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Concept and Viability Design Report

UPGRADING OF MOORDKUIL RAW WATER PUMP STATION

Mechanical, Electrical and Structural Design

Client: Department of Water and Sanitation (DWS)

Implementing Agent: Mossel Bay Municipality (MBM)

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Executive Summary

Zutari (Pty) Ltd was appointed by Neil Lyners and Associates (Pty) Ltd as sub-consultant for the mechanical, electrical, and structural design of the Moordkuil Raw Water Pump Station upgrade. The project, commissioned by the Department of Water and Sanitation (DWS) with Mossel Bay Municipality (MBM) as the Implementing Agent, seeks to improve the reliability efficiency and ease of maintenance of the Moordkuil Raw Water Pump Station supplying Klipheuwel Dam.

The existing pump station, which currently comprises two axial flow pumpsets, has become increasingly maintenance-intensive due to sediment ingress and aging equipment. Earlier feasibility work (2014-2016) recommended a two-stage pumping configuration with no modifications to the existing pump station building; however, a verification study conducted on the riverbed topography at the onset of this investigation identified the presence of a significant rock outcrop upstream of the proposed intake structure, and an increase in sediment load due to developments within the catchment area, prompting a re-evaluation of the original concept.

Following extensive hydraulic, mechanical, structural and economic analyses, three options were developed and assessed. The preferred alternative based on technical and operational considerations - Option 3 (Concept Layout 2) - locates the intake structure onto an existing rock outcrop and introduces a new drywell adjacent to the existing building to house single-stage end-suction pumps. This configuration offers more favourable founding conditions for the abstraction works, allows the existing station to remain operational during construction, and minimizes disruption to the downstream farmer’s pump station.

Option 3 furthermore provides the lowest total project cost (R42.6 million excl. VAT) and avoids approximately R24.5 million in temporary water supply costs by enabling continuous operation of the existing pump station during the 18-month construction period.

Based on the abovementioned considerations, Option 3 (abstraction works/wetwell on rock outcrop with drywell next to pump station, Concept Layout 2) is recommended for implementation.

DWS procured the pumps proposed during the previous (2014-2016) investigation, but the project never proceeded to implementation. To maximise the value of existing assets, a phased approach should be adopted:

- **Phase 1:** Utilisation of existing immersible and end-suction pumps that was bought based on the previous (2014 – 2016) investigation.
- **Phase 2:** Replacement of the Phase 1 immersible pumps with foot valves and installing larger single stage end suction pumps in die drywell.

The hydraulic design ensures the intake structure are self-scouring and resilient to sediment deposition. The civil design provides for robust, flood-resistant structures, with careful integration of new and existing facilities to maintain operational continuity during construction.

The mechanical design supports both current and future pump configurations, with appropriate safety margins for motor sizing and lifting equipment.

The electrical design includes a new 800 kVA transformer, revised LV distribution, simplified control and protection systems, and provision for remote monitoring to the DWS and MBM offices.

The Concept and Viability design confirms that Option 3 is technically feasible, cost-effective, and operationally resilient. It is therefore recommended that the project proceed to the Detailed Design Phase based on this option.

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1 Introduction

1.1 Project Background

Zutari (Pty) Ltd ("Zutari") was appointed by Neil Lyners and Associates (Pty) Ltd ("Lyners") as the sub-consultant responsible for the mechanical, electrical, and structural design of the Moordkuil Raw Water Pump Station Upgrade ("Moordkuil Pump Station"), located near the town of Klein Brak River. Lyners is the principal Professional Service Provider (PSP) or Consultant for this project, with the Department of Water and Sanitation (DWS) as the Client and Mossel Bay Municipality (MBM) as the Implementing Agent.

The Moordkuil Pump Station abstracts raw water from the Moordkuil River and pumps it to the Klipheuwel Dam, an off-channel storage facility for Mossel Bay Municipality (refer to Figure 1-1). The existing pump station comprises two axial flow pumps, of which one is currently operational (refer to Figure 1-2 and Figure 1-3). The design flow is 800 ℓℓ/s with both pumps operating in parallel (two duty, no standby). Over time, the station has become increasingly maintenance-intensive, primarily due to sediment and grit ingress from the riverbed, which has led to more frequent pump maintenance and failures. Historical maintenance records indicate that the pumps required regular refurbishment, causing significant operational disruptions and increased maintenance costs.

1.2 Earlier Studies

Over time, the use of axial flow pumps has declined due to the unavailability of spares and the complexities associated with long drive shafts. In the early 2010's, DWS engaged Zutari for advice on alternative pump types, noting that the proposed solution should require minimal civil construction and/or modification to existing structures. Because the pump station elevation was such that single stage pumping is not possible, it was proposed that two-stage pumping be considered with immersible pumps located in the river and end-suction pumps in the pump station building. In order to protect the immersible pumps during flood events, it was proposed that the pumps be located in a small concrete sump, positioned at the intake of the existing axial flow pumps.

In 2014, DWS appointed Lyners for the Moordkuil Pump Station upgrade. In turn, Lyners appointed Zutari as sub-consultant to undertake the mechanical, electrical and structural designs. At the start of this appointment, DWS indicated that sediment loads have been increasing over time due to developments (both commercial and agricultural) taking place within the upper catchment areas of the Moordkuil River. It was thus proposed that the small concrete sump be replaced with a larger intake structure to address the sedimentation concerns. ASPTech, specialists in river hydraulics and sedimentation, was appointed to undertake the necessary sedimentation modelling and to propose a layout for the intake structure, which was to be located at the intake of the existing pumps. The project was, however, suspended before the Concept and Viability (C&V) design phase could be completed but a Options and Feasibility Report (January 2015) and an Implementation Report (January 2016) were prepared, which form the basis for the current scope of work.

The project has since resumed with Lyners appointed in September 2024 to undertake the completion of project, which includes the phases from Concept and Viability to Close-Out.

1.3 Project Scope and Deliverables

The current appointment is to undertake all phases from ECSA Stage 2 (Concept and Viability Design) up to and including ECSA Stage 6 (Close-out). Zutari's deliverables for the structural, mechanical and electrical scope are summarised as follows:

- Concept and Viability Report (this report);

- Detailed Design Report;
- Tender Documentation as relevant for the structural, mechanical and electrical scope;
- Construction drawings; and
- Close-out Report as relevant for the structural, mechanical and electrical scope.

1.4 Project Area

The Moordkuil Pump Station is located on the banks of the Moordkuil River near the town of Klein Brak River as indicated in Figure 1-1.

Figure 1-1: Project Locality Plan

Figure 1-2: Axial flow pump motors

Figure 1-3: Axial flow pump riser pipes/columns

1.5 Report Structure

The Concept and Viability (C&V) Report is structured as follows:

- Chapter 1 provides the project background, details of the earlier studies, and lists the project deliverables;
- Chapter 2 details the options analysis undertaken to determine the optimal abstraction location and configuration;
- Chapter 3 considers the hydraulic design of the intake structure and raw water pump station;
- Chapter 4 provides details of the civil and structural design of the intake structure and modifications/additions to the existing pump station structure;
- Chapter 5 addresses the mechanical design of the raw water pump station;
- Chapter 6 details the electrical, control and instrumentation aspects of the raw water pump station; and
- Chapter 7 contains the conclusions and recommendations from the concept and viability design.

2 Options Analysis

2.1 Initial Investigations

During the previous project phases (2014 – 2016), several technical studies were undertaken, including:

- **Hydrology, Hydraulics and Sediment Dynamics:** Based on the increased sedimentation caused by developments in the upstream catchment area, sedimentation modelling had to be undertaken. The detailed modelling confirmed that sediment deposition at the existing pump intakes, especially during small floods, was a key contributor to pump wear and system failures. The studies recommended that the previously envisaged small concrete sump rather be replaced by a new larger intake structure to improve local scouring and thereby minimize the risk of sediment ingress.
- **Mechanical and Electrical Options Analysis:** Multiple pumping and intake configurations were evaluated. The preferred solution involved replacing the existing axial flow pumps with four immersible pumps (each 200 ℓ/s), arranged in series with four end-suction pumps. This configuration provides greater redundancy, flexibility, and ease of maintenance. An intake structure with hoppers was recommended to settle out coarse sediment before water reaches the immersible pumps, thereby reducing wear and extending pump service life.
- **Structural and Geotechnical Assessment:** The proposed intake structure is to be constructed within the existing development footprint. Geotechnical investigations evaluated the suitability of the founding conditions, with recommendations for a combination of piles and/or reinforced concrete spread foundations to address the variable alluvium thickness and hydraulic forces on the intake structure.

Based on the outcomes of these studies and subsequent stakeholder engagements, the following key decisions were made during the 2014 – 2016 project phases:

- The pump station will be upgraded to provide a maximum abstraction rate of 800 ℓ/s, utilizing four new pump lines (200 ℓ/s each), which each pump line having 2 x pump sets in series.
- A new low-level intake structure with hoppers will be constructed within the existing footprint to minimize the environmental impact associated with the new infrastructure. The immersible pumps will be housed inside this structure. The structure will be founded on piles.
- Other upgrades required to accommodate the four end-suction pumps will be limited to the extents of the exisitng building.
- All existing mechanical equipment, pumps and electrical switchgear will be replaced, with provision for a possible transformer upgrade.
- Structural modifications to the exisitng building will include a new gantry, repairs to the existing building and improvements to site access.
- An access road will be constructed up to the proposed low-level intake structure to access the mechanical equipment for routine operation, inspection, and maintenance of the mechanical systems, as well as to support the structural upkeep of the intake facility.
- The design should incorporate measures to protect concrete works from aggressive water quality (notably brackish water during tidal events).

Figure 2-1 and Figure 2-2 show the layout and isometric of the pump station developed during the initial project phases. In 2015, before the project was suspended, DWS procured the 4 x immersible and 4 x end suction pumps based on the outcomes of the initial design phases.

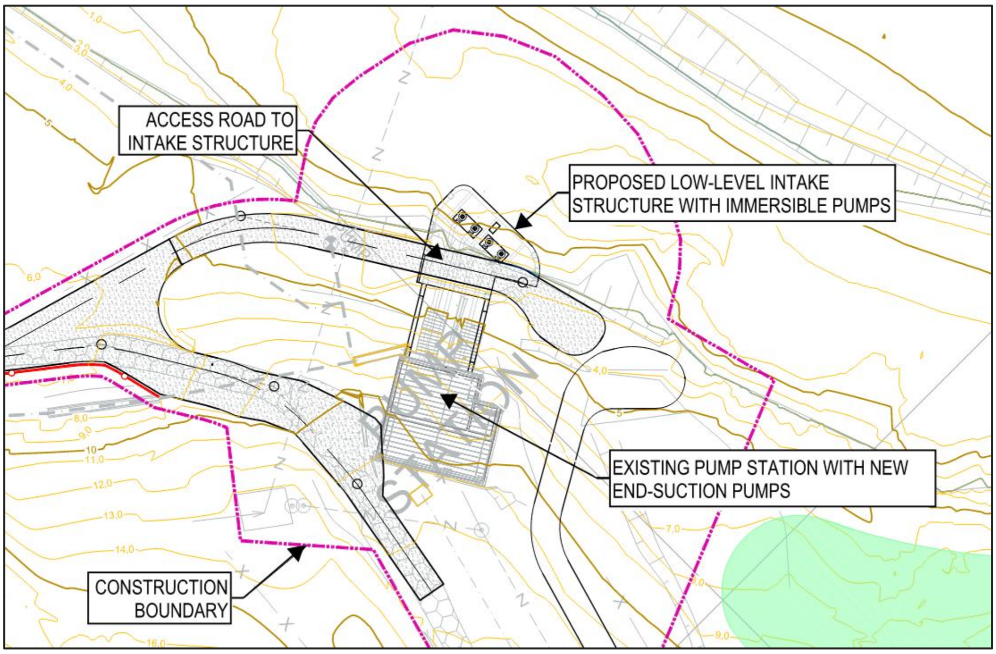

Figure 2-1: Preliminary Pump Station Layout (2014 – 2016)

