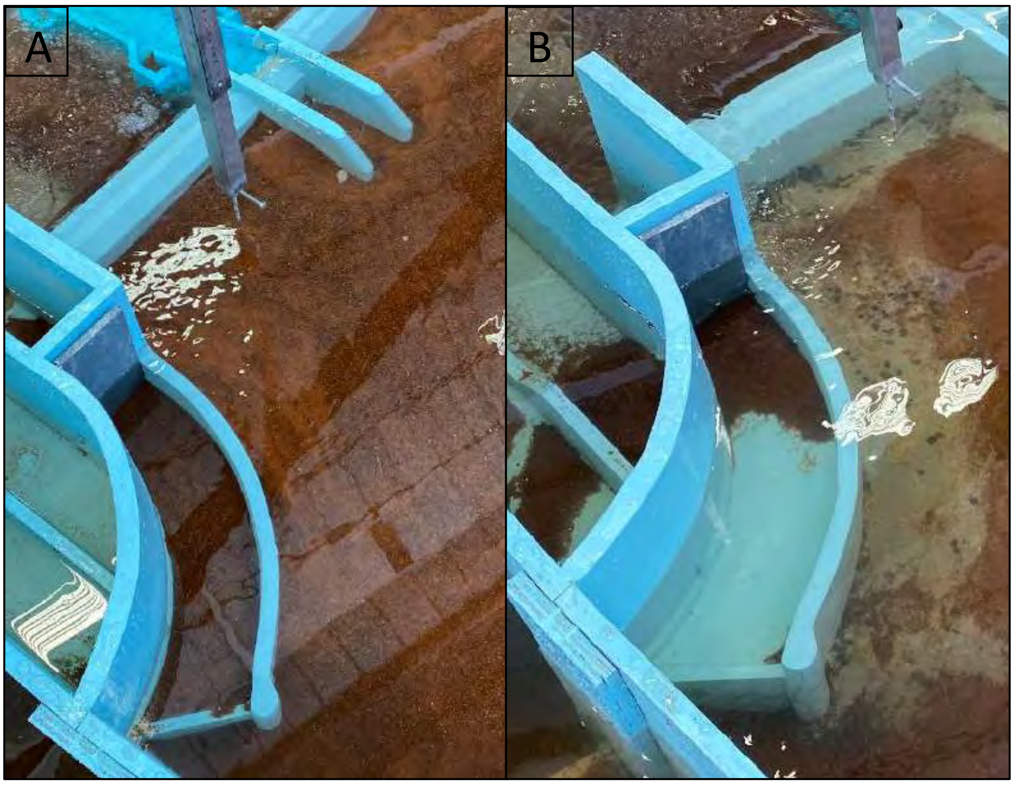


Figure 9-30: No self-scour evident at Q2cc (210 m³/s) in Figure A but self-scour of the boulder trap at Q5cc (424 m³/s) is evident in Figure B

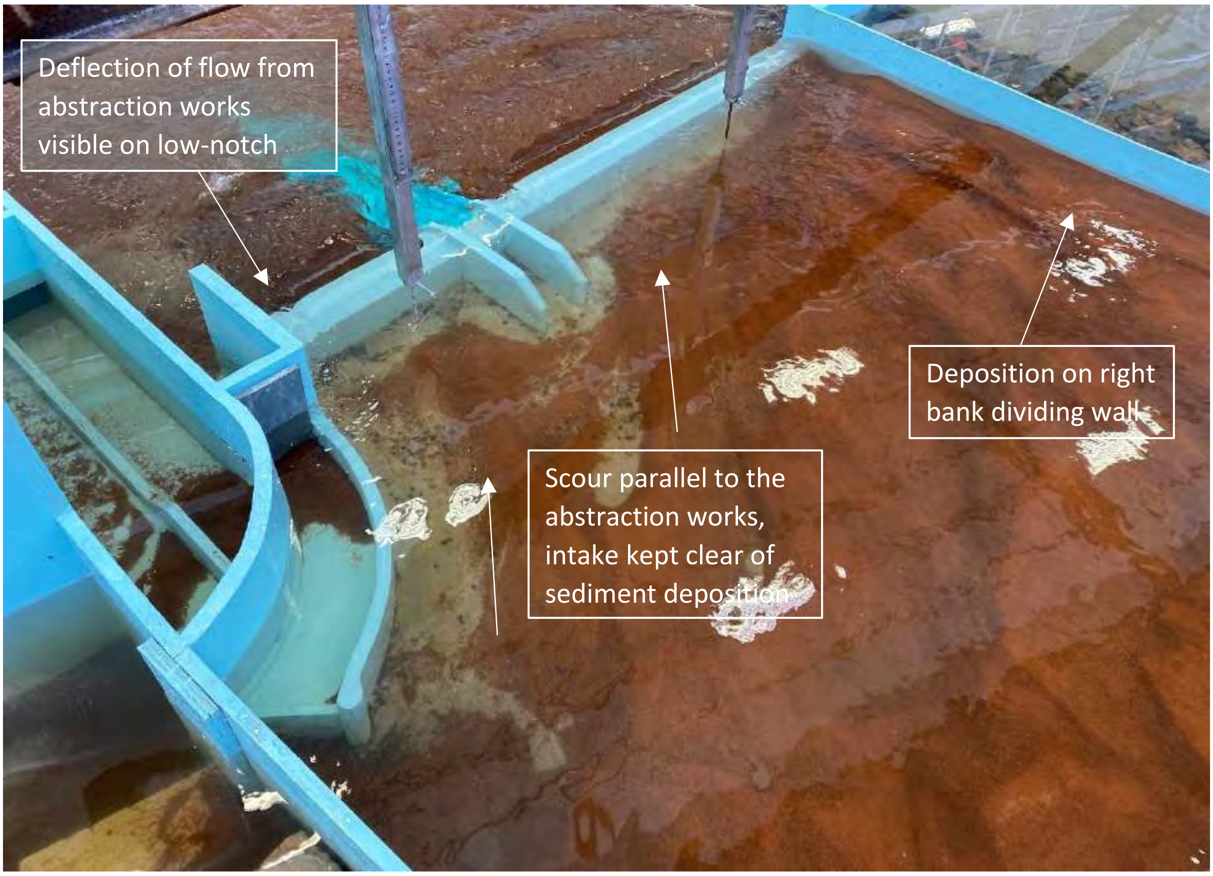


Figure 9-31: Q5cc (424 m³/s) scour on left bank favourable for intake works and deposition observed on right bank guide wall

Figure 9-32: Bed changes downstream of the weir, a new main channel forms in line with the weir for the Q20cc (830 m³/s)

Figure 9-33: Upstream view of riverbed after Q50cc (1169 m³/s) showing areas of deposition, scour and no sediment transport

Figure 9-34: Downstream view of Q100cc (1468 m³/s), left bank flank wall with abstraction works visible just above water surface, right bank is spilling over the 100 m broad crest weir

Figure 9-35: Upstream view of Q100cc (1468 m³/s) flowing over partially submerged 60 m Crump weir

9.6 Scour sections surveyed during movable bed tests

Figures 9-36 to 9-38 show the surveyed bed elevations after the Q10cc, Q50cc and Q100cc tests were completed. Figure 9-36 is for a section 12 m upstream of the weir, Figure 9-37 is for a section 30 m upstream of the weir and Figure 9-38 is for a section 35.6 m downstream of the weir. Locations where the bed had scoured onto the fixed bed are marked with red. The bed upstream of the weir is scoured from left to right between the dividing walls as the flood peak increases, this is a favourable result which will ensure that the outside of the bend near the intake will be scoured first. The shift in the main channel downstream of the weir is evident in Figure 9-38.

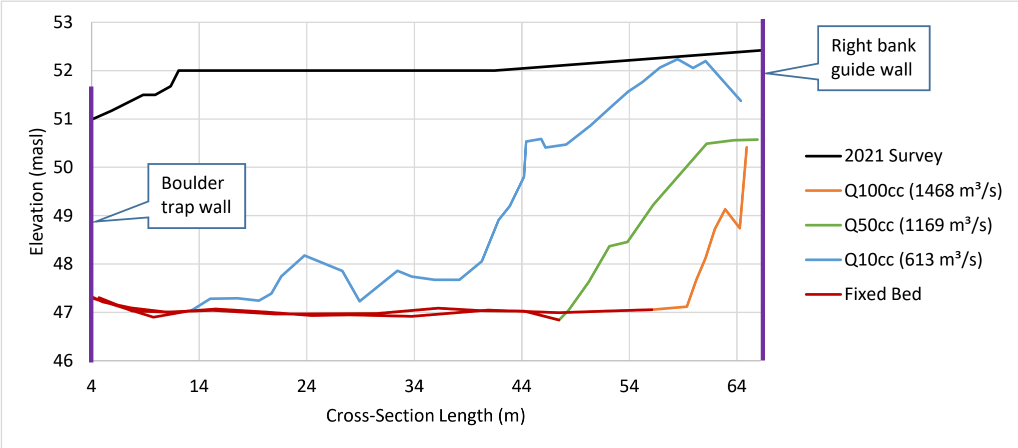


Figure 9-28: Cross-section 12 m upstream of the weir showing the observed bed scour during the Q10cc, Q50cc and Q100cc flood events

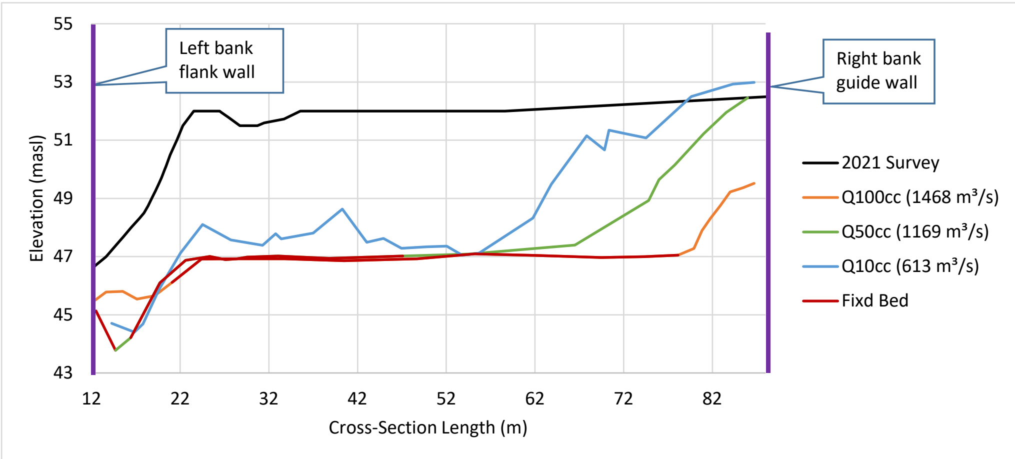


Figure 9-37: Cross-section 30 m upstream of the weir showing the observed bed scour during the Q10cc, Q50cc and Q100cc flood events

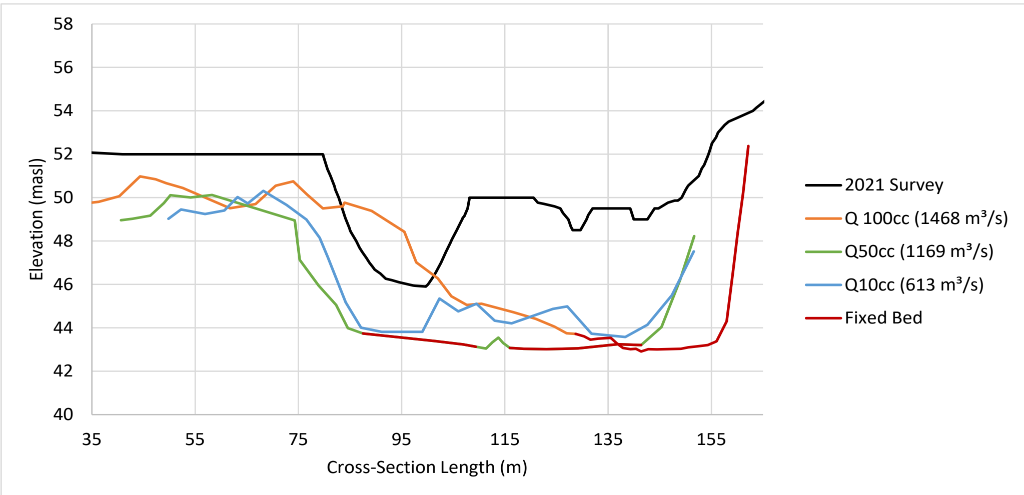


Figure 9-38: Cross-section 35.6m downstream of the weir showing the observed bed scour during the Q10cc, Q50cc and Q100cc flood events