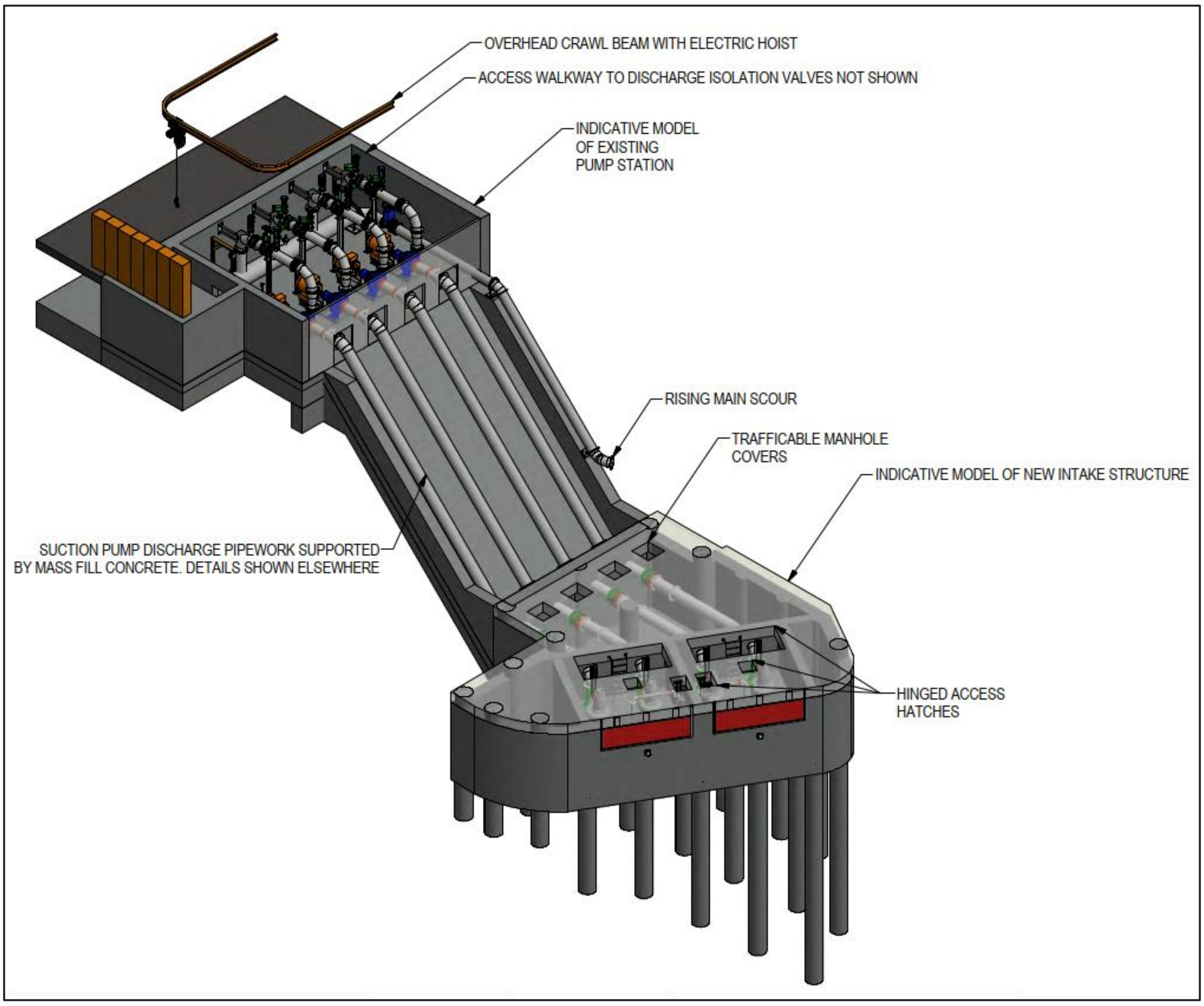

Figure 2-2: Preliminary Pump Station Isometric (2014 – 2016)

2.2 Verification Study

Upon recommencement of the project in late 2024, an underwater survey was undertaken to assess changes in the riverbed topography since 2014. ASP Tech, who undertook the initial sedimentation study, was subsequently appointed by Lyners to conduct a Verification Study to evaluate, among other factors, the appropriateness of the original intake structure in light of the updated bathymetry survey. The resulting report is provided as Appendix A.

The key findings from this verification study in relation to the initial solutions are as follow:

- There is an upstream rock obstruction that resulted in a significant amount of sediment deposition at the location for the existing pump intakes, which location was previously proposed for the new intake structure (refer to Figure 2-3). It is recommended that the rock outcrop be removed as the sediment deposition will influence the effectiveness of the intake structure.
- The width of the hoppers upstream of the pump intakes must increase from 2m to 4m to improve sediment removal and to account for the higher sediment loads in the river.

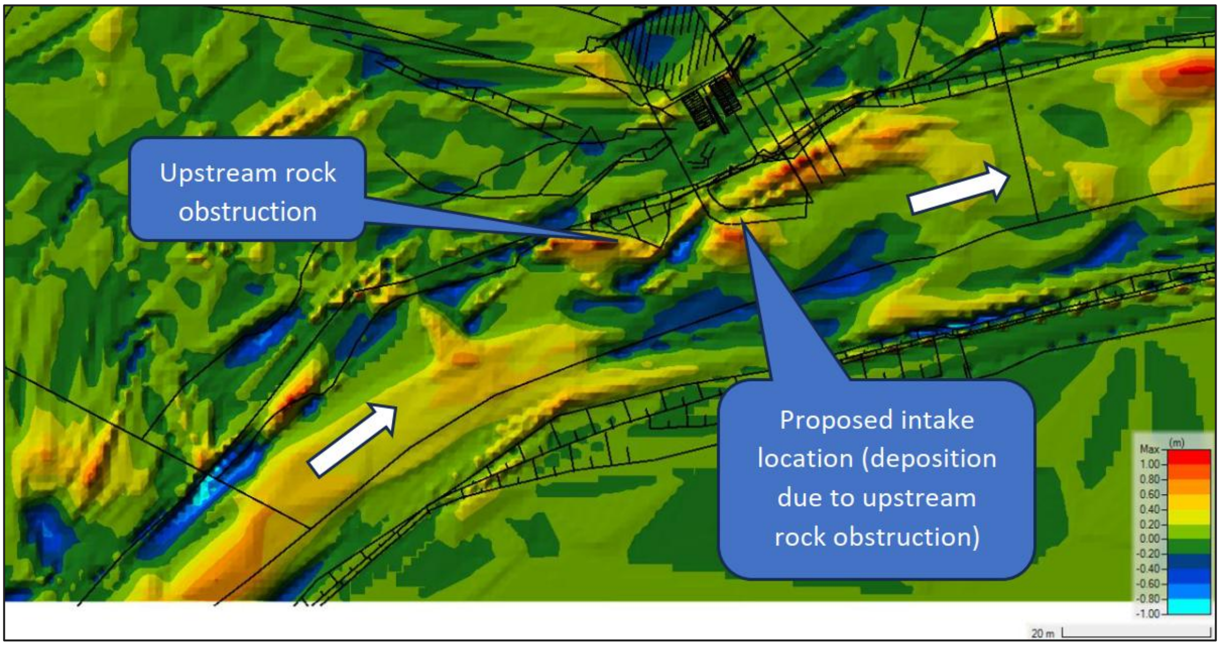

Figure 2-3: Riverbed topography changes from 2014 to 2025 (Red = deposition, blue = scour) (provided by ASP Tech)

The report further included comment on pumping options, noting the following:

- Consideration can be given to a single stage pump solution as the intake structure size can be increased to accommodate larger immersible pumps. KSB KRT 200-402 immersible pumps, or similar, installed inside the intake structure was proposed as a possible solution. Also refer to Section 2.3.1.1.
- End-suction pumps are generally considered more suitable to potable water applications as oppose to pumping raw water that could contain sediment and debris.

With reference to the comment on the end-suction pumps' suitability to pump raw water, it was observed on site that the farmers immediately downstream of the intake structure are using end-suction pumps. In discussion with one of the farmers, he indicated that the pumps required minimal maintenance. Furthermore, the introduction of the intake structure will limit the size of sediment and debris that can reach the pumps. While waste water pumps will be more suited to pump raw water, end-suction pumps are commonly used for river abstractions throughout the country.

2.3 Final Option Selection

Based on the findings of the Verification Study and subsequent engagements with DWS, MBM and other stakeholders, it was agreed that further consideration was required for the location of the intake structure as well as the pump type selection. This section provides an overview of the additional considerations, outlines the revised options that were developed, and describes the process for selecting the final option.

2.3.1 Additional Considerations

2.3.1.1 System Configuration

Single stage pumping systems, in place of the initially proposed two-stage (i.e. in-series) pumping systems, present the following operational and strategic benefits:

- **Simplified control and operation** – Single-stage systems eliminate the need for coordination between the two in-series pumps. This significantly reduces the risk of pumps running dry or operating at shut-off head for extended periods of time due to faulty instrumentation.
- **Reduced Maintenance Requirements** – With only four pumps in operation compared to eight pumps in the two-stage setup, maintenance requirements are much less. This is likely to translates to lower labour costs, fewer spare parts, and reduced downtime for servicing.
- **Improved System Reliability** – Fewer pumps with the same reliabiltiy mean fewer potential points of failure. This enhances overall system reliability.

The disadvantages of a single-stage system are as follow:

- The pumps needs to be installed low enough to satisfy the NPSH requirements of the selected pumps.
- The pumps will be installed below the 1:100-year flood level, which could cause flooding of the pump installation.
- If the pumps are installed above the minimum water level, it requires a priming system consisting of vacuum or priming pumps that requires a high level of maintenance.
- If the pumps are installed above the minimum water level, it requires foot valves on the suction pipe. These valves are prone to malfuntion due to floating debris and gravel that get stuck in the valve and prevent proper closure of the valve and thus increase priming problems.

To overcome the disadvantages noted in the last two points above, immersible pumps, such as the KSB KRT 200-402 pumps proposed by ASPTech, in lieu of the initially proposed two-stage system is an option. Although this solution offer certain advantages, using an immersible pump of this size introduces certain technical challenges that require further consideration. The proposed KSB KRT 200-402 pump measures over 2 m in height, as shown on Figure 2-4, and weighs 1,367 kg. Due to its large dimensions and weight, special lifting provisions will have to be made at the intake structure. Furthermore, due to the close-coupled motor configuration, and specific sealing requirements for maintaining a watertight installation, this size of pump necessitates specialised maintenance procedures that are rarely available within South Africa. This is aggravated by the fact that the pump will be located inside the intake structure with restricted access. Furthermore, each immersible pump cost approximately R1.1 million compared to approximately R335 000 for an end suction pumpset that can achieve the same duty point. Therefore, the end suction pumpsets can be replace at least 3 times at the cost of the immersible pumps.

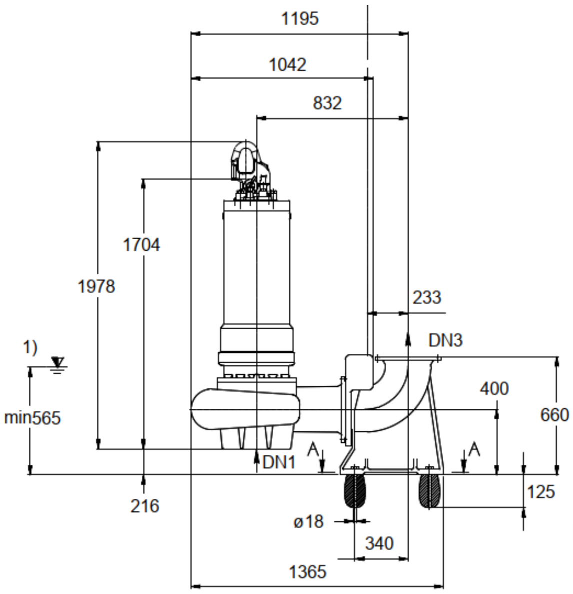

Figure 2-4: KSB KRT 200-402 Dimensions

DWS and MBM noted a preference for end-suction pumps due to their simplicity to maintain and the fact that DWS have experience in maintaining these pumps themselves.

However, as noted during the initial phases (2014 – 2016) of the project, the floor level of the current pump station is too high for the installation of end-suction pumps without the addition of immersible pumps located at the intake structure. **As such, an alternative system configuration, that maintains the benefits of a single-stage pumping system, as well as the benefits of using end-suction pumps were investigated. This alternative entails the construction of a drywell structure next to the current pump station but at a suitable level to enable end-suction pumps to be used in a single-stage pumping system.** This alternative is discussed further in the following sections.

It should, however, be noted that DWS procured the immersible and end-suction pumps after the 2014 – 2016 study with the intention to free-issue the pump to the installation contractor during the construction phase. The project was, however, suspended before construction commenced. The procured pumps are still available for installation.

2.3.1.2 System Layout

Due to the requirement for removal of the upstream rock outcrop, in terms of river sediment transport, and considerations about the founding of the intake structure in the initial solution (such as the potential need for piles to achieve suitable founding), an alternative layout for the intake structure upgrade was developed. This layout places the intake structure directly on the rock outcrop. As shown on Figure 2-5, if the intake structure is to remain in the current position (Concept Layout 1), the drywell proposed in the previous section, will be placed north of the existing pump station. If the intake structure is moved to the rock outcrop (Concept Layout 2), it is proposed to place the drywell to the east of the current pump station building.

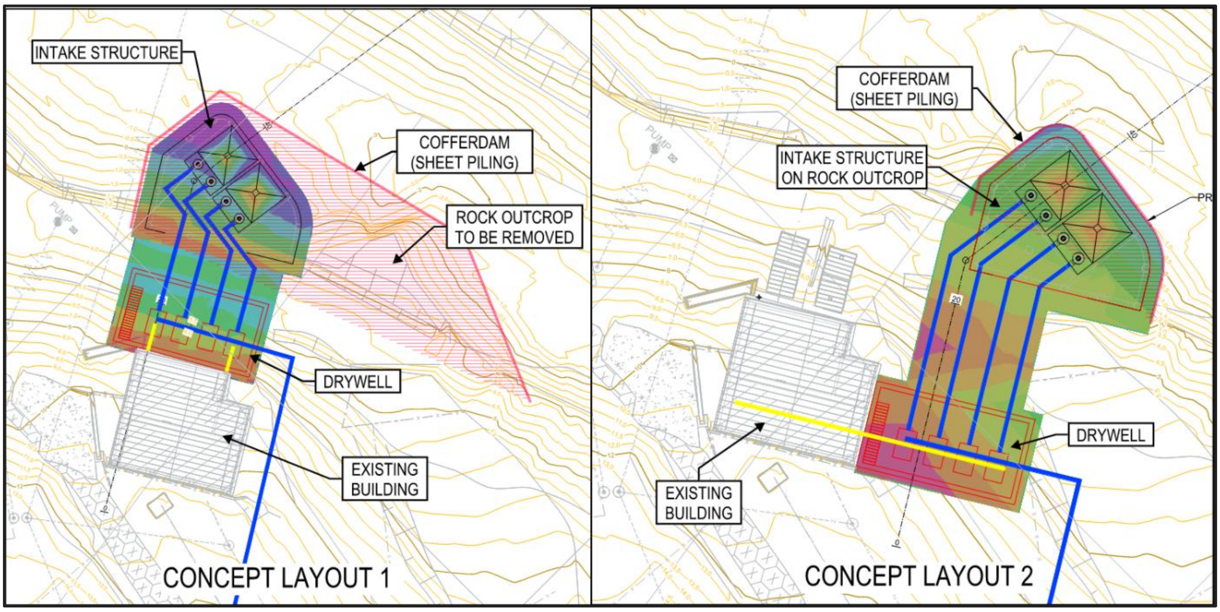

Figure 2-5: Concept Layout Options

The main advantages of the revised Concept Layout 2 option are as follow:

- Due to the rock outcrop, it is likely that this option will provide better and more economical founding for the abstraction works (to be confirmed with a supplementary geotechnical investigation);
- There is a reduced risk that the intake structure would affect the current dynamics of the river, such as causing scouring of the opposite bank. The structure would essentally replace the existing rock and not cause a localised narrowing of the river as is the case for the initial solution (Concept Layout 1);
- The existing pump station can remain operational during the construction of the intake structure and drywell; and
- For Concept Layout 2, the pumps installed in the drywell can be serviced by a single crawl beam and hoist, whereas a more complicated lifting equipment arrangement will be required for Concept Layout 1.

2.3.1.3 Water Treatment Considerations

Klipheuwel Dam is the raw water source of preference for Mossel Bay Municipality due to its lower treatment costs compared to Wolwedans Dam, which is the other main water source. Table 2-1 presents a cost comparison between using water from Klipheuwel Dam and Wolwedans Dam for Mossel Bay Municipality's bulk water supply, based on data from May 2022 to May 2025. The analysis indicates a **saving of R3.93/kL** when sourcing water from Klipheuwel Dam.

Flow meter records, for the same period, show an average monthly consumption of 347 017 kL from Klipheuwel Dam. If Moordkuil Pump Station is taken out of operation during the upgrade, and water is sourced from Wolwedans Dam instead, the bulk water supply will cost nearly **R1.4 million more per month**, totalling to **R24.5 million** over an 18-month construction duration. The benefit of Concept Layout 2, which will allow the existing pump station to remain operational during the construction phase, is significant in terms of the overall project costs.

Table 2-1: MBM Bulk Water Supply Cost Comparison

	Klipheuwel Dam	Wolwedans Dam
Approximate Treatment Cost (R/kL)	R8.26	R11.5 - R12.19 (R11.85 average)
Raw Water Tariffs (R/kL)	R3.014	R3.358
Total Cost (R/kL)	R11.274	R15.208

2.3.2 Option Selection

Taking into account the additional considerations outlined in the previous section, three final implementation options were evaluated:

- **Option 1** – Original concept (2016 report) with two stage pumping and without a drywell, as proposed during the initial project phases
- **Option 2** – Inclusion of a drywell with single stage end suction pumps as per Concept Layout 1
- **Option 3** – Inclusion of a drywell with single stage end suction pumps as per Concept Layout 2

Table 2-2 presents a comparative costing of the three options.

Table 2-2: Comparative Costing

		Option 1: Original Concept	Option 2: Concept Layout 1	Option 3: Concept Layout 2
Intake structure		R7,018,300	R7,018,300	R5,575,900
Drywell		R0	R2,417,620	R2,362,120
Mechanical Works		R8,420,760	R8,420,760	R8,995,760
Electrical Works		R4,700,000	R4,700,000	R4,400,000
Access Road [1]		R2,134,598	R2,134,598	R2,134,598
Meter Chamber and Air Valves [1]		R2,955,500	R2,955,500	R2,955,500
Rising Main Pipe [1]		R432,550	R432,550	R475,175
Temporary Pumping [2]		R2,900,000	R2,900,000	R0
P&Gs	30%	R8,568,512	R9,293,798	R8,069,716
Sub Total		**R37,130,220**	**R40,273,126**	**R34,968,768**
Add Forward Escalation on Civils	8%	R1,003,276	R1,196,685	R1,080,263
Add Forward Escalation on M&E	16%	R2,099,322	R2,099,322	R2,143,322
Sub Total		**R40,232,817**	**R43,569,133**	**R38,192,353**
Contingencies	20%	R8,046,563	R8,713,827	R7,638,471
Total (Excluding VAT)		**R45,176,783**	**R48,986,952**	**R42,607,239**

Note 1 – The cost estimates for these items were provided by Lyners in accordance with their civil design.
Note 2 – This value account for the temporary pumping system that will be required for options 1 and 2 to mitigate the additional water treatment cost (see section 2.3.1.3)

Option 3 is considered the most economical, followed by Option 1. While Option 3 includes additional costs related to the drywell, it benefits from significantly reduced costs for the intake structure – primarily due to the smaller cofferdam required (refer to Figure 2-5). The difference in mechanical and electrical (M&E) costs as shown in Table 2-2 is attributed to temporary pumping requirements during construction to ensure continues water supply to Mossel Bay Municipality.

Table 2-3 summarises the key advantages and disadvantages of each option. Option 3 is preferred from an Operational, Technical, Construction and Stakeholder perspective. Although Option 3 has a larger construction footprint, it has a lower long-term impact on the river dynamic, which makes this option beneficial from an environmental perspective.

Table 2-3: Option Comparison

Considerations	Option 1: Original Concept	Option 2: Concept Layout 1	Option 3: Concept Layout 2
Operational	•Complex two-stage system with higher maintenance and control requirements. •Specialised maintenance on the immersible pump.	•Simplified single-stage system with end-suction pumps. •Improved reliability. •End-suction pumps are easy to maintain	•Simplified single-stage system with end-suction pumps. •Improved reliability. •End-suction pumps are easy to maintain
Technical	•Existing pump station offers limited space for the mechanical and electrical equipment associated with the upgrade. •Complex lifting equipment arrangement inside the pump station to service all booster pumps.	•Includes the addition of a drywell for the mechanical installation •Existing pump station can be used for electrical installation. •Complex lifting equipment arrangement inside the pump station to service all pumps.	•Includes the addition of a drywell for the mechanical installation •Existing pump station can be used for electrical installation. •All pumps can be serviced by a single crawl beam.
Construction	•Requires pile foundations for the intake structure. •Existing pump station will be taken out of operation for the entire construction period. •Rock outcrop to be demolished.	•Requires pile foundations for the intake structure. •Restricted construction for the drywell between the existing pump station and the river •Existing pump station will be taken out of operation for the entire construction period. •Rock outcrop to be demolished.	•Intake structure founded on rock outcrop •More space available for drywell construction, but drywell will be deeper. •Existing pump station can remain operational for most of the construction period.
Environmental	•Smallest construction footprint. •Intake structure narrows the existing river and can potentially negatively impact on river dynamics.	•Slightly larger construction footprint. •Intake structure narrows the existing river and can potentially negatively impact on river dynamics.	•Largest construction footprint. •Intake structure location on rock outcrop minimises the narrowing and potential impact on river dynamics.
Stakeholder	•Relocation required of the farmer's pumps downstream of the intake structure to prevent the intakes from silting up.	•Relocation required of the farmer's pumps downstream of the intake structure to prevent the intakes from silting up.	•Farmer's pumps can remain in place.

Based on the advantages noted in Table 2-3, along with its lower overall cost, Option 3 was selected for implementation. To maximise the value of the existing pumps already procured, a phased implementation strategy will be adopted:

- **Phase 1** – Utilisation of existing pumps (immersible pumps in the hopper/intake structure and end-suction pumps in the drywell); and
- **Phase 2** – Replacement of the immersible pumps with a single stage of end-suction pumps installed in the drywell.

It should further be noted that a supplementary geotechnical investigation will be required for the revised location for the intake structure and the new drywell. This investigation will form part of the detailed design phase of the project.

3 Hydraulic Design

3.1 Intake Structure

After the initial Verification Study, ASPTech updated their report to included 2D hydrodynamic modelling of the sediment dynamics expected for the new location of the intake structure on the rock outcrop as per Concept Layout 2 in Figure 2-5.

For ease of reference, the key findings from this modelling, as documented in Appendix A, are summarised below:

- The proposed intake is situated within the scour zone on the outer side of the river bend and will be self-scouring during both minor and major flood events;
- The height of the proposed intake remains low and submerged during floods, so it does not divert flow towards the right bank. Simulations show that the opposite right bank is a sediment deposition zone, so erosion protection is not required here;
- The left bank and floodplain between the proposed intake and the causeway are subject to scouring during 10-year and 100-year floods. Erosion in this area should be monitored, and infrastructure in critical locations requires protection from scour; and
- The proposed intake location on the left bank bedrock upstream of the existing pump station should be used for the detailed design of the new river intake structure.

3.2 Pump Station

The minimum, normal and maximum system curves were calculated for the system. The minimum system curve scenario will occur when the pipeline is still new (i.e. smooth), using a roughness coefficient (k_s) of 0.03 mm, and when the river level is high, conservatively assumed at 8.5 masl (note, the 1:100-year flood level is estimated at 8.2 masl). The normal system curve scenario represents typical operating conditions throughout most of the pipeline's lifespan, with a k_s value of 0.15 mm and a river level of 2 masl. The maximum system curve scenario will occur when the pipeline is aged (i.e. rough), using a k_s value of 0.6 mm, and when the river level is at its lowest minimum recommended operating water level of 1.95 masl.

Figure 3-1 illustrates the system curves for the minimum, maximum and normal operating conditions. Figure 3-1 also shows the recommended duty point for the pump station duty point of 800 ℓ/s @ 34 m (i.e. 200 ℓ/s per pump @ 34 m head). It is recommended that the duty point is selected on the maximum system curve to ensure that the pump station can deliver the design flow for its entire design life.

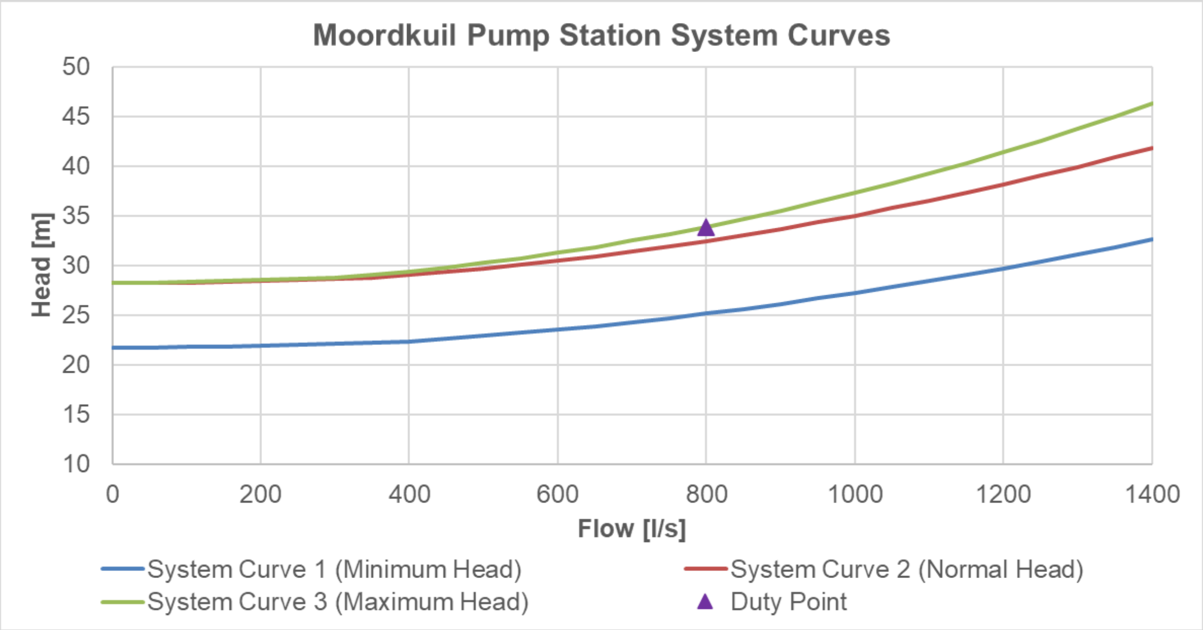

Figure 3-1: Moordkuil Raw Water Pump Station system curves

4 Civil Design

Figure 4-1 gives the floor plan for the proposed Moordkuil pump station upgrades. The design considerations for each of the main civil components are discussed in further detail in this section.