9.7 Stage-discharge rating curves

The stage-discharge rating curves measured during the physical model movable bed tests for the final proposed BRVAS weir (option B2) is shown in Figure 9-38 for up to the Q100cc of 1458 m^3/s. The water levels were measured at the centre of each weir (low notch Crump weir, high notch Crump weir and broad crested weir) approximately 6 m upstream it (based on the 4 x the design head). DWS intends to use these locations to measure the river discharges < 123 m^3/s. While the levels measured for the low notch and high notch show excellent agreement, the measurements were taken in the drawdown zone for discharges > 123 m^3/s. The levels for the broad crested weir were measured in stagnant water until it started spilling during the Q50cc of 1169 m^3/s. The difference in levels between the Crump weir and broad crested weir are therefore indicative of the approximate 0.5 m drawdown for discharges < Q50cc and of the discharge coefficients for > Q50cc. The new weir orientation is better aligned for perpendicular flow across it. The tailwater levels directly downstream of the sediment traps are also shown in Figure 9-18 indicating that the weir will not be drowned for all floods < Q100cc and that the sediment traps will flush effectively during small floods and at the end of large floods.

Important elevations that were measured during the physical model tests are summarized below:

- The dividing walls for the Crump weir are built for the weir's design flood of 123 m^3/s to a level of 52.8 masl (between a 1-year and 2-year flood).
- The broad crested weir and guide wall elevation of 57.0 masl coincides with the Q50cc flood.
- The level of the top of the intake structures and flank wall is 58.48 masl i.e. the Q100cc flood level plus an additional 0.5 m for freeboard against wave action.
- The berm on the right bank was designed to prevent spilling up to the RMFcc of 61.2 masl with no freeboard included.

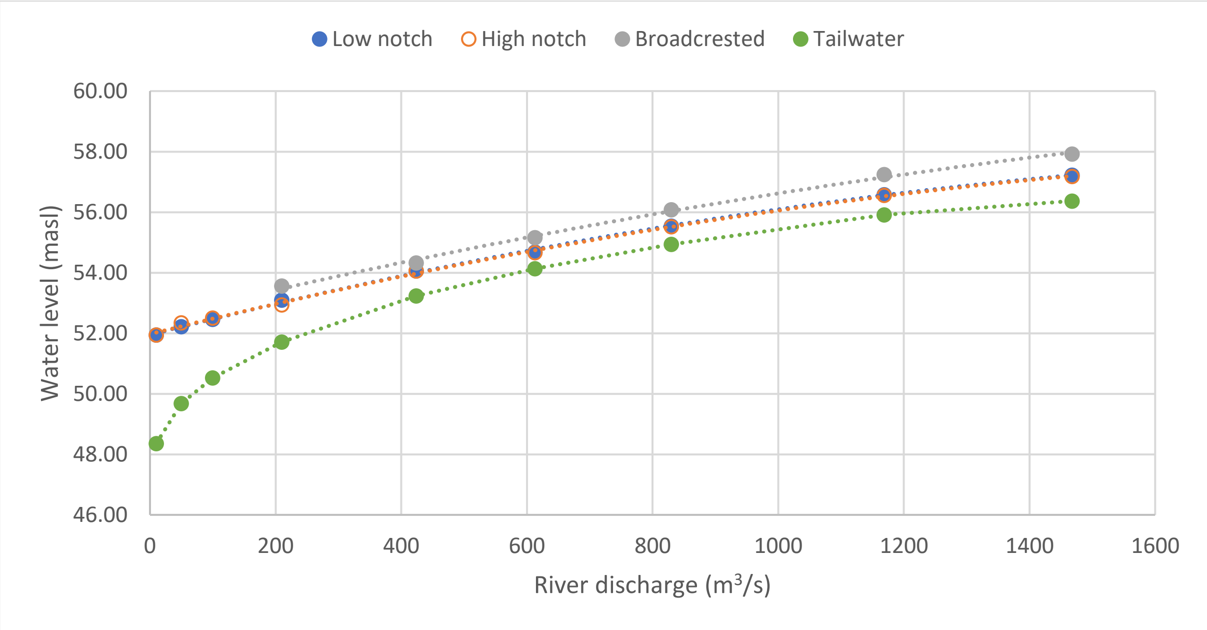


Figure 9-39: Observed stage-discharge rating curves for the proposed weir

The coordinates for the needle locations used to measure the water levels are summarized below:

- Low notch Crump: X -1822.433 Y -3689125.924 (crest 51.6 masl)
- High notch Crump: X -1790.755 Y -3689115.031 (crest 51.9 masl)
- Broad crested: X -1723.611 Y -3689091.940 (crest 57.0 masl)
- Tailwater: X -1879.413 Y -3688996.597

It is legally required that buildings be erected outside the 50-year floodlines along natural water courses (Administrator's Notice No. 1220 – Provincial Gazette, 16 July 1975, still enforced in the northern provinces). In terms of the National Water Act, 1998 (Act No. 36 of 1998) the 100-year floodlines need to be indicated on layout plans for providing information on potential flood hazards. The top of the intake structures and flank walls were designed for the 100-year flood while the top of the hoppers and pump station will protrude above the RMFcc elevation (refer to Figure A1-5 in **Appendix A1**).

The probability of a flood event is often described in various terms. For example, a 100-year flood is a flood peak that has a 1% chance of being exceeded in any year. Terms in general used to describe such a flood include "100-year flood", "1% flood", "100-year Average Recurrence Interval (ARI)" flood, "Q100 flood" and "1% Annual Exceedance Probability flood". Note that this does not mean the structure is designed for 100-year lifespan but rather to be safe for floods smaller than the 100-year flood without overtopping. The Annual Exceedance Probability (AEP) avoids the common misconception that, for example, a 100-year flood can only occur once every 100 years, or that you are 'safe' for another 100 years after you experience such an event. The actual risk of experiencing a different size flood events is set out in Table 9.2. The table shows that a structure in a certain location for 70 years will have a 50% chance of experiencing at least one 100-year flood and a 16% chance of experiencing at least two 100-year floods.

If a 50 year flood is considered for the design of the guide wall, over a period of 70 years, there is a 75% chance of experiencing at least one 50-year flood, and a 41 % chance of experiencing at least two 50-year floods. Compared to a 100-year flood, the risk is considerable higher with a 50-year flood. If the berm on the right bank is designed for the RMFcc, it corresponds approximately with a 10 000-year flood (according to Figure C-2 of Appendix C) and a 0.01% AEP. It is not necessarily possible to fully eliminate flood risk as this would require placing all development above the Probable Maximum Flood level. This often cannot be justified economically and is often not even possible for many places.

Table 9-2: Probabilities of experiencing a given size flood once or more in a lifetime

Size of flood (chance of occurrence in any year) ARI/(AEP)	Probability of experiencing the given flood in a period of 70 years*	
	At least once (%)	At least twice (%)
1 in 10 (10 %)	99.9	99.3
1 in 20 (5 %)	97.0	86.4
1 in 50 (2 %)	75.3	40.8
1 in 100 (1 %)	50.3	15.6
1 in 200 (0.5 %)	29.5	4.9

**Predicted by statistical theory for random events*

9.8 Floodlines

The contour drawing with floodlines for the 50 year and 100 year floods (with climate change) with and without the abstraction works and weir are given in **Appendix F**. The floodlines are based on measured maximum water levels in the physical model and were extended upstream of the physical model domain with the hydrodynamic model. Figure 9-40 illustrates the difference between the two models and on average the difference was 0.13 m. The Q50cc model water levels with the weir are slightly higher than the Q100cc without the weir and the Q100cc model water levels with the weir are also higher than the Q100cc without the weir. The physical model observed flood levels with the weir and abstraction works were extended upstream by 2D hydrodynamic modelling to be able to generate the floodlines (**Appendix F**).

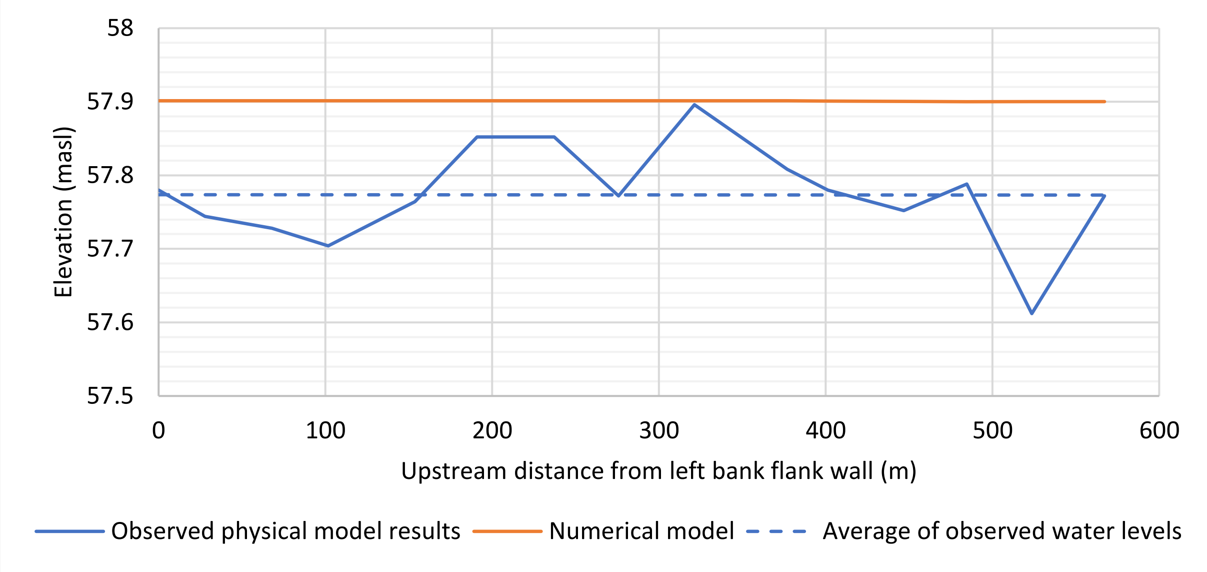


Figure 9-40: Longitudinal section of left bank water levels as observed in the physical model and simulated in the 2D numerical model

These floodlines may be used to quantify the incremental damming to establish whether landowners would require compensation. The TCTA proposed expropriation/compensation lines were determined based on the larger of the following two horizontal distances from the Q100cc floodline:

- The horizontal distance measured 15 m landward from the Q100cc level.
- The horizontal distance intersecting the topography at 1.5 m vertical height above the Q100cc level.

The floodlines for the Q50cc and Q100cc floods with the weir and abstraction works indicate that a saddle berm is required with its crest level based on the RMFcc SEF, to the east of the proposed berm at the right bank side of the proposed weir (**Figure F-1, Appendix F**). Floodlines were however also determined with the future proposed weir, main berm and abstraction works but without the saddle berm to determine the flood flow patterns downstream of the saddle berm site.