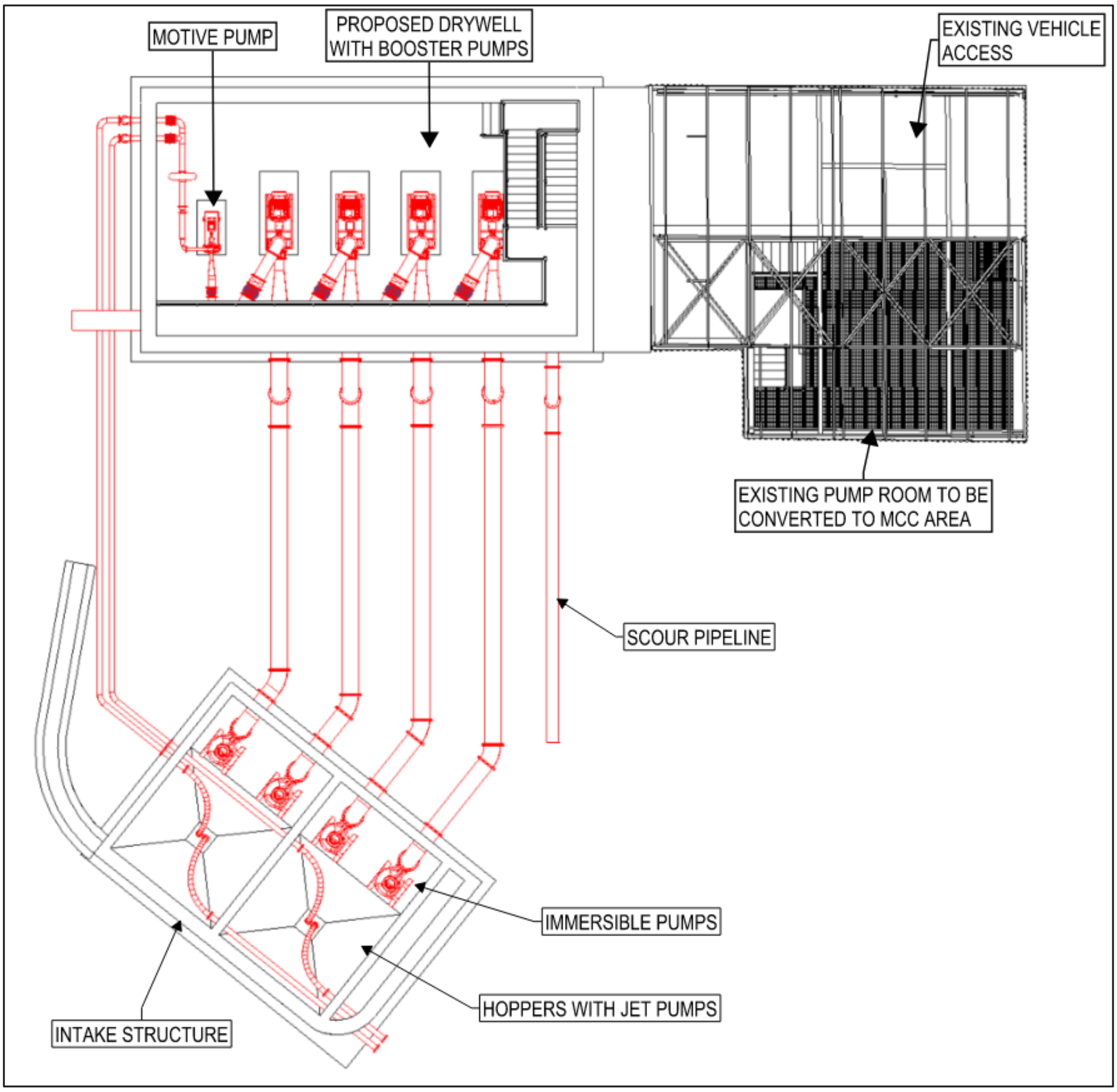

Figure 4-1: Pump station floor plan

4.1 Intake Structure

This section summarizes the design for the intake structure, which was based on the recommendations by ASP Tech during their initial investigations as well as the subsequent hydraulic verification.

Figure 4-2 shows the positioning of the intake structure on the existing rock outcrop within the natural narrowing of the river.

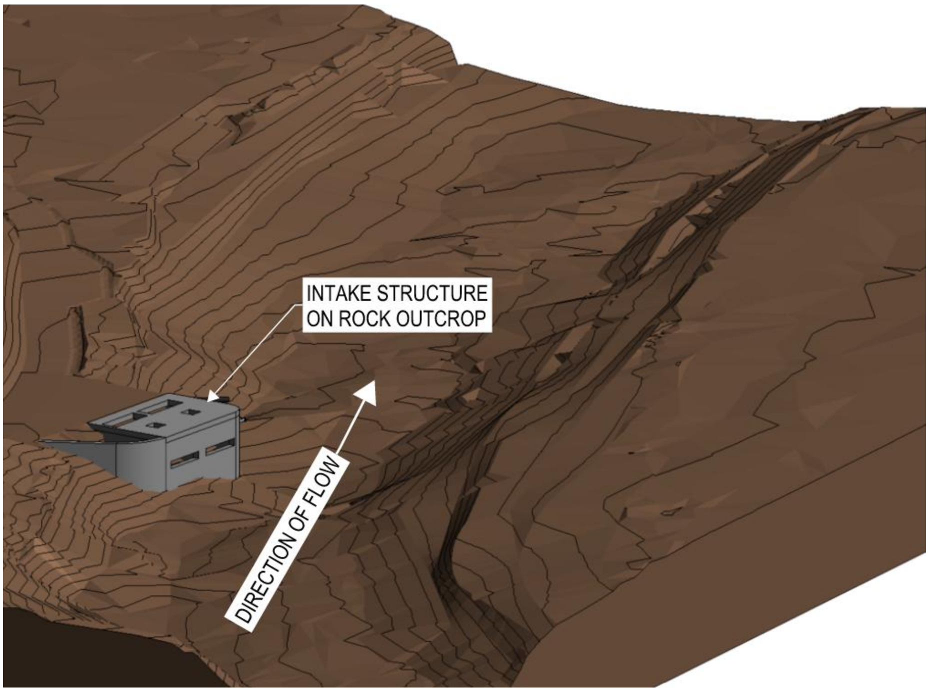

Figure 4-2: Intake structure positioning

Figure 4-3 and Figure 4-4 illustrate the plan layout and sectional view of the intake structure.

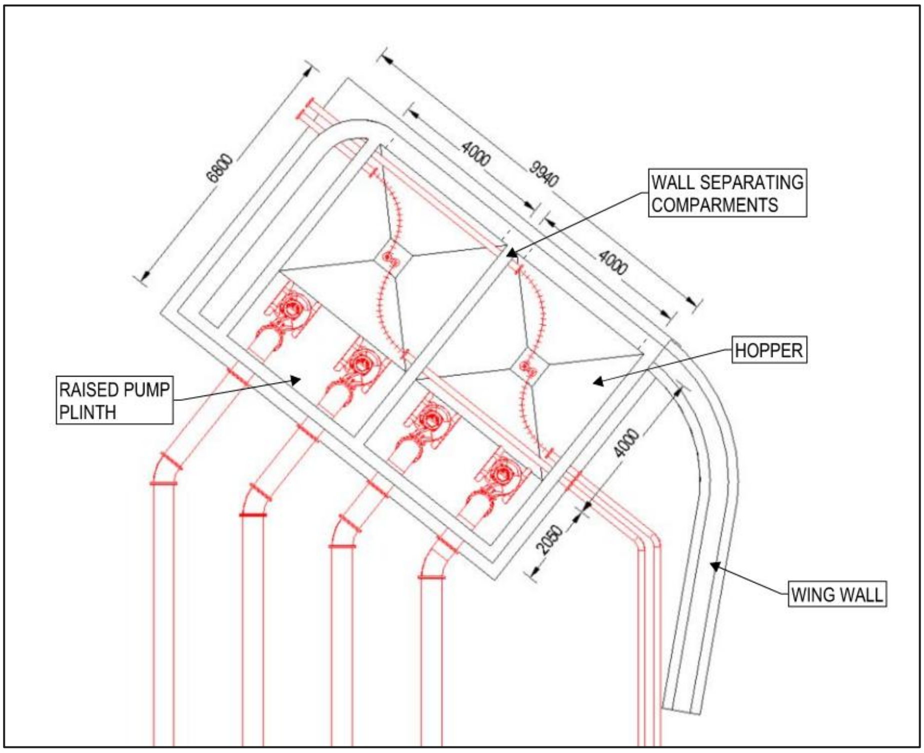

Figure 4-3: Intake structure plan layout

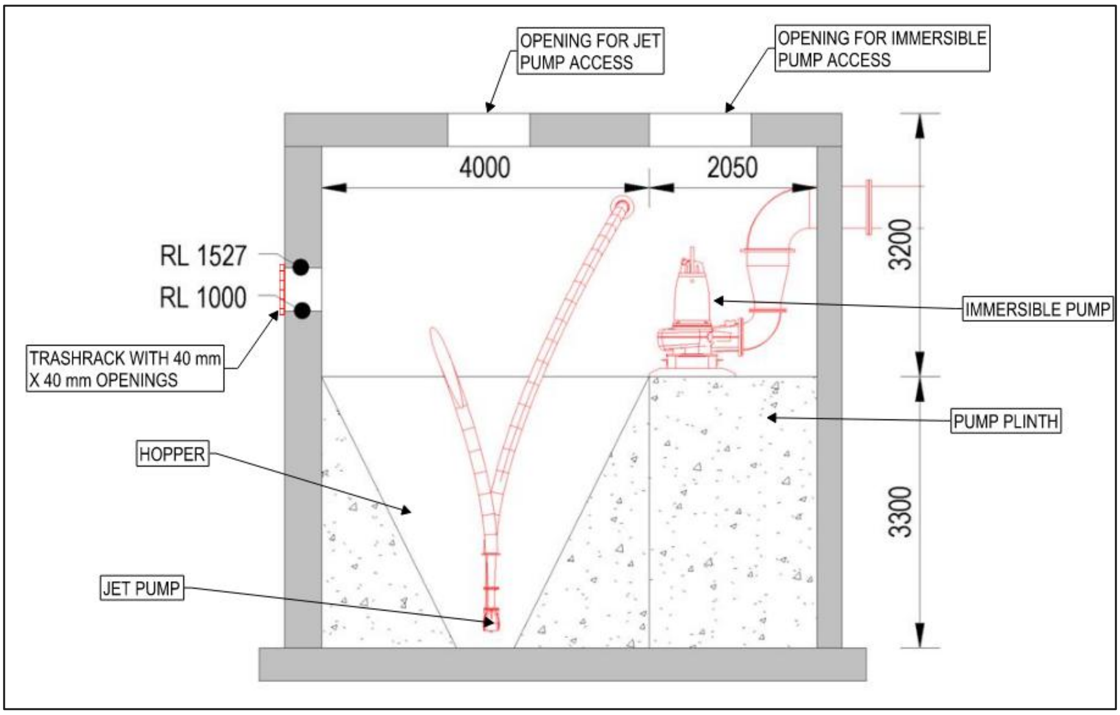

Figure 4-4: Intake structure sectional view

The design comprises two independent compartments to ensure operational redundancy. Each intake opening measures 527 mm in height and 3.4 m in width, and will be fitted with a trashrack featuring 40 mm x 40 mm openings.

To facilitate maintenance, each compartment will be equipped with a stoplog system allowing for isolation when required.

As recommended in the Verification Study, the sediment collection hoppers located upstream of the immersible pumps will be enlarged to dimensions of 4 m (width) × 4 m (length) × 3.3 m (depth). Jet pumps will be used to clean sediment from these hoppers.

The existing immersible pumps are mounted on dedicated pump plinths. When these pumps are eventually replaced, foot valves will be installed in place of the current immersible pumps.

Access to the immersible and jet pumps will be provided through roof openings, which will be sealed using removable precast concrete slabs. These slabs will be secured in place with lockable stainless-steel bars.

4.2 Drywell

The proposed drywell will be located to the east of the existing structure as per Concept Layout 2 (refer to Figure 2-5). Figure 4-5, Figure 4-6 and Figure 4-7 show the isometric, plan and sectional views for the proposed drywell.

The width of the drywell was selected at 8.2 m to match the width of the upper level of the existing building. The length of the structure will be 13.5 m to accommodate the four end-suction pumps as well as the motive pump required to drive the two jet pumps.

The floor level for the drywell will be at RL 4.5 m based on the net positive suction head (NPSH) requirements of the proposed end-suction pumps (see Section 5.3). The drywell will feature a reinforced concrete lower structure supporting a steel upper structure. It is proposed that the concrete lower section will extend from the foundation up to the upper floor level of the existing pump station (RL 10.5 m). The steel upper structure will be designed to integrate seamlessly with the existing building's steel framework. The concrete lower portion will be constructed to be watertight, ensuring protection against water ingress during river flood events; the 1:100-year flood level is anticipated to reach approximately RL 8.2 m.

It is proposed to place the drywell at an offset of at least 2 m from the edge of the existing structure due to the level difference between the two structures. This is to ensure sufficient space is available for temporary lateral support and to ensure that the existing structure does not get undermined.

The drywell will be accessed from the existing pump station through a stairway. It is proposed that the stairway has two landings, of which one leads onto an intermediate walkway to access the handwheels of the discharge valves. It is proposed to construct the stairway and walkway from steel with GRP grating. This will also allow for simple and relatively quick construction.

The drywell will incorporate a drainage channel and sump pump to mitigate the risk of flooding due to a leaking coupling. An alternative option is a free drainage outlet with a non-return valve to prevent water from pushing back into the drywell when the river is in flood. The detailed design phase will include a detailed comparison between these options.

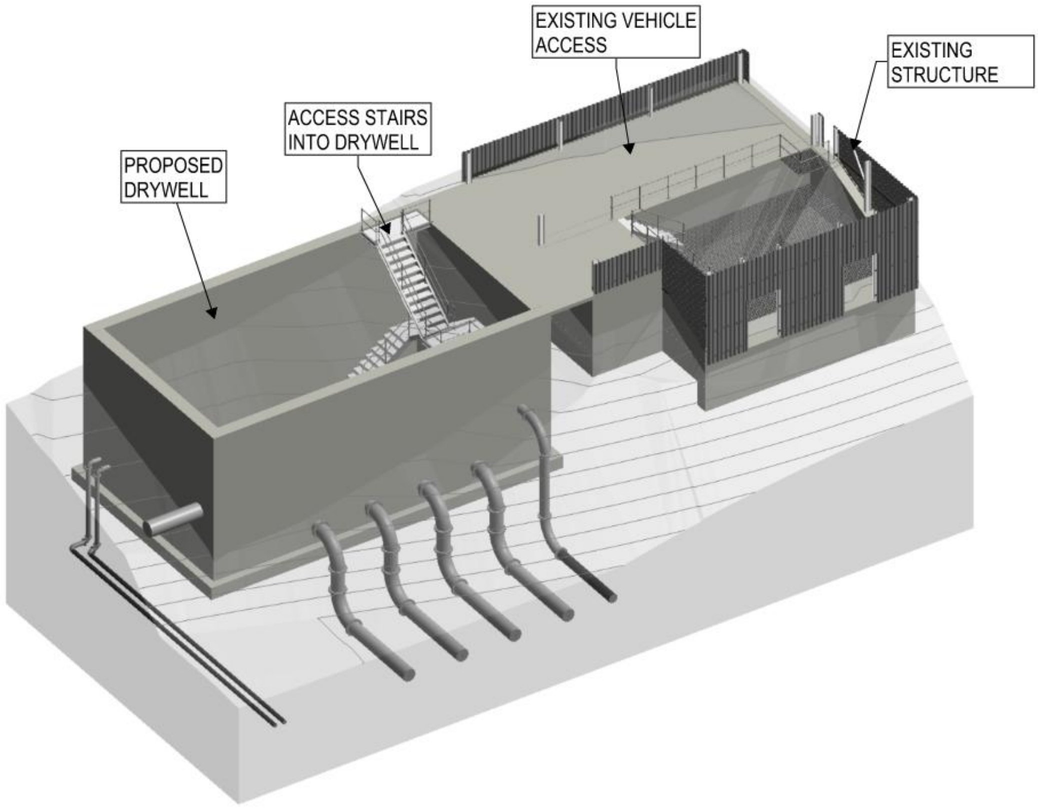

Figure 4-5: Drywell isometric view

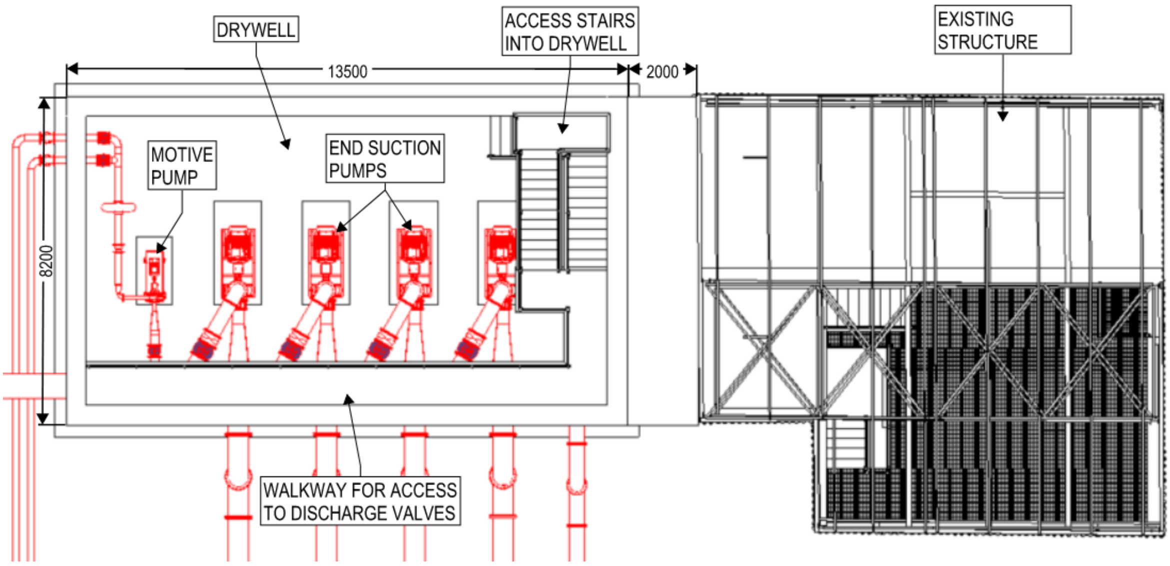

Figure 4-6: Drywell plan layout

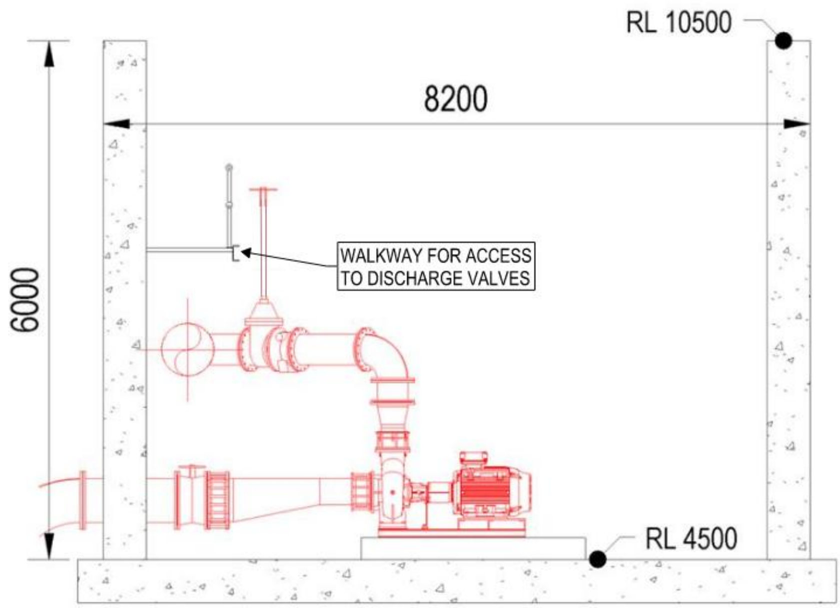

Figure 4-7: Drywell sectional view

4.3 Modification to existing structure

The modification of the existing structure will include the following:

- Removal of the eastern cladding for tying in with the drywell steel top structure;
- Replacement of the existing U-shaped crane beam with a single crane beam into the drywell; and
- The construction of a false floor above the existing pump floor for the electrical equipment.

Figure 4-8 shows the cladding that will be removed from the existing building's eastern face and where the steel top structure for the drywell will tie-in. Figure 4-8 also shows that the roof of the existing structure falls towards the drywell. As such, the roofs for the existing steel top structure and for the drywell will have to be tied together with a suitably sized central drainage channel.

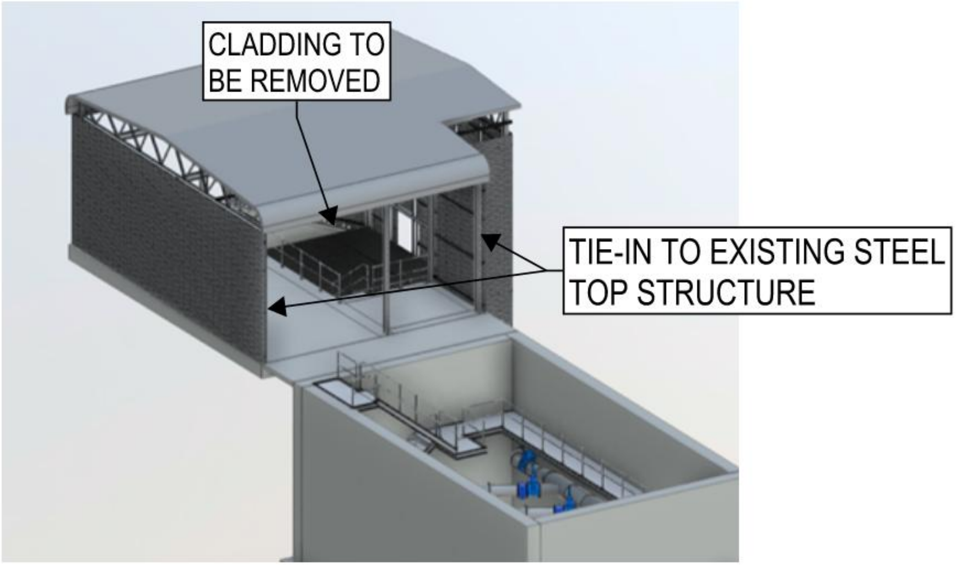

Figure 4-8: Existing structure cladding to be removed

Figure 4-9 shows the proposed crane beam that will be installed to service the pumps inside the drywell. The installation will allow for the existing vehicle bay to be utilised to load the pumps.

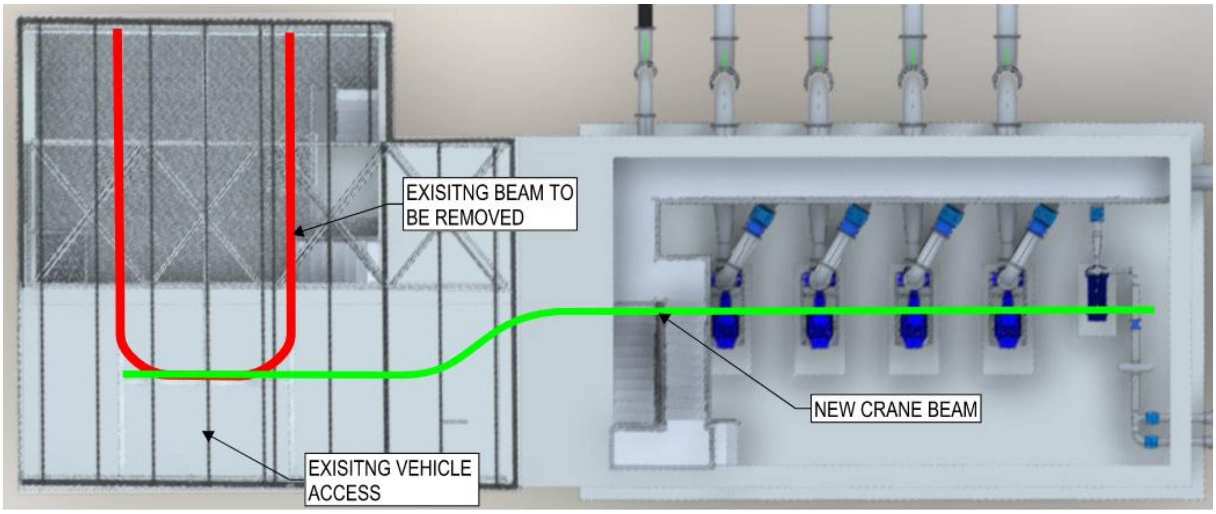

Figure 4-9: Crane beam modifications

Figure 4-10 and Figure 4-11 indicatively show the proposed false floor to be installed above the existing pump floor to house the electrical equipment. As the current pump floor is at RL 8 m – close to the 1:100-year flood level – the new false floor will be at least 1 m above the existing floor level to reduce the flooding risk for the electrical equipment. This elevation also keeps the false floor above the reinforced concrete motor plinths, mitigating the need for these plinths to be demolished.

It is proposed that the false floor is constructed from steel supports with solid GRP panels. The crawl space below the false floor will be used for cable routing to and from the MCC units.

Additionally, a cut-off wall is planned on the river side of the structure for extra flood protection.