The following floodlines, water level and other lines were indicated on the floodline drawing which is enclosed in CAD, in **Appendix F**:

- Q50cc floodline: current scenario without weir and abstraction works, and no saddle berm
- Q50cc floodline: future scenario with weir and abstraction works, but no saddle berm
- Q50cc floodline: future scenario with weir and abstraction works and with saddle berm
- Q100cc floodline: current scenario without weir and abstraction works, and no saddle berm
- Q100cc floodline: future scenario with weir and abstraction works, but no saddle berm
- Q100cc floodline: future scenario with weir and abstraction works and with saddle berm
- MOL of proposed weir and abstraction works at 51.6 masl (EWR spilling with 0.3 m head on the fishway-canoe chute)
- Compensation lines based on the TCTA guidelines given above.
- Cadastral boundaries

9.9 Berm scour tests

The proposed S-berm layout was tested in a physical model setup to evaluate the erosion on the berm near the headwall where the velocities are high. A section of the berm was set-up with the movable bed material at the proposed 1:2.5 (V:H) slope as shown in Figure 9-41. Only the RMFcc flood (4494 m^3/s) was tested for the S-berm scour test. An Acoustic Doppler Velocimeter (ADV) instrument was used to measure the mean flow velocities at points along the toe of the embankment.

Figure 9-41: Movable bed setup of the s-berm test for erosion protection

Figure 9-42 and Table 9-3 summarises the results from the movable bed test of the S-berm without erosion protection. The area of berm material that were completely washed away is 760 m^2, or a 608 m long section when measured along the toe line from where the earth embankment starts against the headwall. The ADV measurements were taken at points 1 to 5 at a depth of 0.6 times the flow depth. The flow velocity increases as the flow accelerates around the bend of the berm toward the broad-crested weir. Figure 9-43 illustrates how the flow curves around the S-bend towards the broad-crested weir.

During the RMF test it was observed that the flow would approach the broad-crested weir parallel to the berm upstream, away from the s-bend, and would then reach the s-bend at an almost perpendicular angle before curving and accelerating around the bend. Some of the flow would bend towards the berm creating a clockwise current against the s-bend which caused further scour. Figures 9-43 and 9-44 show the effect of the current against the s-bend.

Table 9-3: Results for the movable bed test of the s-berm embankment for the RMFcc

Point	X-coordinate	Y-coordinate	Flow velocity (m/s)	Water level (masl)
1	-1658	-3689081	3.66	60.40
2	-1642	-3689075	2.84	60.88
3	-1625	-3689058	2.70	61.00
4	-1621	-3689034	2.51	61.00
5	-1620	-3689016	2.55	61.00

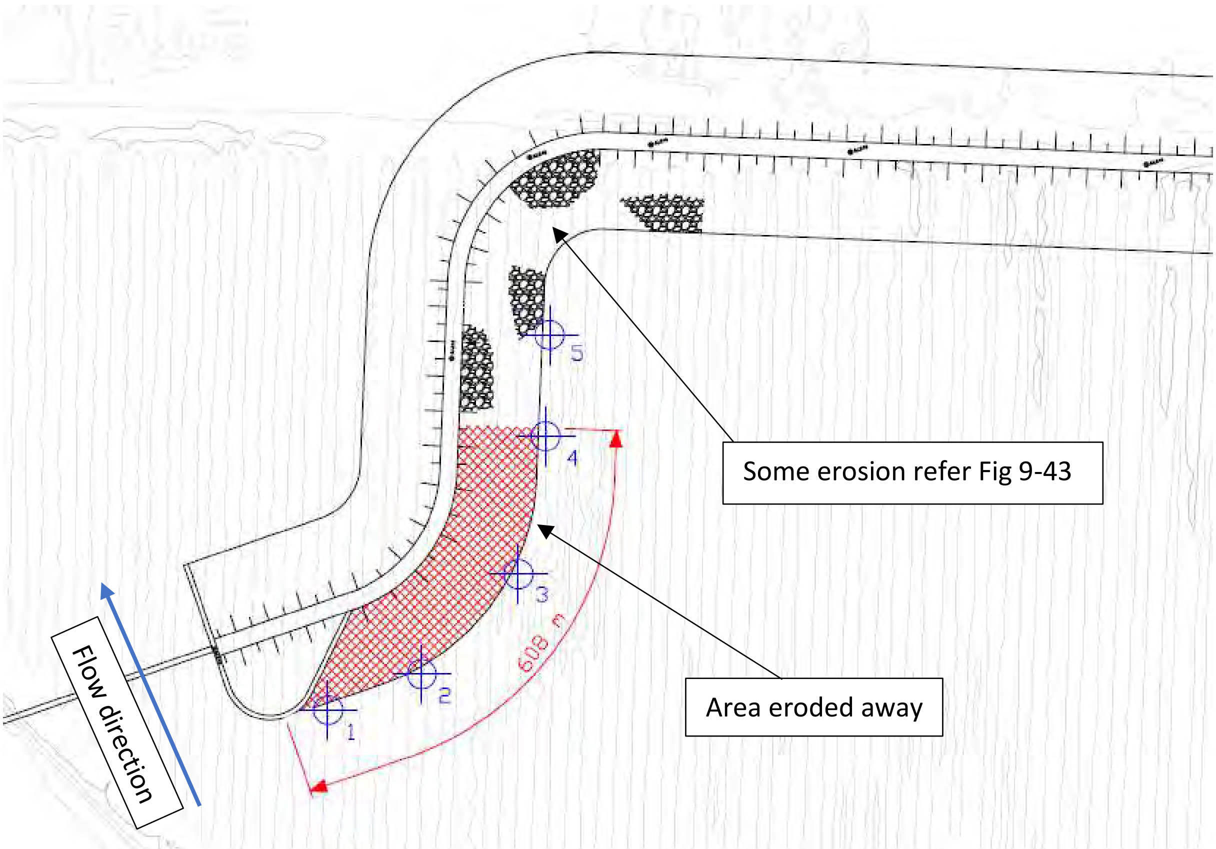


Figure 9-42: Area of berm eroded away during RMFcc test and the flow velocity measuring locations

Figure 9-43: Downstream view of the RMFcc flood flowing around the berm headwall

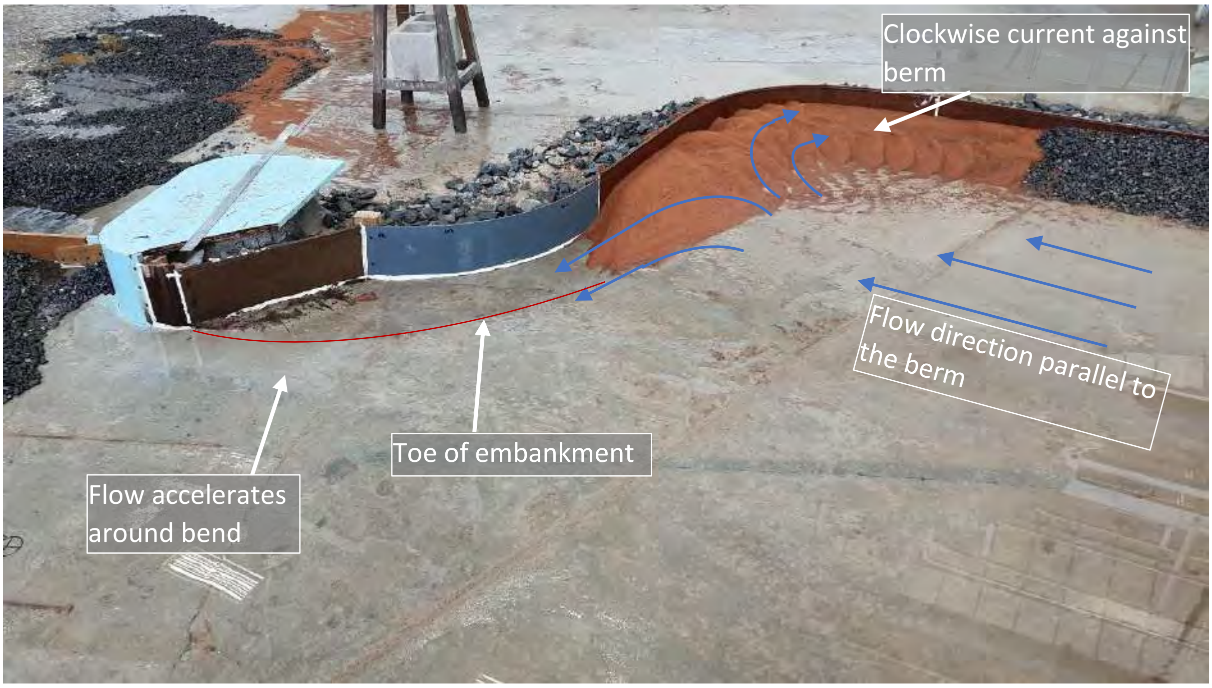


Figure 9-44: Area of unprotected berm eroded away during the RMF cc flood

From the RMFcc model tests it is recommended that 608 m length at the end of the berm requires erosion protection such as riprap and filter layers, while erosion protection at the 90 degree bend should also be investigated based on the Table 9-3 observed hydraulic data.

10. Temporary works model tests

Four possible options to construct temporary works (coffer damming) and river diversion considerations for the construction of the proposed BRVAS abstraction works and weir were investigated. The 2 year, 5 year and the 10-year annual recurrence interval (ARI) floods (182, 369 and 533 m^3/s respectively), without the effects of climate change, was assumed as the possible design floods for the temporary works and was tested in the 1:40 physical model to evaluate the temporary works options.

The following design conditions were assumed for the layout options of the temporary works options:

- The maximum water level downstream of the weir site is expected not to exceed 54 masl for a Q10 flood event. The tailwater level of the Q10 flood without the effects of climate change is 53.75 masl.
- The maximum water level upstream of the weir site caused by the temporary works was assumed to not exceed 55 masl for a Q10 flood event; 1 m was added to the assumed downstream maximum water level to account for the increase in water level caused by the cofferdam.
- Cofferdams (earth embankments) and excavations have bank slopes of 1:2.5 (V:H).
- Cofferdams (earth embankments) have a crest width of 3 m.
- The river diversion channel must return to the main Berg River channel at the same elevation as the bed of the present main channel to prevent retrogressive erosion.
- Temporary concrete walls were not considered as the bedrock at the site of the weir is relatively deep, between 41 masl and 44 masl.
- The selected temporary works were tested in the hydraulics laboratory for the current Q2, Q5 and Q10 floods (without future climate change impacts) to guide the contractor to decide on the acceptable risk during construction.

10.1 Description of Temporary Work Options Considered

Four possible temporary works scenarios (Options A, B, C & D) were considered, and they are described under the respective headings below (in no particular order of preference):

Option A: Figure 10-1 shows the general layout of this option which include two main cofferdams, each constructed in phases, with Option A constructed in three Phases (phase 1, 2 and 3). As part of Phase 1, the right bank 100 m broad-crested weir, guide wall and berm headwall are constructed. In Phase 2 one cofferdam is constructed on the left bank isolating all the structures to be constructed from the left bank up to and including the fishway-canoe chute. A diversion canal is excavated on the right bank. The general layout of Phase 2 is shown in Figure 10-1 (left). Phase 3, shown in Figure-10-1 (right), diverts the river flow over the completed low notch Crump weir and through the boulder trap and a cofferdam is constructed over the high-notch, to the right bank side of the fishway-canoe chute, and extends to the right bank to complete the construction of the high-notch Crump weir where the diversion canal was excavated in Phase 2.