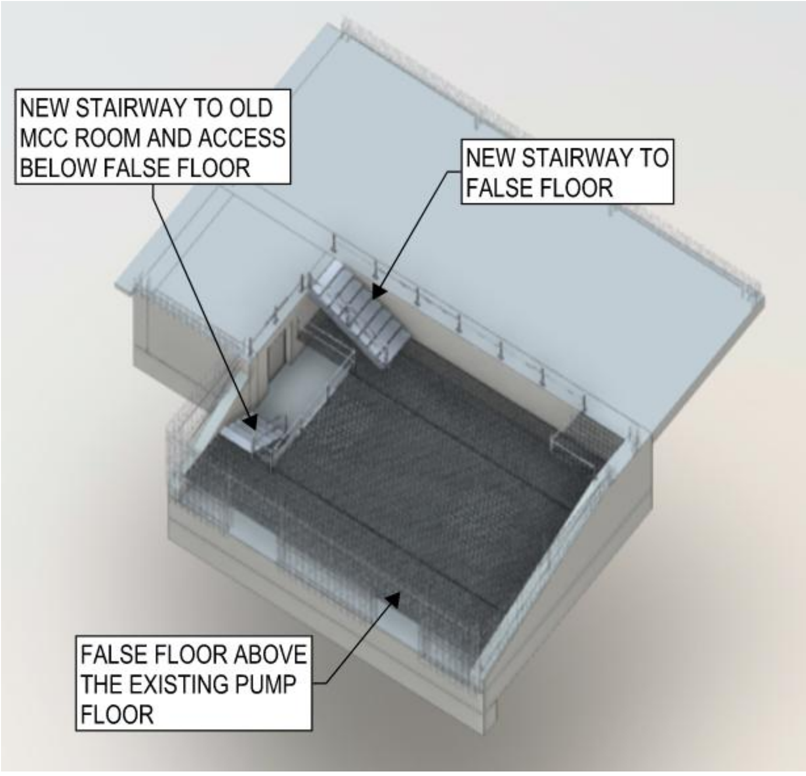

Figure 4-10: Proposed false floor for electrical equipment – isometric view

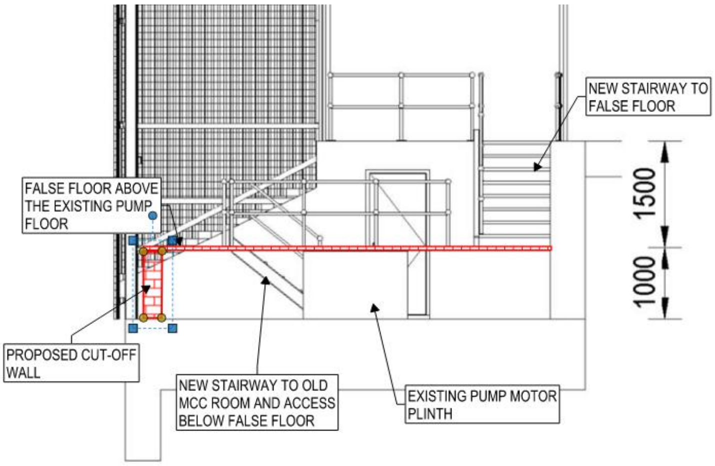

Figure 4-11: Proposed false floor for electrical equipment – section view

Figure 4-12 illustrates examples of this type of raised floor arrangement which is commonly implemented within a data centre environment.

Figure 4-12: False floor example typically used in data centres

5 Mechanical Design

5.1 Design Philosophy

As noted in Section 2.3.2, the mechanical design for the Moordkuil Pump Station upgrade will involve a phased approach so that the existing pumps can be utilised as part of the first phase of the project.

In Phase 1, the existing pumps will be configured to operate in series, with each end suction and submersible pair producing 200 ℓ/s. The pumps will operate in a 4 duty, 0 standby, configuration to deliver a total flow of 800 ℓ/s under normal operating conditions.

In Phase 2, the arrangement will be simplified by removing the immersible pumps and installing foot valves onto the inlet pipework in the intake structure. The existing end-suction pumps will be replaced with larger end-suction pumps, which will draw directly from the intake structure via its individual suction lines. Each end suction pump will then independently deliver 200 ℓ/s in a 4 duty, 0 standby, configuration supplying a total flow of 800 ℓ/s.

To facilitate this transition, the pump plinths have been designed to accommodate the larger end suction pumps that will be installed in Phase 2. This staged approach ensures the efficient use of the existing pumps with minimal modifications required when the Phase 2 pumps are installed.

The mechanical installation discussed in the following section are primarily for the Phase 1 installation. The detailed considerations for the phase 2 system, such as the operation of the foot valves and the suction pipework priming, will be included in the detailed design report.

5.2 Pump Selection

The pump selection for the Moordkuil Pump Station was done in conjunction with the options analysis and phasing discussed in Section 2. Table 5-1 below summarises the details of the existing pumps (Phase 1) and the outcome of the pumps selected for Phase 2.

Table 5-1: Pump Analysis

Description	Submersible Pumpset	End Suction Pumpset	End Suction Pumpset	Motive Pumpset
Phase	1	1	2	1 and 2
Orientation	Vertical	Horizontal	Horizontal	Horizontal
Configuration	4 duty, 0 standby	4 duty, 0 standby	4 duty, 0 standby	1 duty, 0 standby
Duty Flow (ℓ/s)	200		200	30
Duty Head (m)	37		33.9	37 (2)
Make and Model (1)	Flygt NS 3202 LT 3~ 614 (existing)	Lowara NSCF 250-315/750X/W45VDC4 (existing)	KSB ETA 250-40 (future)	KSB Etanorm 100-080-315 (future)
Efficiency at Duty Point	80%	82%	83%	78%
Absorbed Power at Duty (kW) per pump	24.5	64	80.5	14
Total Absorbed Power at Duty (kW)	98	256	322	14
Motor size (kW)	30 3	75 3	110	22
Total Installed Capacity (kW)	120	300	440	22

Note 1 – The listed pumps' make and model for the future phase are examples only; other manufacturers or models that meet the performance specifications are also available in the market.
Note 2 – The motive pump will be supplied from the discharge side of the end-suction pumps and will therefore deliver an effective head of ± 70m.
Note 3 – These are the motor sizes for the existing pumps.

For the conceptual design considerations, the new pump motors were sized with an 25% spare capacity margin above the required absorbed hydraulic power to prevent overloading under varying operating conditions and to ensure reliable operation. A more generous spare capacity is generally recommended for motors started Direct-on-Line (DOL). It should be noted that, for Phase 1, the spare capacity available in the existing pumps is less than the 25% standard specified for the new pumps. The immersible pump motors have a spare capacity of 22%, while the end-suction pump motors provide 17% spare capacity at the duty point. More advance start-up procedures might be required to ensure reliable operation of these existing pumps. This will be investigated further in the detailed design phase.

It is also noted that the existing Lowara end-suction pumps are generally proposed for clean water applications. The additional measures, such as mechanical seal flushing water, that may be required in order to utilise these pumps in a raw water application will be investigated further during the detailed design phase.

5.2.1 Phase 1: Existing Pumps

Phase 1 will utilise the already procured Flygt submersible pumps in series with the Lowara end-suction pumps. The combination of the existing pumpsets will deliver a flow of 800 ℓ/s at a pressure head of 37 m. The combined performance curves for each pump pair, Lowara & Flygt NS 3202 pumps, are given in Figure 5-1. The combined performance curves were generated by summing the individual pump delivered head by each pump for a specific flow rate.

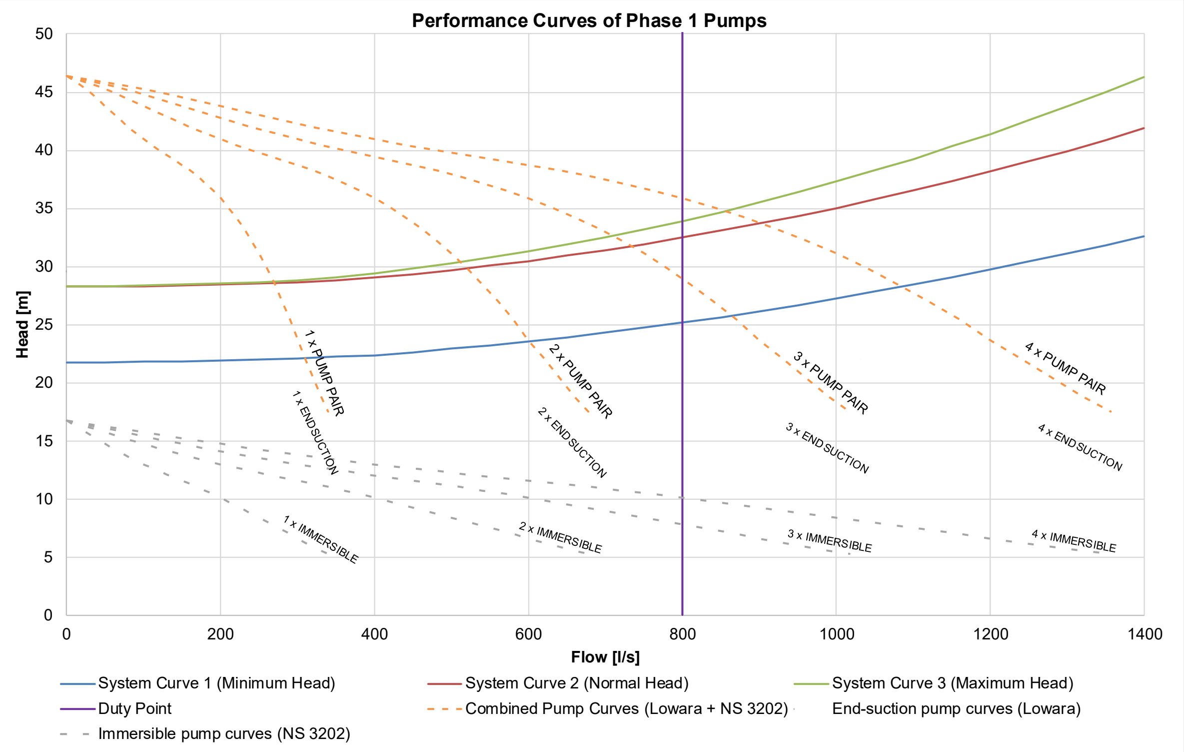

Figure 5-1: Performance Curve for Phase 1 Pumps

Table 5-2 below provides the operating scenario flow rates for all four pump configurations.

Table 5-2: Flow Rates for Operating Scenarios of Phase 1 Pumps

Operating Scenario	Minimum Flow (ℓ/s)	Normal Flow (ℓ/s)	Maximum Flow (ℓ/s)
1 x pump	270	271	310
2 x pumps	520	530	600
3 x pumps	720	750	870
4 x pumps	880	980	1080

5.2.2 Phase 2: Proposed Future Pumps

The performance curves for the phase 2, KSB ETA end suction pumps are given in Figure 5-2.

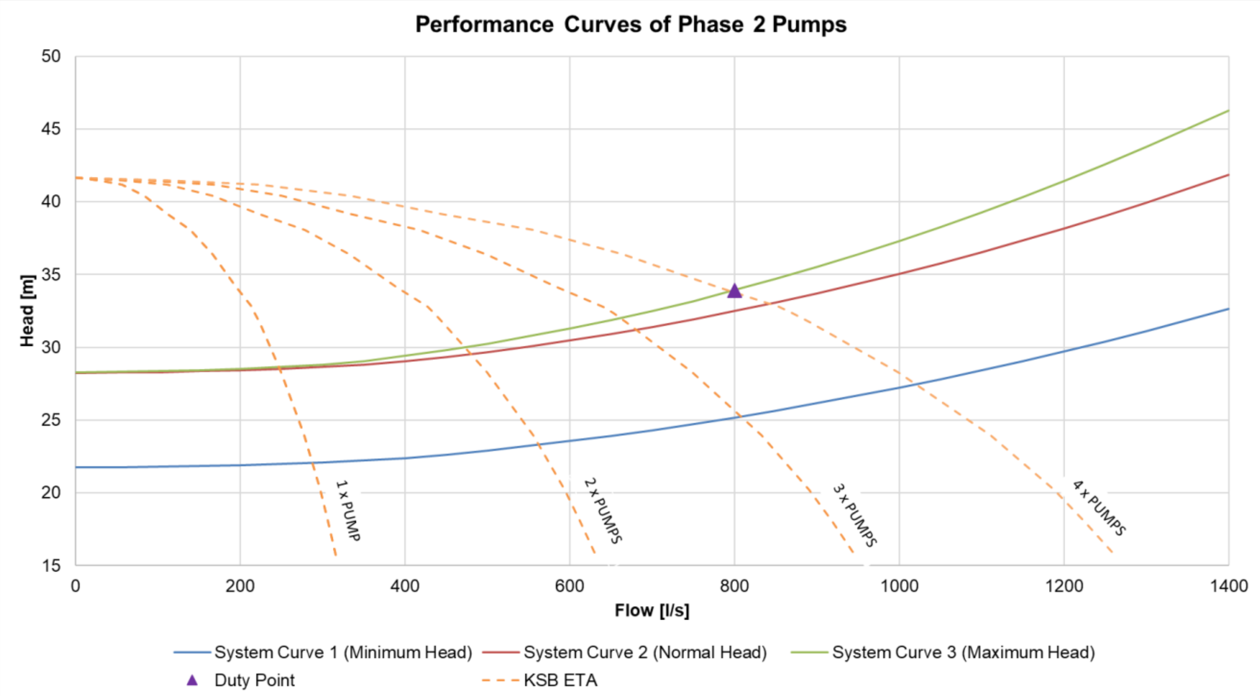

Figure 5-2: Performance Curve for Phase 2 Pumps

Table 5-3 below provides the operating scenario flow rates for all four pump configurations.

Table 5-3: Flow Rates for Operating Scenarios of Phase 2 Pumps

Operating Scenario	Minimum Flow (ℓ/s)	Normal Flow (ℓ/s)	Maximum Flow (ℓ/s)
1 x pump	250	250	297
2 x pumps	500	505	594
3 x pumps	669	749	832
4 x pumps	800	854	1010

5.3 NPSH Calculations

The water flowing through a pump drops in pressure at the eye of the impeller due to the acceleration. If it drops low enough to the vapour pressure, boiling will occur leading to the formation of vapour bubbles. As the vapour bubbles move from the impeller eye it reaches a region with a pressure higher than the vapour pressure. This results in the bubbles collapsing (imploding) as they change back to a liquid phase, causing shockwaves (cavitation) which result in damage to the impeller. This cavitation also results in a decrease in the pressure head produced from the pump.

To prevent damage due to cavitation, it is important to ensure that the $NPSH_A$ (Net Positive Suction Head Available) is more than the $NPSH_{req}$ (Net Positive Suction Head required) as specified by the manufacturers. $NPSH_{req}$ is the pressure head needed to ensure that the eye of the impeller is always above vapour pressure. $NPSH_A$ is the actual positive suction head available for the particular installation.