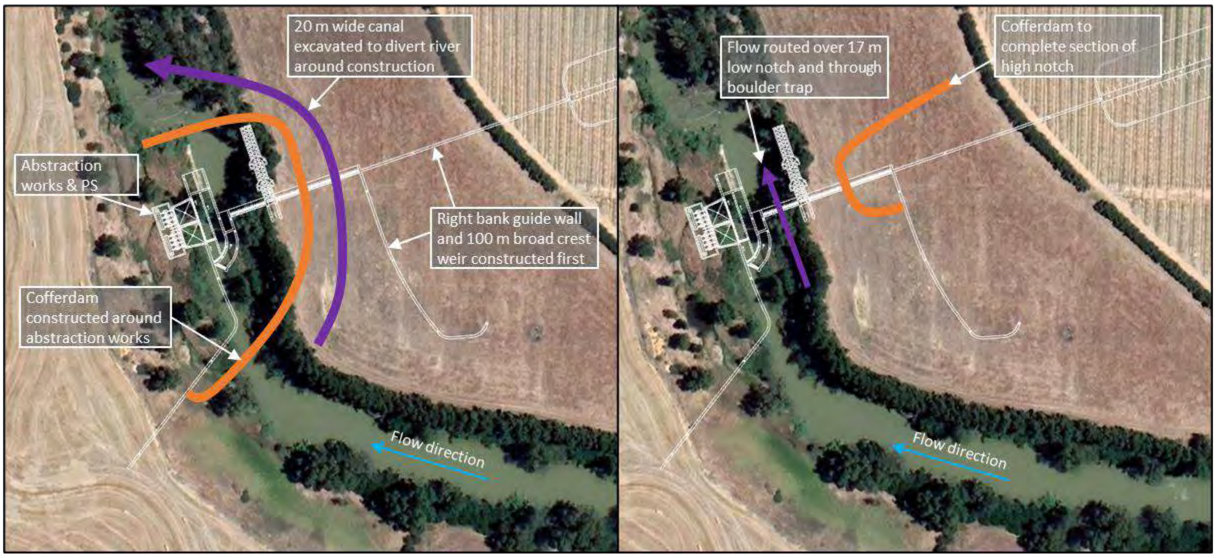


Figure 10-1: Option A temporary works layout, Phase 1 (left) and Phase 2 (right)

Option B: Figure 10-2 (left) shows the general layout of temporary works for Option B. This option can also be constructed in two phases. Phase 1 includes the construction of the concrete guide wall and the 100m long right bank portion of the broad-crested weir. In Phase 2 the river is routed around the abstraction and pump station via a canal on the left bank with a cofferdam protecting the abstraction works, pump station, part of the left bank flank wall, low notch, fishway-canoe chute, and high notch. The remainder of the left bank flank wall is completed once the cofferdam has been removed.

Option C: Figure 10-2 (right) illustrates the layout of Option C. In Phase 1 a portion of the left bank flank wall is constructed together with the right bank guide wall as well as the broad-crested weir which is constructed up to the bed invert level of the temporary canal. A temporary canal is excavated through the broad-crested weir to the right of the right bank dividing wall. In Phase 2 a cofferdam is constructed upstream between the right bank guide wall and the portion of the flank wall that has been constructed as well as a second cofferdam constructed downstream from the right bank dividing wall to the left bank. Once all construction of the permanent works has been completed, the cofferdams are removed, and the canal is backfilled. The last upper portion of the broad-crested weir in the zone of the diversion canal is constructed last.

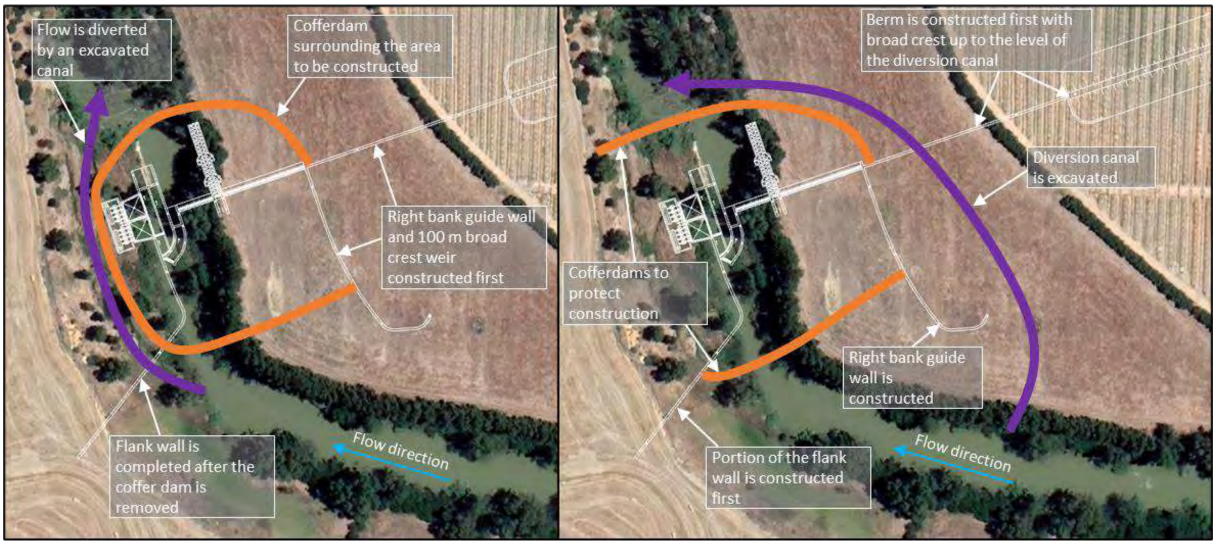


Figure 10-2: Temporary works layout for Option B (left) and Option C (right)

Option D: Option D is shown in Figure 10-3. Phase 1, shown in Figure 10-3 (left), consists of a river diversion canal that is excavated through the centreline of the right bank guide wall to divert the flow around the construction of the abstraction works, pump station, low notch, fishway-canoe chute, and a portion of the high notch. Phase 2 (shown in Figure 10-3 (right)) comprises a cofferdam across the high notch weir around the right bank guide wall to complete construction of the high notch and the construction of the dividing wall. The motivation for locating the river diversion canal on the centre line of the right bank guide wall is to construct the right bank guide wall foundation excavation since it is required for the permanent works, and it is therefore beneficial to use this excavation as part of the temporary works.

Figure 10-3: Temporary works layout for Option D, Phase 1 (left) and Phase 2 (right)

10.2 Desktop Evaluation of the Options A, B, C and D

A desktop evaluation of the four options described in Section 2 was carried out to decide on the two most promising options which could be evaluated in more detail. Table 10-1 summarises the evaluation in the form of advantages and disadvantages.

Table 10-1: Summary of desktop evaluation of temporary works Options A, B, C and D

Option	Advantages	Disadvantages
A	• Simple configuration and layout • Phased construction results in shorter cofferdams required per section • Less excavation required compared to other options	Not optimal to have flow through boulder trap during construction, since the mechanical works are typically installed last (flow through the boulder trap could be prevented by sand bangs but this will only be possible for very small floods) • Limited available space for cofferdam and excavation of diversion canal to fit over the high notch • Phased construction results in time lost to move cofferdam and machinery to the right bank
B	• Floodplain low on left bank resulting in less excavation required • All construction is done within one cofferdam • Short distance to excavate and stockpile • All construction is from the same riverbank	• Shallow bedrock on the left bank constricts available area for excavation • The size of the pump station further constricts the area to construct • Long cofferdam around abstraction works and weir
C	• Excavated canal partially separated by a concrete dividing wall • Short sections of cofferdam needed to block flow to the construction area • Only one phase of cofferdam construction is necessary to complete the construction of the abstraction works and Crump weirs	• Long and deep excavation required due to high floodplain may become costly • Excavation of a large volume of material may be time-consuming
D	• Excavation required for right bank guide wall used as part of diversion canal which will result in a reduced excavation volume for the temporary works	• Both phases require long cofferdams compared to other options • Deep bedrock 40-41 masl in the area where the dividing wall is to be constructed means that deep excavation will take place next to a raised water level caused by the weir (MOL = 51.6 masl)

Based on the evaluation as summarized in Table 10-1 it was concluded that Options A and C are the two most promising options for the reasons as discussed below:

Option B is not considered a viable option due to the shallow bedrock in the vicinity and the limited space created by the pump station. Option D is also not considered viable due to the deep excavation adjacent to the upstream area of the weir which will permanently raise the low flow water level to at least 51.6 masl and even higher during the winter rainfall season. Option D is therefore considered to be a high-risk option.

10.2.1 More Detailed evaluation of Options A and C

Options A and C have been investigated further in more detail by considering the required excavation for river diversion canals and cofferdam layouts. Excavation volumes and cofferdam fill volumes were considered for each layout as well as the physical footprint of each to ensure that the proposed layout is feasible. The findings of the further evaluation of Options A and C are presented below:

Option A: Figure 10-4 shows the required excavation and cofferdam layout for Phase 2 of Option A to accommodate Q10. The required 20 m bottom bed width of the river diversion canal (similar to the existing river main channel) causes interference with the left bank cofferdam and is not practically possible for Option A. It is estimated that an additional 12 m width would be required to fit the cofferdam between the fishway-canoe chute and the diversion canal. A 12 m decrease in diversion canal width only leaves enough space to fit the cofferdam on the present natural ground level, without provision for the required excavations to bedrock for the construction of the weir and fishway-canoe chute. The size of the river diversion canal is governed by the need to return the flow to the river at the same elevation as the bed of the present river main channel to prevent retrogressive erosion. The restrictions caused by the available area to construct the temporary works makes Option A therefore an unsuitable option for temporary works.

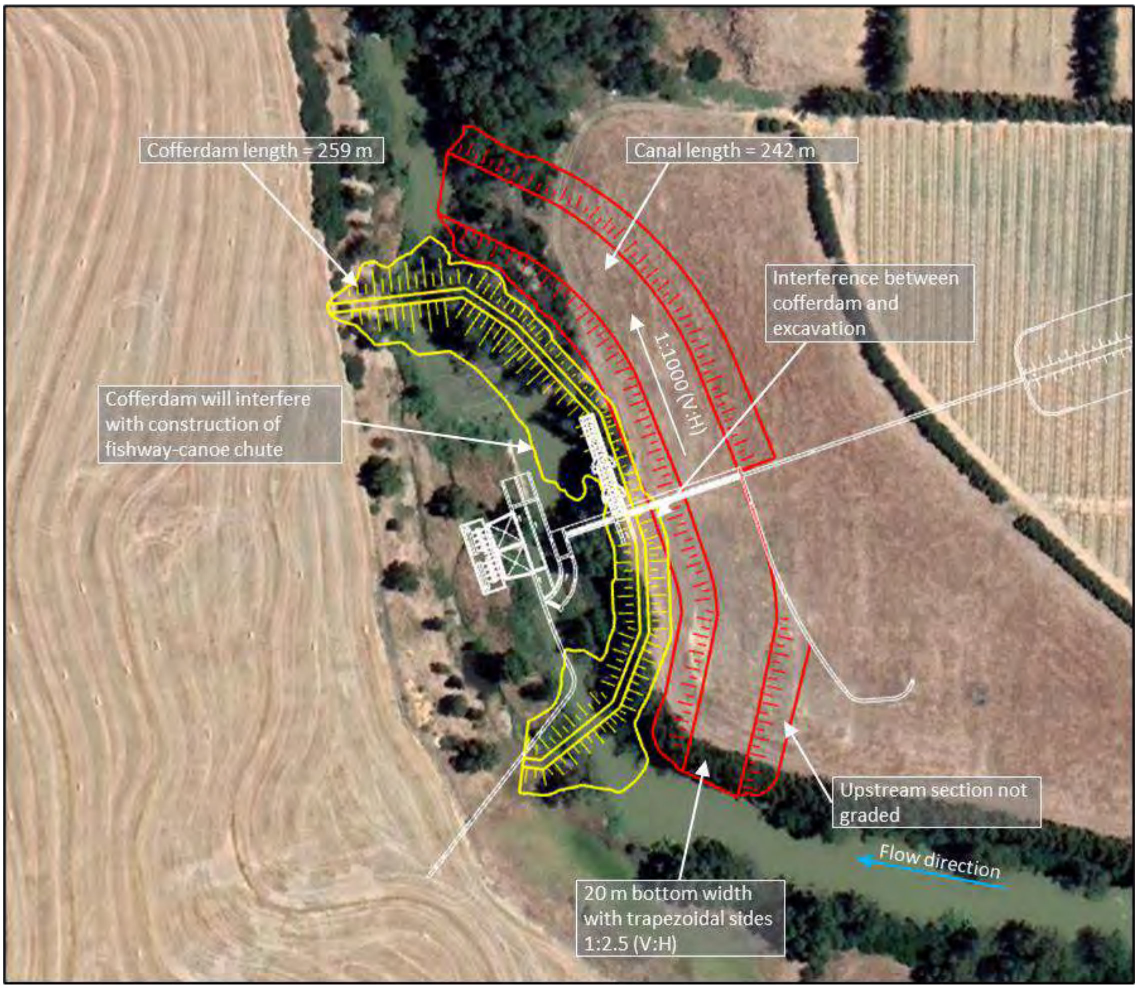


Figure 10-4: Option A, cofferdam and diversion canal footprint

Option C: Figure 10-5 shows the layout of Option C of the proposed temporary works. By moving the diversion canal onto the right bank floodplain, the required area for construction is made available within the cofferdam. The diversion canal is 100 m longer than that of Option A, but it is worth noting that the difference in ground level for each location is only approximately 1 m (the diversion canal of Option A was also located on the right bank floodplain). The calculated excavation volume is 61 222 m³ and the typical excavation depth on the centreline of the canal is 5.7 m on the upstream section of the weir site; the ground elevations gradually decrease downstream. The required volume for the cofferdam has been calculated to be 12 994 m³. The cofferdam was made longer downstream to pass through a narrower section in the river, this reduces the required fill material by approximately 2 000 m³.

The layout of Option C allows for all construction that requires temporary works, to be carried out simultaneously. This is beneficial for work on the abstraction works and pump station that requires more time to be constructed and the layout requires no flow through the boulder trap until construction has been completed. The construction of the Berg River supplement scheme abstraction works (construction period from 2005 to 2008) followed a similar layout where the entire river was diverted around the structure for the duration of construction. Option C offers a lower risk layout because sections, where the flow would be in the direction of the cofferdams, would be protected by a concrete guide wall, and flow downstream of the weir site is expected to flow away from the cofferdam. Option C as shown in Figure 10-5 has been tested in the physical model.

Figure 10-5: Option C, cofferdam and diversion canal footprint

10.3 Physical Model Tests on Temporary Works Option C

10.3.1 Model Setup

The existing 1:40 scale physical model in the Stellenbosch University Hydraulics Laboratory (used to evaluate and optimise the ultimate intake structures) was used to test the Option C temporary works. The ARI-flood peaks (without future climate change impacts) and tailwater levels that were tested are shown in Table 10-2.

Table 10-2: Flood peaks and tailwater levels tested on the temporary works

ARI-flood (years)	Discharge (m³/s)	Tailwater level (masl)
2	183	51.40
5	369	52.84
10	533	53.75

Figure 10-6 shows the laboratory setup of Option C. A fixed bed concrete canal was constructed to serve as the river diversion canal. The bed was fixed to simulate maximum water levels at the temporary works. The areas where there was a movable bed previously was filled with gravel back up to the surveyed ground level. The cofferdams were constructed of a hardboard core, sealed off on the floor to prevent water from flowing through, and shaped with gravel to a 1:2.5 (V:H) slope.

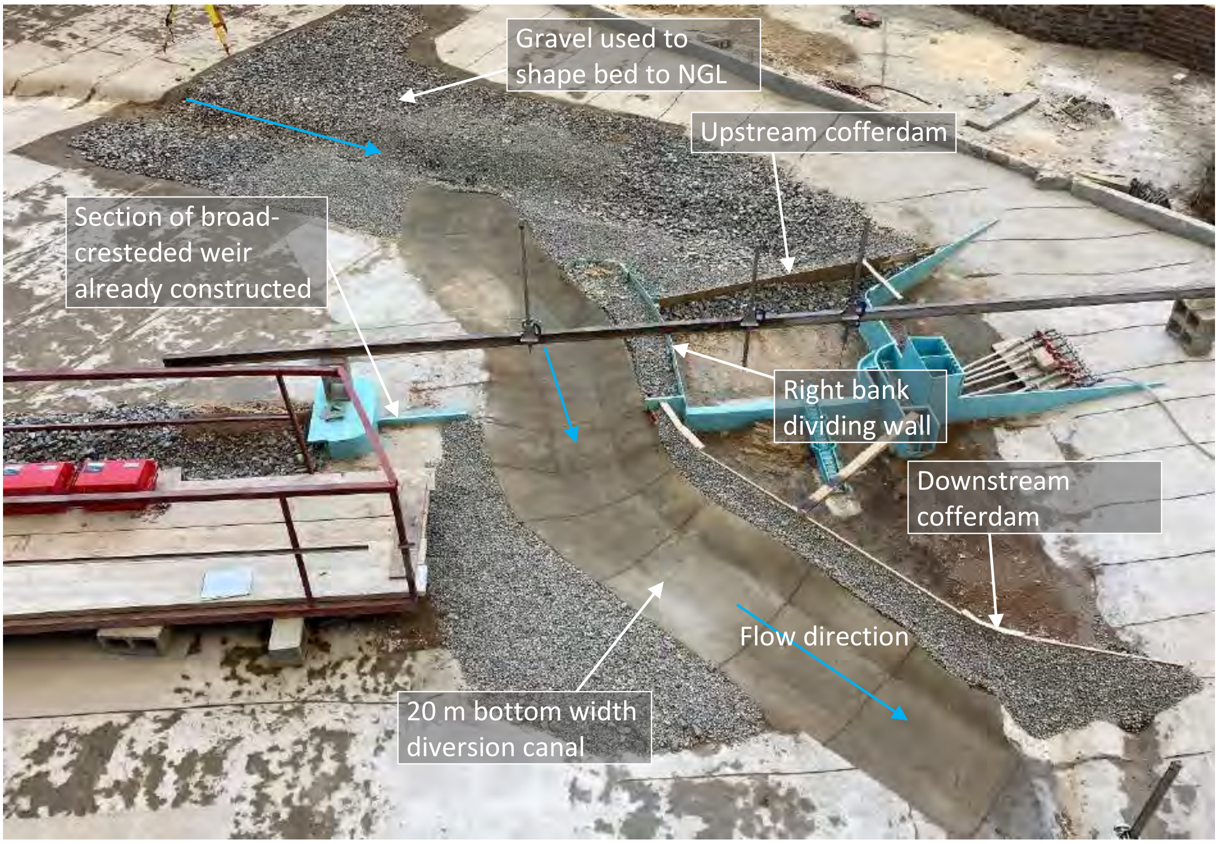


Figure 10-6: Laboratory setup of Option C viewed from downstream with a diversion canal bottom width of 20 m

For each flow rate, the maximum water levels were observed in the diversion canal and against the upstream and downstream cofferdams. The surface flow velocities of each test were observed at three locations along the sides of the diversion canal, and the locations where the velocities were observed are illustrated in Figure 10-7.

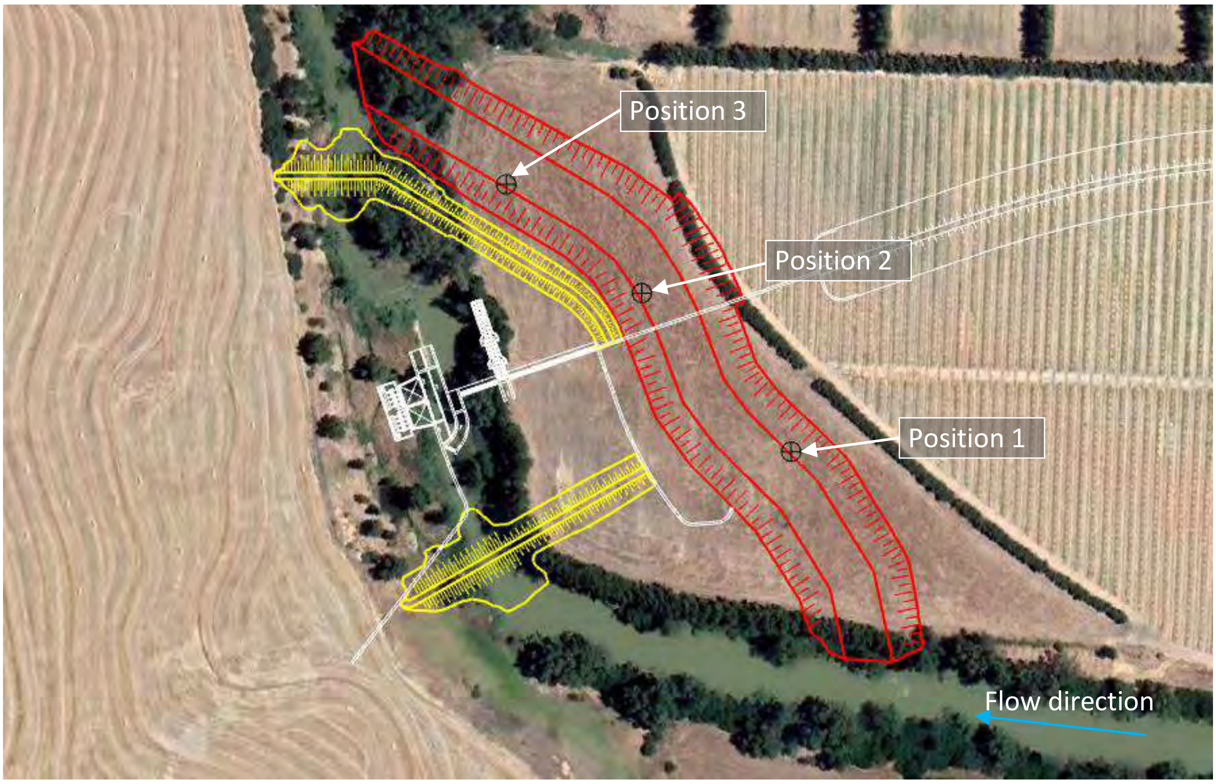


Figure 10-7: Locations where flow velocities were measured in diversion canal

10.3.1 Physical Model Tests Results of Temporary Works Option C

The findings of a series of tests are presented in this section. Table 10-3 presents the observed water levels against the upstream and downstream cofferdams, and the flow velocities observed in the diversion canal are also shown. Figure 10-8 illustrates a longitudinal section of the diversion canal with the observed flood levels with freeboard included. Freeboard of 0.5 m has been added to account for turbulent waves and the flow around the bends in the diversion canal. The required upstream cofferdam crest level was determined to be 54.7 masl and the downstream cofferdam crest level required is 54.6 masl to prevent overtopping during the 10-year ARI-flood of 533 m^3/s. The right bank concrete dividing wall has an elevation of 57.0 masl and requires no further protection measures to prevent overtopping. The flow velocities observed at positions 2 and 3 (left bank) are of importance to ensure that the cofferdam is protected against erosion during flood peaks. A maximum flow velocity of 2.3 m/s has been observed against the downstream cofferdam for the 5- and 10-year ARI-floods.

The 10-year ARI-flood test has been repeated with a 10 m wide (bottom width and 1:2.5 (V:H) sides) diversion canal to determine whether a narrower canal would be possible. A maximum velocity of 4.3 m/s was, however, observed for the 10 m wide diversion canal, which would require some form of erosion protection measures for the downstream cofferdam and diversion canal. A 20 m wide bottom width canal is therefore recommended based on the observed flow velocities.