The NPSH is of particular concern for the phase 2 installation, where the pumps will be installed above the normal water level. As such, the NPSH calculations was used to inform the floor level required for the drywell i.e., the floor level was selected to ensure the $NPSH_A$ will always be more than the $NPSH_{reg}$. The following equation was used in the $NPSH_A$ calculations (from *Pumping Station Design* by *Garr M. Jones, 2006*):

$$NPSH_A = H_{bar} + h_s - H_{vap} - h_{fs} - \Sigma h_m - h_{vol}$$

Where:

H_{bar} = barometric pressure (10.3m water head in this case);

h_s = static pressure;

H_{vap} = vapour pressure (0.13m in this case);

h_{fs} = pressure loss due to friction;

Σh_m = sum of minor pressure losses; and

h_{vol} = partial pressure of dissolved gases (deemed to be negligible in this case).

It is recommended that a **safety margin of at least 0.6 m** is maintained for the $NPSH_A$ compared to the $NPSH_{req}$ for all operating conditions.

Table 5-4 below shows a summary of the results of the NPSH calculations for the expected operating flow range for the phase 2 pumps, as obtained from Figure 5-2, and for a **drywell floor level of RL4.5 m**. For the reference pumps used in this design, the safety margin for the entire operating range will be greater than 0.6 m.

Table 5-4: NPSH Required and Available Summary

	Flow (ℓ/s)	$NPSH_{req}$ (m)	$NPSH_A$ (m)	Safety margin (m)
Minimum Flow	200	4.2	6.4	2.2
Maximum Flow	297	5.0	5.9	0.9

Figure 5-3 below further illustrates that with a RL4.5 m floor level for the drywell, the $NPSH_A$ for the Phase 2 pumps will exceed the $NPSH_{req}$ for the full operating flow range.

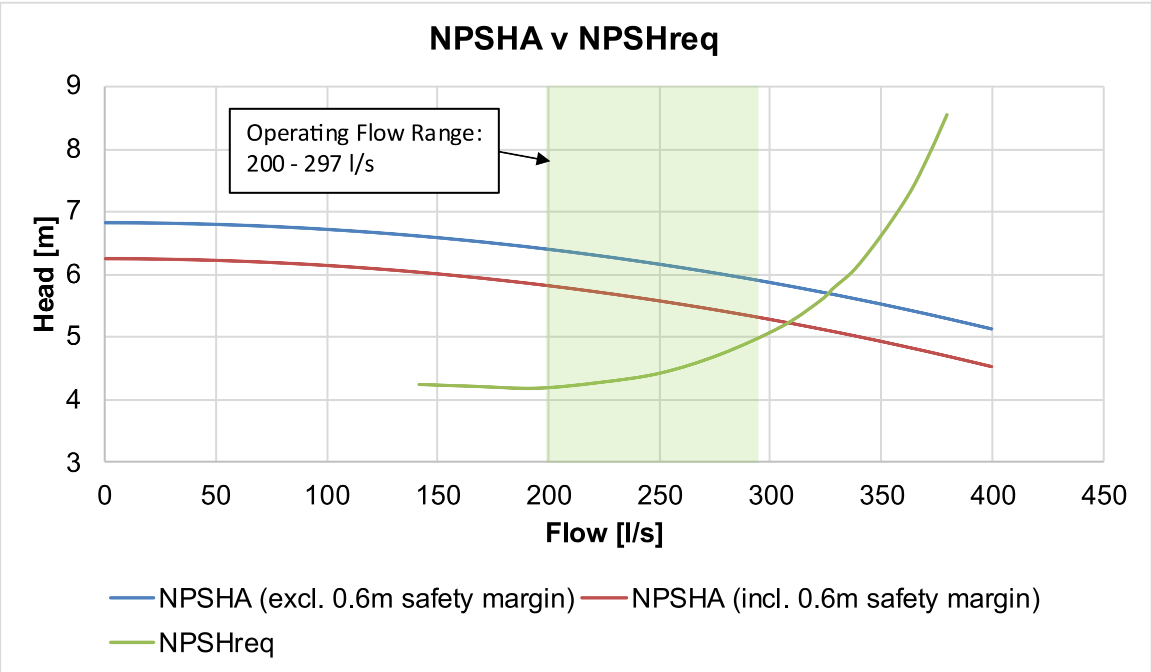

Figure 5-3: NPSHA vs NPSHreq

It should be noted that this calculation is dependent on the equipment used in the installation, and as such, the following should be considered when procuring equipment for the phase 2 installation.

- The calculation assumed a local loss factor of **5** over the foot valve proposed for phase 2. According to literature, based on the type of foot valve selected, this actual local loss factor can range from 1 up to 15.
- The $NPHA_A$ was compared to the reference pump used for the phase 2 design. The final pump procured must have a similar or lower $NPSH_{req}$ curve than the reference pump.
- The pumps should not be operated for a water level below the specificied minimum operating level of **RL 1.95 m.**

5.4 Pipework and Valves

Table 5-5 gives a summary of the preliminary sizing of the pipework for the Moordkuil Pump Station. The sizing is based on minimizing velocities in the suction and discharge pipework to be under 1.6 m/s and under 3.1 m/s, respectively.

Table 5-5: Pump Station Pipework Sizing

Pipe Segment	Minimum Flow (ℓ/s)	Maximum Flow (ℓ/s)	Proposed Size	Minimum Velocity (m/s)	Maximum Velocity (m/s)
Suction Pipework	250	310	DN500	1.3	1.6
Discharge Pipework	250	310	DN350	2.6	3.1
Discharge Manifold	800	1080	DN600	2.8	3.8 [1]
Motive Suction Pipework	30		DN200	1.2	
Motive Discharge Pipework	30		DN150	1.8	
Jet Pump Flexible Hose	30		DN150	1.8	

Note 1 – the discharge manifold was sized to match the rising main. It is noted that under normal conditions, the velocity in the rising main (and thus in the discharge manifold) is at the upper limit of the acceptable velocity range. Under maximum flow condition, this velocity is above the acceptable range. The implication of this will be further assessed in the detailed design phase in conjunction with a water hammer analysis. However, to mitigate the high velocities, it is recommended not to operate the pumps while the river is in flood conditions.

Figure 5-4 below shows the proposed pipework and valve arrangement for the pump station. The following valve types and sizes are proposed for the different pipe segments:

- Suction Isolation Valve: DN500 Butterfly Valve
- Discharge Check Valve: DN350 Slanted Seat Tilting Disc Check Valve
- Discharge Isolation Valve: DN350 Resilient Seal Gate Valve
- Rising Main Scour Valve: DN350 Wedge Gate Valve
- Motive Pump Suction Valve: DN200 Resilient Seal Gate Valve
- Motive Pump Discharge Valve: DN150 Nozzle Check Valve
- Jet Pump Isolation Valves: DN150 Resilient Seal Gate Valve

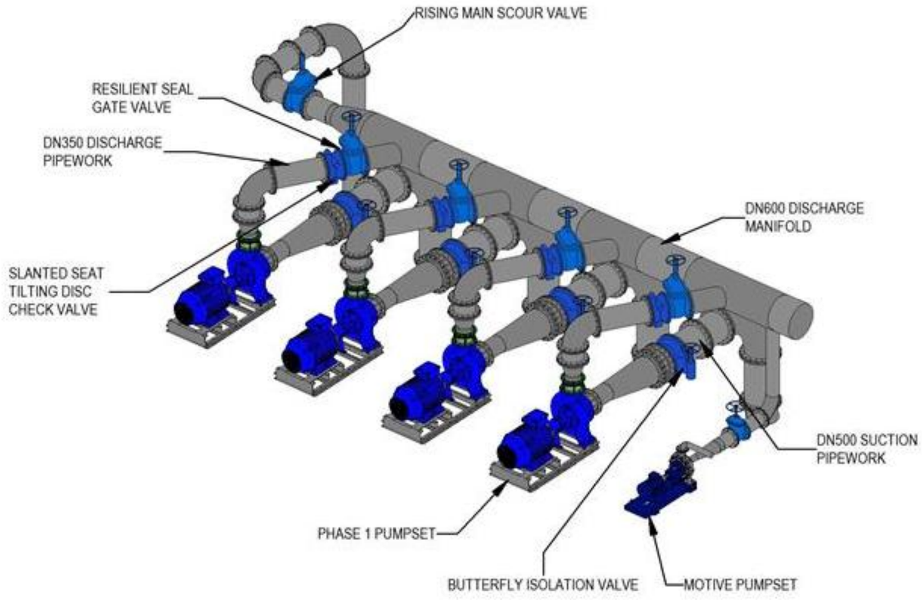

Figure 5-4: Pump Station Pipework and Valve arrangement

While vertically installed nozzle or silent check valves were considered, they are more prone to blockages from floating debris like plastic, branches, and rags in raw water applications. As such, slanted seat tilting disc check valves are preferred for this application. As these valves are not suited for a vertical installation, the check valves will be placed on the horizontal pipework upstream of the discharge isolation valves.

It is further proposed that a scour pipeline back to the river is provided to enable the rising main to be scoured.

5.5 Lifting Equipment

The existing I-beam and attached crawl beam will be modified and extended to span across both the current and the new adjacent dry well (refer to Figure 4-9). A motorised trolley with an electric chain hoist will operate on the extended crawl beam, which will run over the centre line of the pumps and be installed to allow pumps to be lifted over one another with adequate clearance for the hoist and hook. The minimum safe working load of the lifting equipment shall be 2000 kg. The Phase 1 combined pumpset weight (i.e. pump, motor and baseplate) is approximately 1200 kg. The Phase 2 pumpset (i.e. pump, motor and baseplate) will weight approximately 1500kg. The lifting equipment will be rated for 2000kg (2-ton) to meet the recommended safety margin of 30%.

The option exists to repurpose the existing hoist in the pump station, shown in Figure 5-5, which is also rated for 2000 kg. The feasibility of using the existing hoist will be considered in the detailed design phase. Alternatively, and based on client preference and requirements, the existing crane beam and host can be left in place to assist in handling the electrical equipment.

Figure 5-5: Existing pump station hoist

5.6 Motive and Jet Pump System

Permanently installed jet pumps, one located in each of the two hoppers, are proposed to intermittently remove settled sediment. A section of the intake structure is illustrated in Figure 5-6 in which the hoppers, jet pumps and submersible pumps can be seen. The timeous removal of coarser settled sediment protects the submersible pumps, the low-lift pipeline, the end suction pumps and the rising main from transporting abrasive particles that can damage the mechanical equipment and settle in the pipelines. The removal of settled non-cohesive sediment through the jet pumps to the river should occur in short bursts with minimal to no impact on the ecology of the river downstream of the works.

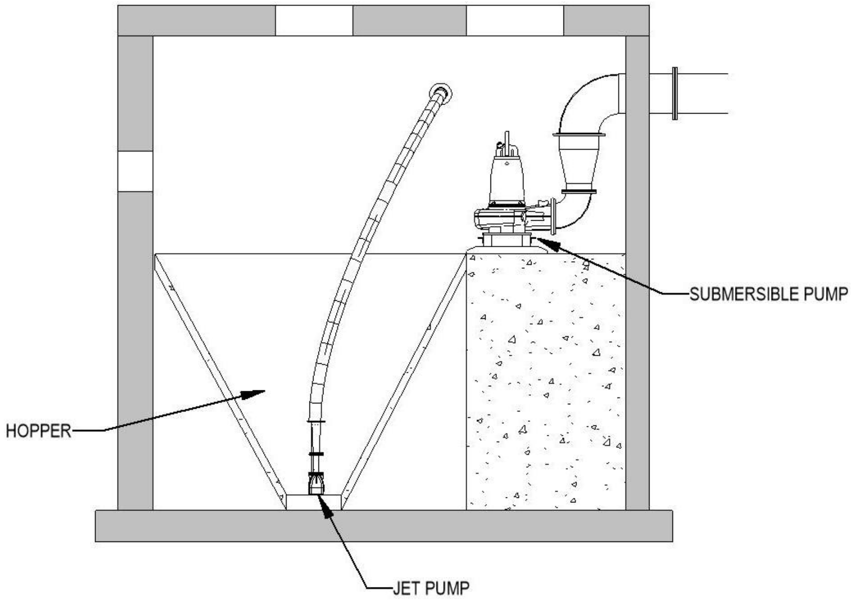

Figure 5-6: Intake structure section

Sediment-free motive water is required for each jet pump at a rate of 30 ℓ/s and a total head of 65 m to ensure optimal sediment removal in the hoppers, as recommended by ASPTech in the Verification Report.

It is recommended that the motive water be supplied by tapping into the DN600 discharge line from the Moordkuil pump station. Since the discharge line already provides at least 28 m of head (minimum delivered head when one pump is running), it is recommended that the dedicated motive pump only needs to supply the remaining 37 m of head to meet the total 65 m required by the jet pumps.

As per Table 5-1, a KSB Etanorm 100-080-315 pump was selected for the reference design for the motive pump. Figure 5-7 shows the pump curve for this pump against the proposed duty point.

A sediment filter will be installed downstream of the motive pump to ensure that the water supplied to the jet pumps remains sediment-free.

For operation and control, it is envisaged that only one jet pump will be used at a time. Automatically actuated isolation valves will be installed to alternate between the two jet pumps as required.