Table 10-3: Observed water levels and flow velocities for temporary works option C

	Location	2-year ARI-flood	5-year ARI flood	10-year ARI flood
Water levels (masl)	Upstream of cofferdam	52.2	53.7	54.7
	Downstream of cofferdam (tailwater level)	52.0	53.4	54.2
Flow velocity (m/s)	Position 1	1.6	2.5	2.5
	Position 2	1.8	2.1	2.3
	Position 3 (left bank)	1.6	2.3	1.7
	Position 3 (right bank)	-	2.4	2.4

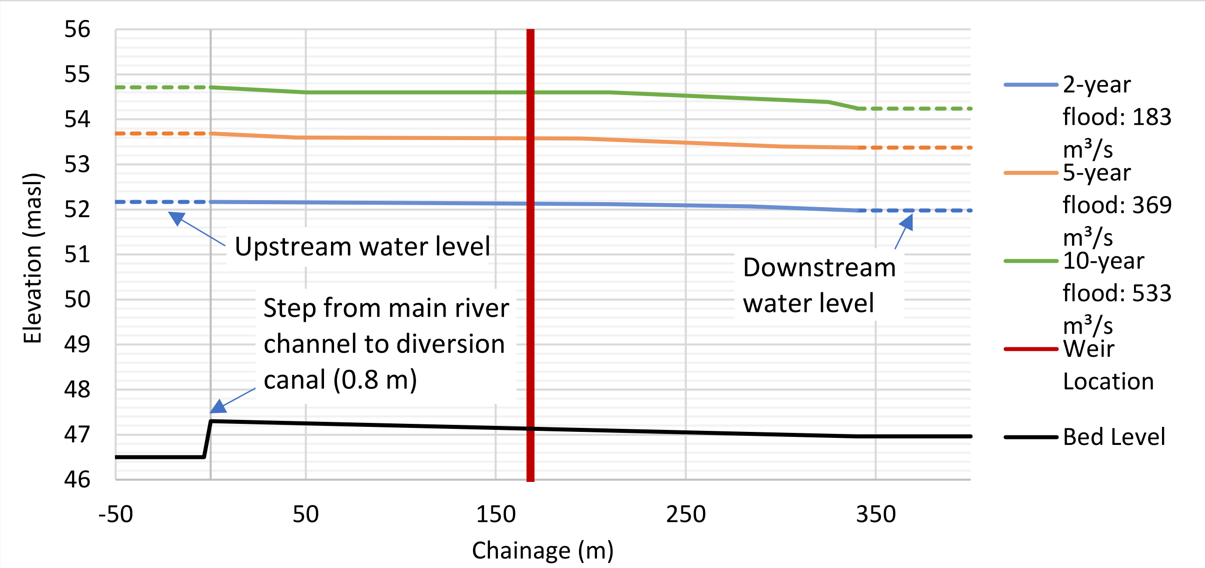


Figure 10-8: Longitudinal section of the diversion canal with the observed water levels for temporary works option C

Figure 10-9 illustrates a 10-year ARI-flood (533 m^3/s) being diverted through the diversion canal. The backwater effect of the diversion works was observed to be 0.5 m for the 10-year ARI flood which results in minimal additional damming. No flow velocities were observed against the upstream cofferdam in the main channel of the Berg River and low velocities were observed against the downstream section of the cofferdam in the main channel of the Berg River with some turbulence. A maximum flow velocity of 2.3 m/s was observed against the left bank section of the cofferdam and diversion canal downstream of the weir location.

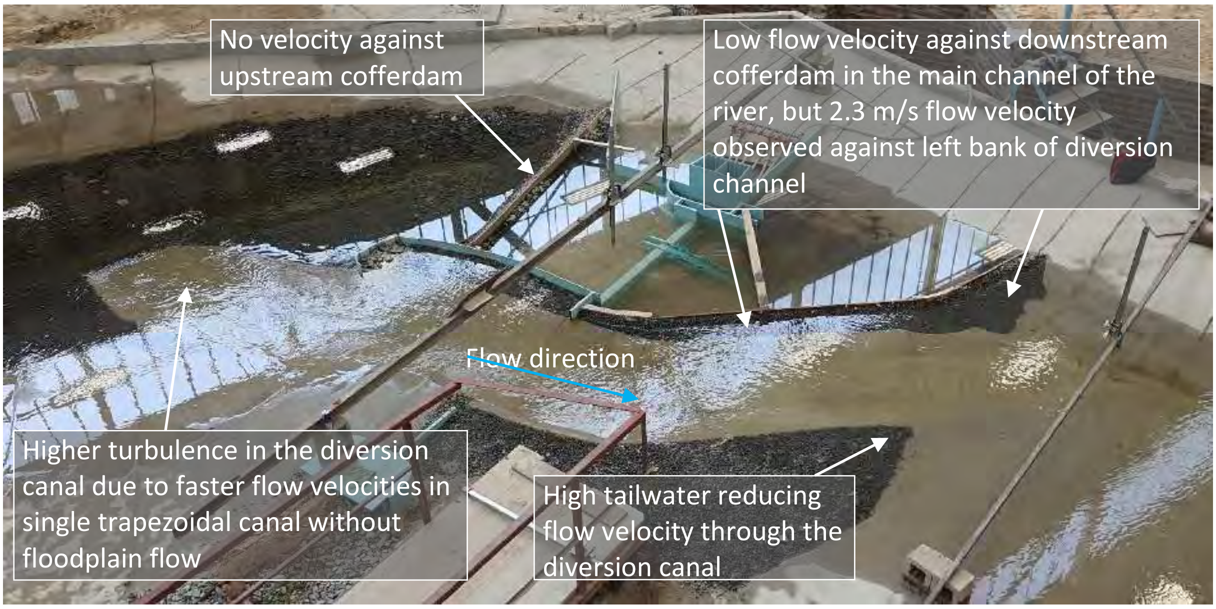


Figure 10-9: Temporary works layout of Option C for a 10-year ARI-flood (533 m³/s), viewed from downstream and the right bank side

Figure 10-10 illustrates the 5-year ARI-flood peak of 369 m³/s. The high floodplain on the right bank causes all the discharge for the 5-year flood to flow through the inlet of the diversion canal from the main channel of the Berg River. The highest flow velocity was measured at position 1 for the 5-year ARI-flood, a flow velocity of 2.5 m/s was observed. Higher turbulence is visible at the diversion canal inlet; this turbulence can be reduced by having a diversion canal inlet with rounded sidewalls. The turbulence in the diversion canal can also be reduced by straightening the canal and by having large radius curves where the canal changes direction. All the flow is contained within the diversion canal with no flow over the right bank floodplain, the backwater effect is 0.3 m for the 5-year ARI-flood.

Figure 10-10: Temporary works layout of Option C for a 5-year ARI-flood (369 m³/s), viewed from downstream

Figure 10-11 illustrates the 2-year ARI flood peak (183 m³/s) which represents the typical flood event that can be expected during the construction period during the winter rainfall season. The maximum flow velocity through the diversion canal is 1.8 m/s.

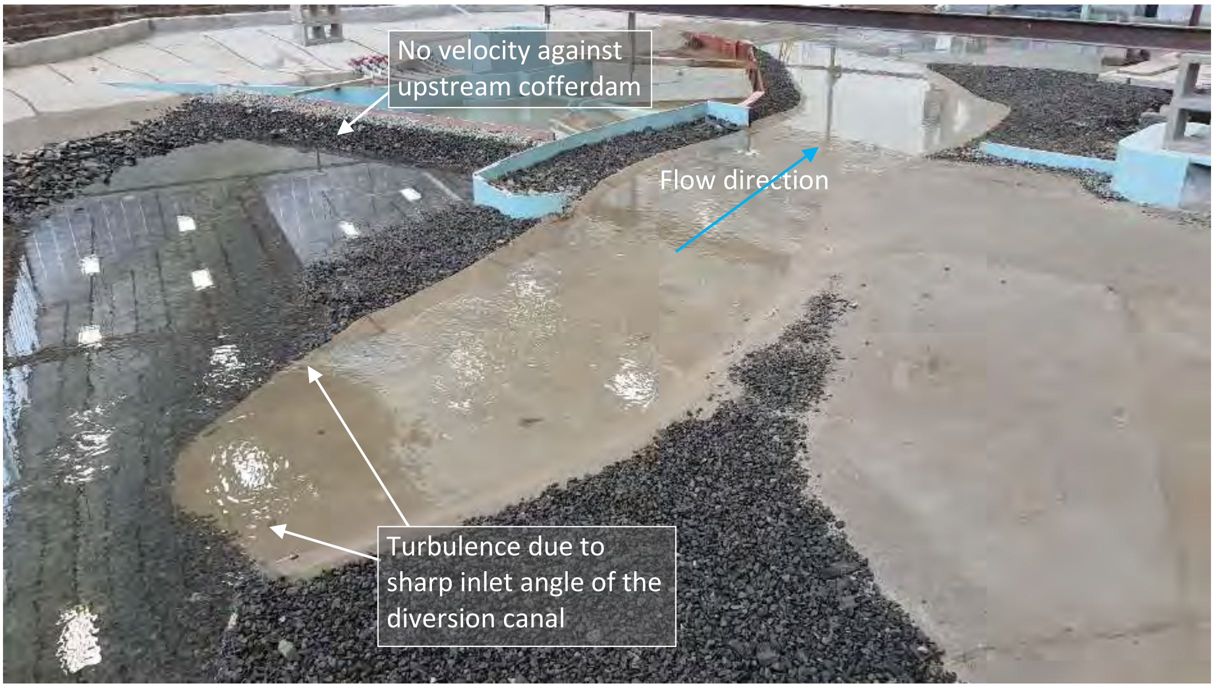


Figure 10-11: Temporary works layout of Option C for a 2-year ARI-flood (183 m³/s), downstream view from right bank floodplain

A proposed modified layout of the temporary works is shown in Figure 10-12. The diversion canal alignment has been straightened with only two bends with 50 m radii to streamline the flow patterns to reduce turbulence in the diversion canal. The inlet of the diversion works has been adjusted to include a 10 m radius inlet shape for the left and right bank of the canal to better streamline the flow transition from the main river channel to the diversion canal. The excavation volume of the canal is 58 600 m³. The layout of the upstream cofferdam will be dependent on the length of the left bank flank wall that can be safely constructed without the need for temporary works.

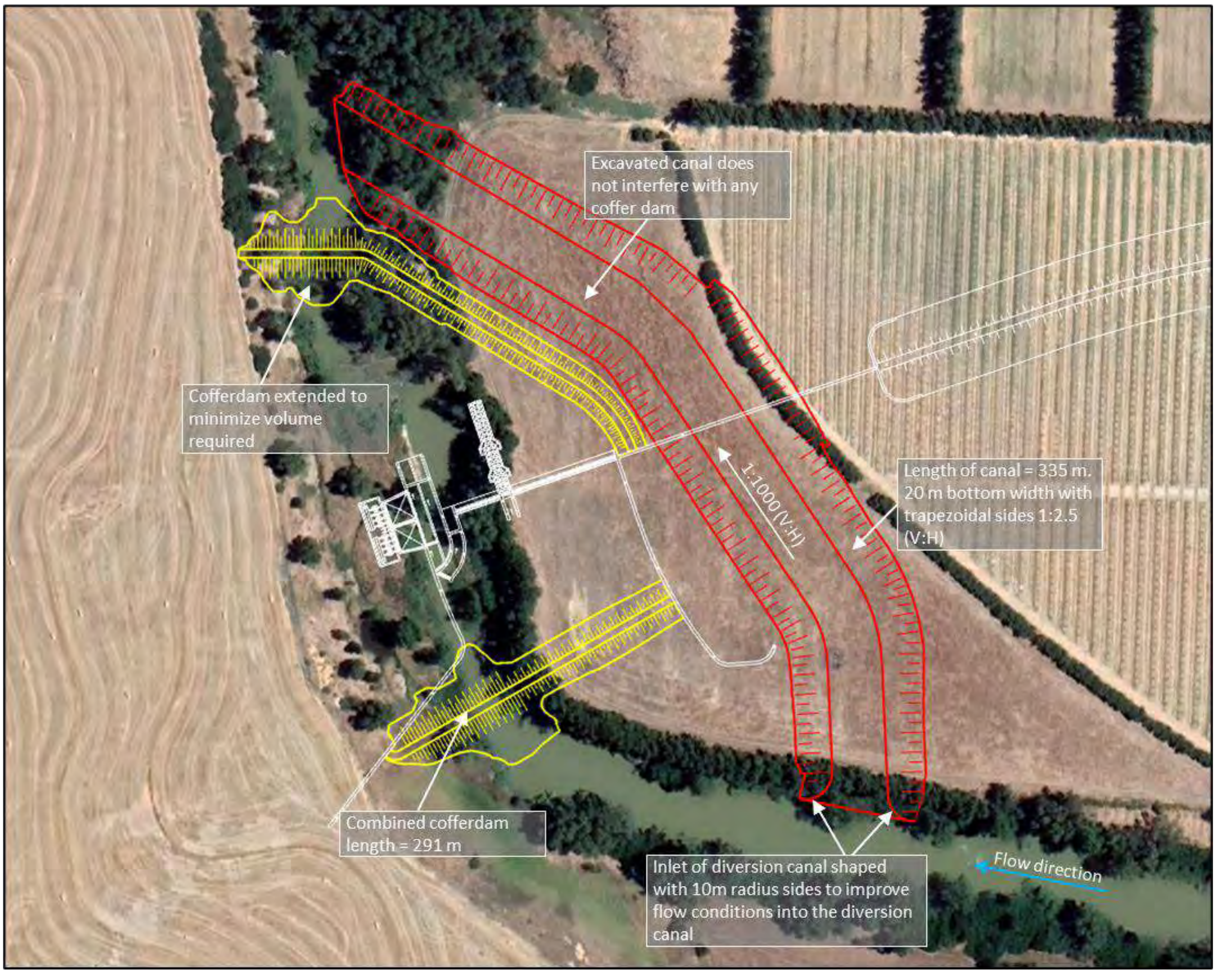


Figure 10-12: Modified proposed layout of the temporary works for Option C

10.4 Physical model tests on the modified Option C layout

The proposed layout Option C of the temporary works shown in Figure 10-12 was further modified with two 50 m radii bends to reduce the required excavation of the temporary works. The bends in the design are not optimal and the inlet of the diversion canal is not aligned with the flow direction in the main channel of the Berg River. The modified proposed layout shown in Figure 10-13 is an improved layout with only one 50 m bend on the downstream section of the diversion canal. The inlet has been shaped starting at 40 m wide at the Berg River main channel and curving to a final diversion canal width of 20 m wide over the length of 30 m (1:3). The right bank has been shaped to gradually curve from the Berg River main channel to the diversion canal to prevent a sharp bend on the right bank of the diversion canal. These two modifications to the inlet of the diversion canal would reduce scour and turbulence as observed from the physical model tests in Section 10.3.

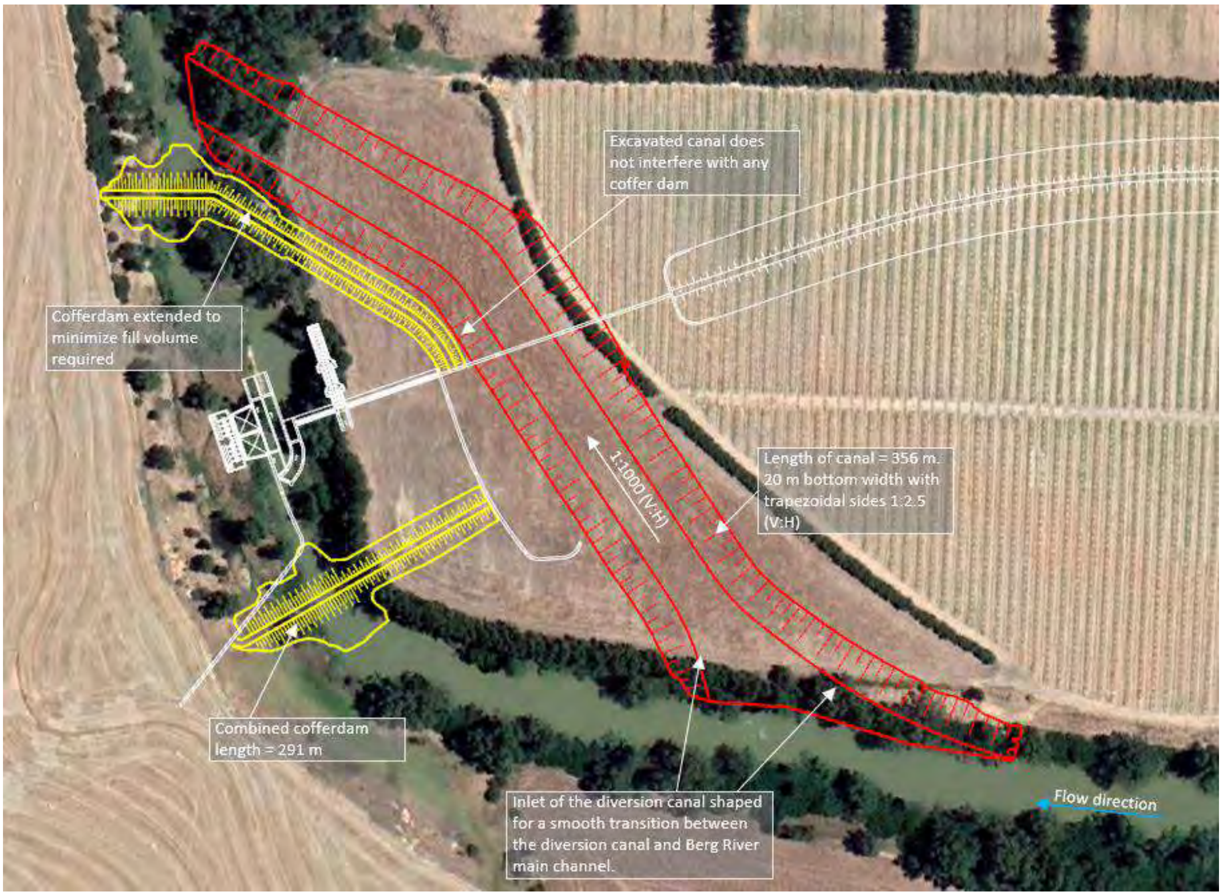


Figure 10-13: Modified final option C layout with a shaped inlet, straight upstream section and one downstream bend

Figure 10-14 shows the physical model setup of the modified final Option C layout in a fixed-bed setup to test the maximum water levels and flow velocities on the sides of the diversion canal. The water levels and velocities were measured at the toe of the diversion canal. An ADV was used to measure the near bed flow velocities which could be used for erosion protection design, the focal point of the ADV recordings were approximately 0.4 m above the bottom of the diversion canal. Measurements were taken on the left and right bank of the diversion canal at the toe of the banks.

Figure 10-14: Modified option C layout constructed as a fixed bed model viewed from downstream

Figure 10-15 shows the setup of the ADV flow velocity measurement during the Q2 flood (183 m³/s). Table 10-4 shows a summary of the fixed bed test results, the maximum velocity for each flood peak is shown with the corresponding measured water level and flow depth. The flow through the diversion canal is controlled by the downstream water level, reducing the flow velocity through the diversion canal. A maximum flow velocity of 2.84 m/s was recorded for the Q5 flood peak, but the velocities of all three flood peaks tested were between 2.25 m/s and 2.84 m/s. The results for the velocity and water level measurements are shown in **Appendix G**.

Figure 10-15: a) setup of the ADV for velocity measurements, b) close up of the ADV probe during a Q2 flood measurement

Table 10-4: Summary of the velocity measurements for the modified option C layout

<table>
<tr><th colspan="2">Description</th><th>Q2 (183 m³/s)</th><th>Q5 (369 m³/s)</th><th>Q10 (533 m³/s)</th></tr>
<tr><td colspan="2">Tailwater level (masl)</td><td>51.4</td><td>52.84</td><td>53.75</td></tr>
<tr><td colspan="2">Cofferdam water level – Upstream (masl)</td><td>51.31</td><td>52.68</td><td>54.01</td></tr>
<tr><td colspan="2">Cofferdam water level – Downstream (masl)</td><td>51.43</td><td>52.99</td><td>53.66</td></tr>
<tr><td rowspan="3">Left bank</td><td>Velocity (m/s)</td><td>2.83</td><td>2.83</td><td>2.74</td></tr>
<tr><td>Water level (masl)</td><td>51.38</td><td>52.77</td><td>53.80</td></tr>
<tr><td>Flow depth (m)</td><td>4.28</td><td>5.55</td><td>6.62</td></tr>
<tr><td rowspan="3">Right bank</td><td>Velocity (m/s)</td><td>2.82</td><td>2.84</td><td>2.81</td></tr>
<tr><td>Water level (masl)</td><td>51.32</td><td>52.90</td><td>53.6</td></tr>
<tr><td>Flow depth (m)</td><td>4.25</td><td>5.69</td><td>6.63</td></tr>
</table>

Figure 10-16 show the flow through the diversion works for the Q10 flood peak. The curved inlet works well to reduce turbulence at the inlet of the diversion canal, there is some minor turbulence caused by the downstream bend. There is also turbulence caused by the flow obstructions on the left bank of the diversion canal in the form of the right bank flood plain guide wall that protrudes into the flow. For the Q10 flood the flow returns to the main Berg River channel over the right bank of the diversion canal at the downstream end of the diversion canal. For all the flows tested there were no velocities against the upstream cofferdam.

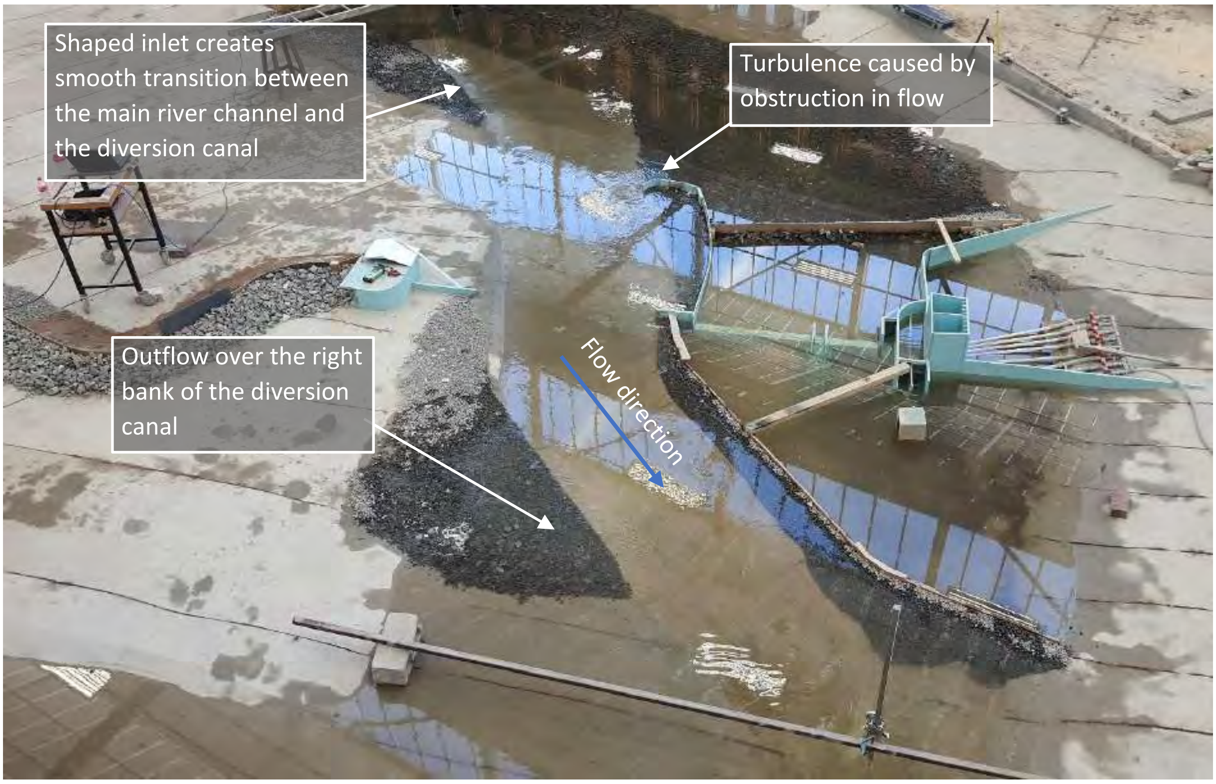


Figure 10-16: Modified option C layout, fixed bed tests for the Q10 year flood

10.5 Movable bed tests on the modified Option C layout

The modified option C layout was reconstructed with a movable bed setup as shown in Figure 10-17 to evaluate the stability of the proposed diversion canal. Suitable protection measures against scour of critical areas could be derived from the test results of the movable bed model and the fixed bed model with the recorded flow velocities and water levels. A large area of the main channel of the Berg River was part of the movable model to evaluate the effect of the flow returning to the main channel. After each test the movable bed area was surveyed, and the change in the bed levels for each test is shown in **Appendix G**.

Figure 10-17: Movable bed setup for the modified option C layout

2-year ARI flood

Figures 10-18 to 10-20 show the extent of the scour after the Q2 year flood (183 m³/s). After the Q2 year flood the channel has lost the trapezoidal shape and started to widen. The shaped inlet worked well to reduce turbulence and scour at the inlet of the diversion canal. Figure 10-19 shows the effect of the widening of the diversion canal. Some scour around the section of broad-crested weir is visible which will have an impact on the downstream cofferdam stability. Erosion protection measures would be required in this location. Figure 10-20 shows limited scour on the left bank of the main channel of the Berg River where the diversion canal flows back into the main channel.

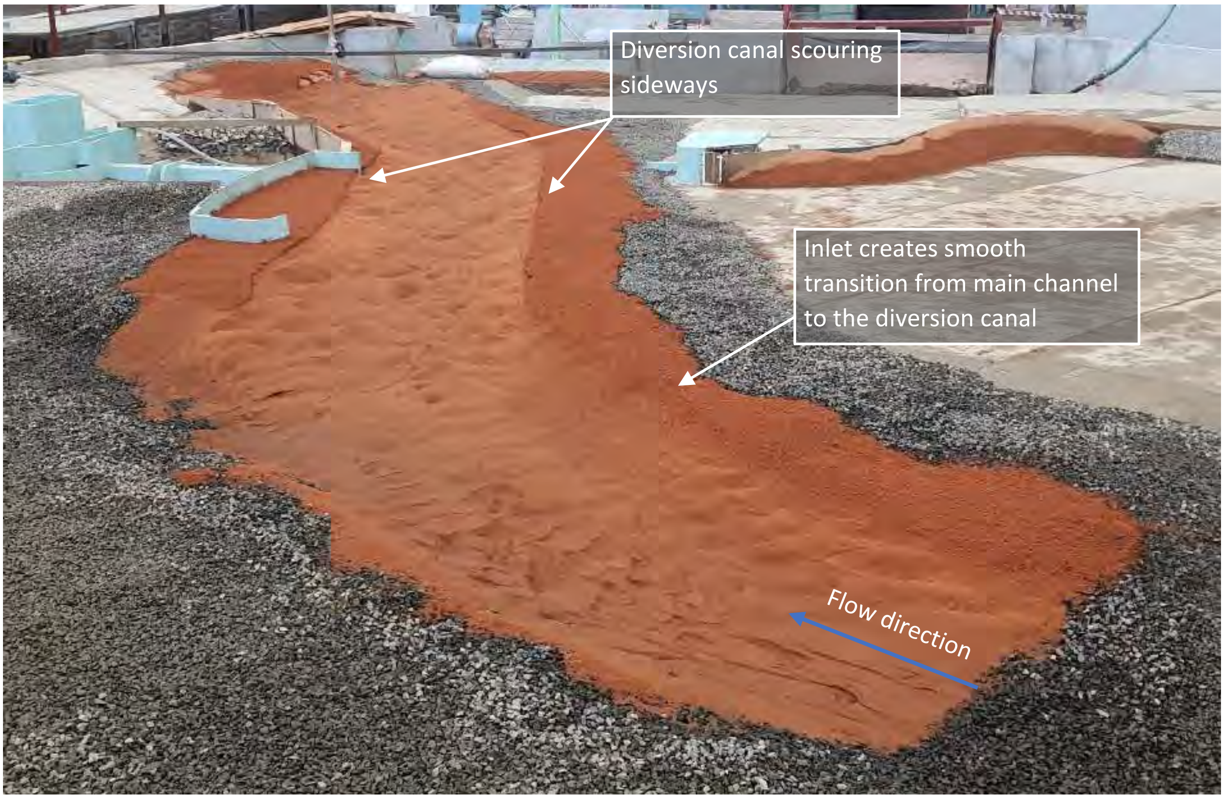


Figure 10-18: Downstream view of the inlet after the Q2 flood peak

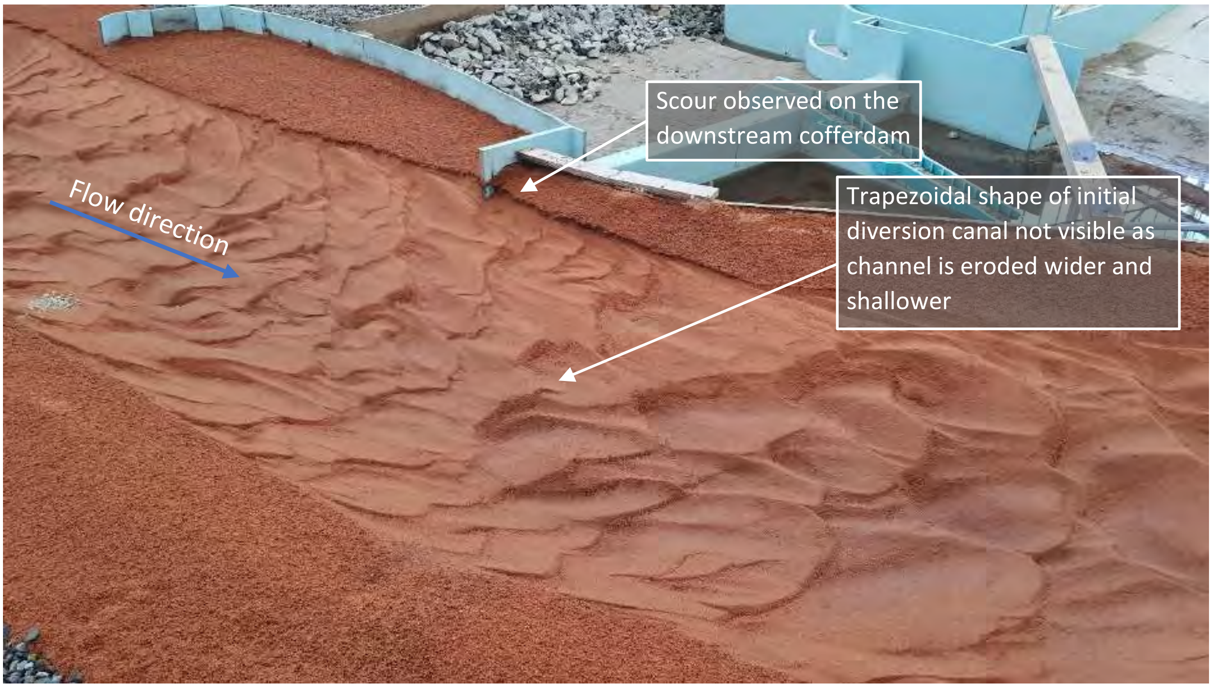


Figure 10-19: Scour around the section of broad-crested weir after the Q2 year flood.

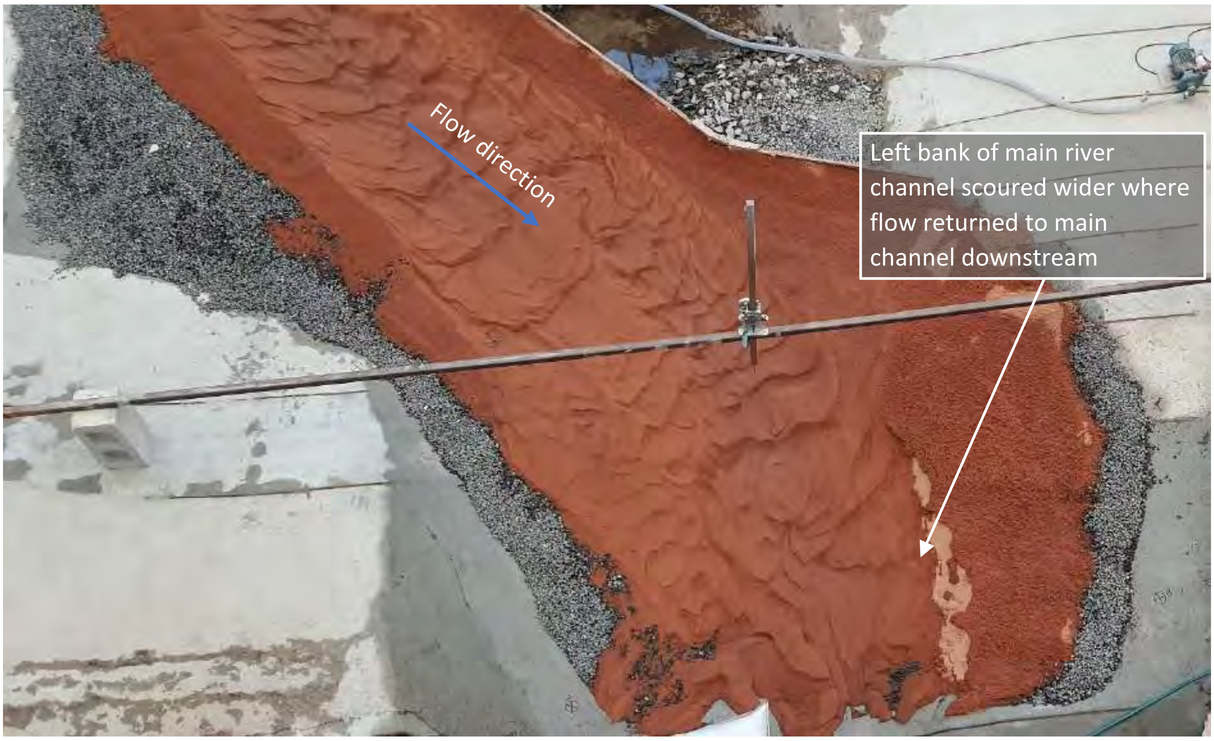


Figure 10-20: Diversion canal after the Q2 year flood downstream where the flow returns to the main channel of the Berg River

5-year ARI flood

Figures 10-21 to 10-24 show the extent of the scour after the Q5 year flood (369 m³/s). Figure 10-21 shows the inlet of the diversion canal guiding the flow into the diversion canal without any noticeable turbulence. The Q5 flood eroded the right bank of the diversion canal just downstream of the bend in the canal to flow straight towards the main channel of the Berg River. All the sediment has been scoured in this location to the boundary of the movable bed. Figure 10-22 shows erosion of the downstream cofferdam just downstream of the broad-crested weir. The scour of the cofferdam started against the weir as observed for the Q2 year flood before moving downstream. Figure 10-23 shows the scour observed at the right bank floodplain guide wall, (a) upstream and (b) downstream, the model has scoured to the boundary in these locations. The extent of the Q5 flood is shown in Figure 10-24, the straight line that has been eroded towards the main channel can be seen. Scour against the left bank of the main channel has been reduces, but due to the limited space towards the downstream boundary, the effect of scour further downstream is unknown.

Figure 10-21: Modified option C layout of the diversion canal during the Q5 flood