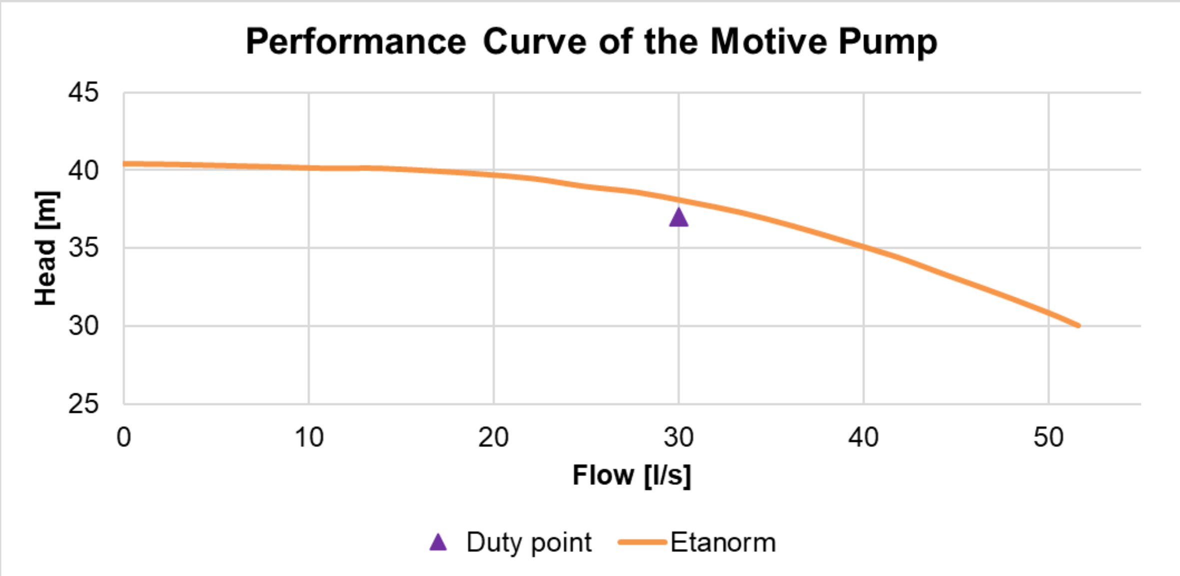

Figure 5-7: Pump Performance Curve for the Motive Pump

6 Electrical, Control & Instrumentation Design

6.1 Bulk Power Supply

The electrical supply authority is Mossel Bay Municipality. The supply to site includes a 22kV overhead line and a 500kVA 22kV/400V outdoor transformer. The transformer is located approximately 60m from the existing building. Initially, this transformer fed the Moordkuil Pump Station and two water pumps belonging to local farmers located on either side of the river. Upon further investigation it has been found that the transformer now only feeds the Moordkuil Pump Station and one farmer, located on the opposite side of the river. However, this project will make allowance for both farmers' pumps to be reconnected to the bulk supply. Further details on the farmers' pumps can be found in Section 6.3.

As stated in Section 2.3.2, the option chosen for the project will have two phases; the first comprising of four 30 kW submersible pump motors and four 75kW booster pump motors while phase two will have four standalone 110 kW end suction pumps. The existing and new equipment for this project fed from the transformer are listed below in Table 6-1. The rated power listed below is preliminary and is subject to change.

Table 6-1: List of proposed electrical equipment

Description	Rated Power (kW)	
Project Phase	Phase 1	Phase 2
2 x Farmers Water Pump @ 45kW each	90	90
4 x Submersible Pumps @ 30kW each	120	-
4 x Booster Pump motors @ 75kW each	300	-
4 x Future End-Suction Pumps @ 110kW each	-	440
1 x Motive Pump @ 22 kW	22	22
Miscellaneous (Small power & lighting, gantry crane, instrument power etc.)	10	10
Total Installed power	542	562

When accounting for efficiency and power factor losses, the estimated load to be supplied by the transformer is about 652 kVA for Phase 1 and 592.6 kVA for Phase 2. In addition, the client has requested that we use direct online (DOL) starters for all pumps, which has increased the maximum apparent power demand. The existing 500 kVA transformer will not have sufficient capacity for the required upgrades and will need to be replaced with an 800 kVA transformer.

The cables running from the transformer to the incomer will also need to be upgraded to accommodate the increase in power demand. It is estimated that 6x95mm² PVC Cu cables per phase are required. These cables will be routed in cable sleeves to the existing building due to the construction of a new access road.

6.2 LV Distribution

As mentioned in Section 4.3, the existing Moordkuil Pump Station building will be retrofitted to accommodate the additional LV MCC equipment required for the new pump systems. A false floor constructed from steel supports with solid GRP panels will be installed at a level above the 1:100-year floodline. This was chosen as the existing electrical room is too small to accommodate the new MCC and to allow cables to run underneath for ease of installation and maintenance. The area will be carefully controlled as it will now be designated as an electrical room; appropriate access control and safety measures will be in place to ensure compliance with regulations. The LV MCC will be placed on the GRP panels and the flooring beneath the MCC shall be reinforced to accommodate the weight of the MCC panels.

The approximate dimensions of the MCC are 4800mm wide x 600mm deep x 2050mm high from the level of the floor, however, this is subject to change pending final vendor information. The MCC will include an allowance for a programmable logic controller (PLC) panel to be placed at the end of the MCC. The 30kW drives, which will supply the submersible pumps, will be housed in the main MCC. Once Phase 2 commences, the submersible pumps will be removed and the 30kW drives will become equipped spares. In addition, 110kW DOL starters will be installed from Phase 1. The 75kW booster pumps will initially be connected to these starters which will eventually be replaced by the new 110kW end-suction pumps in Phase 2.

It has been agreed with DWS that the drives will be direct online (DOL) starters and not variable speed drives (VSD). This was decided for ease of maintenance and longevity of the pump station. A motor starting study should be carried out to ensure all equipment are suitable sized and that appropriate standards are met, such as NRS-048-4. This study shall be performed as part of the detail design phase of the project.

DWS has previously indicated that they do not prefer using any other drive besides DOL drives, but the health of the equipment and the stability of the bulk supply network must be considered. If the motor starting study deems the in-rush current and voltage drop not acceptable, the project will consider replacing the DOL drives with soft-starter drives.

The cables from the MCC to the pumps in the drywell will be routed underneath the GRP panels and through the existing building to the drywell. In the drywell, the cable will run underneath the elevated walkway and tee off to each pump. From the same MCC, the cables going towards the intake structure will be routed to the intake structure. Reasonable measures will be put in place to ensure exposure to water is minimised, especially given that the intake structure is located below the 1:100-year floodline. The final layout and design will be done during the detail design phase of the project.

6.3 Existing farmers’ pump system

As mentioned in Section 6.1, only one of the farmers’ pumps (situated on the other side of the river) is connected to the existing 500kVA transformer. The farmer whose pumps are placed on the same side of the river as the pump station is powered from an independent solar panel and inverter power supply.

However, the scope of this project will be to install an appropriately sized metered connection for each farmer, placed close to the new 800kVA transformer. A 90A connection has been provisioned for each farmer for transformer sizing, with the final connection size to be determined during the detail design phase of the project.

Alterations to the farmer’s pump system and electrical equipment (including the solar panels, inverters and relevant cabling) shall be kept to a minimum. If alterations are made to any electrical equipment or cabling, the project becomes responsible for compliance.

6.4 Control System

The Moordkuil Pump Station will have a straightforward control and operating scheme and shall include, but is not limited to, the following:

- Pump system protection and condition monitoring;
- Pump system control;
- Automated valve control;
- Measurement, indication and control of pressure, flow, level etc.;
- Electrical power quality measurement and power failure sequences; and
- Emergency stop sequences.

As a minimum, the control system will comprise of the following equipment:

- Programmable logic controller (PLC);
- Uninterruptible power system (UPS);
- Human machine interface (HMI);
- Input-output (IO) interface cards;
- Control network cabling (ethernet TCP/IP, Modbus etc.); and
- Instrumentation.

6.4.1 Business intelligence and reporting

To promote digitisation and remote monitoring, it is proposed that the Moordkuil Pump Station include equipment to allow remote, real-time monitoring to the Mossel Bay Municipality and DWS Wolwedans Water Control Officer/DWS Technician. Currently no remote monitoring system is in place for the existing system and routine visits are required to ensure the pump station is running as planned. DWS has advised only monitoring is required, no remote control will be implemented for this project. Remote monitoring will ensure that the operators receive real-time information of the status of the plant and to allow the operators to act accordingly for any breakdowns or alarms that might arise.

Due to the complexity of the hardware and software required to implement remote monitoring, the appointed contractor must ensure that the development team, either in-house or sub-contracted, have specialist experience and certification in SCADA and historian development as well as experience and certification in data warehouse and business intelligence development.

The software and hardware requirements for remote monitoring include, but is not limited to:

- SCADA servers and licensing;
- VPN routers;
- At least one PC based operator workstation for remote monitoring located at the Mossel Bay Municipality Technical Services office;
- At least one PC based operator workstation at DWS premises.
- Managed switches; and
- GPS clock.

This equipment will be housed in either the PLC panel or a dedicated area within the Mossel Bay Municipality Technical Services offices.

7 Conclusions and Recommendations

The objective of this report is to finalize the concept and viability design undertaken for the structural, mechanical and electrical components for the Moordkuil Pump Station upgrade.

A Verification Study conducted on the riverbed topography at the onset of this investigation identified the presence of a significant rock outcrop upstream of the proposed intake structure. As a result, it was determined that the outcrop would need to be removed, and the intake structure enlarged beyond the dimensions proposed in the 2014 – 2016 feasibility study to ensure effective operation.

An alternative solution involves relocating the intake structure directly onto the rock outcrop, which presents several technical and economic advantages:

- **Improved Foundation Conditions:** Relocating the intake would result in more favourable geotechnical conditions, potentially yielding a cost saving of approximately R1.7 million.
- **Operational Continuity:** By situating the intake upstream at the rock outcrop, it may be possible to maintain operation of the existing pump station throughout the construction period.
- **Minimal Disruption to Adjacent Infrastructure:** The nearby farmers' pump station, located just downstream of the existing station, would remain unaffected.

Should the existing pump station remain operational during construction, an estimated cost saving of approximately R24.5 million could be realized over the 18-month construction period by avoiding the need to purchase water from the Wolwedans Dam and the associated saving in chemicals at the water treatment works.

A new dry well pump station could be constructed at an estimated cost of approximately R2.4 million. This facility would enable continuous operation of the existing pump station with minimal interruption of the water abstraction during the construction phase. An additional benefit of constructing a permanent dry well is that it would allow the end-suction pumps to be installed at a lower elevation. This could possibly eliminate the requirement for immersible pumps within the intake structure in the future and enable the use of foot valves in combination with a priming system. A new dry well will also provide additional space for the installation of the proposed electrical equipment as the existing MCC room is very small.

Based on the abovementioned, **Option 3** (wetwell on rock outcrop with drywell next to pump station, Concept Layout 2) is recommended for implementation for the Moordkuil Pump Station upgrade, as it offers the lowest capital and operational costs, best operational reliability, and acceptable environmental impact. To maximise the value of existing assets, a **phased approach** should be adopted:

- ► **Phase 1**: Utilisation of existing immersible and end-suction pumps that was bought based on the previous (2014 – 2016) investigation.
- ► **Phase 2**: Replacement of the immersible pumps with foot valves and installing larger single stage end suction pumps in die drywell. *The detailed considerations for this system, such as the operation of the foot valves and the suction pipework priming, will be included in the detailed design report.*

The hydraulic design ensures the intake structure are self-scouring and resilient to sediment deposition. The civil design provides for robust, flood-resistant structures, with careful integration of new and existing facilities to maintain operational continuity during construction.

The mechanical design supports both current and future pump configurations, with appropriate safety margins for motor sizing and lifting equipment.

The electrical design requires upgrading of the transformer and cabling to accommodate increased power demand, with a focus on direct online (DOL) drives for reliability and ease of maintenance.

The control system will be kept as simple as possible, with automated protection, measurement, and reporting. Remote monitoring will be implemented for real-time status updates, but remote control will not be enabled, as per client requirements.

Based on this concept and viability design report, it is recommended to proceed with the detailed design of the selected option, i.e. Option 3 at an estimated cost of R42.6 million.

APPENDIX A

Moordkuil River Abstraction Works Detailed Design

Report Rev 02 – July 2025

MEMO IN PROGRESS

MOORDKUILS RIVER ABSTRACTION WORKS DETAILED DESIGN

Review of the proposed hydraulic design of the intake works with 2D hydrodynamic of the scour/deposition and 3D CFD modelling of hydraulic forces

REPORT rev02

July 2025

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Table of Contents

1. Introduction

The Department of Water Affairs and Sanitation (DWS) recently appointed the Mossel Bay Municipality as their Implementation Agent to implement the proposed upgrading of the Moordkuil Pump Station which is part of the Mossel Bay Regional Water Supply Scheme (RWSS). The Moordkuil Pump Station abstracts water from the Moordkuil River and discharges into the Klipheuwel Dam which is an off-channel storage dam – see Figure 1-1. The required pumping capacity of the Moordkuil pumpstation is 800 litre/s to be provided by four pumps. The purpose of the proposed upgrade (mainly the river works) include:

- Prevention of sedimentation interrupting the operation of the pumpstation under all river flow conditions.
- Provision of a permanently installed sediment removal system to remove suspended sediment that settles in the pump forebay.

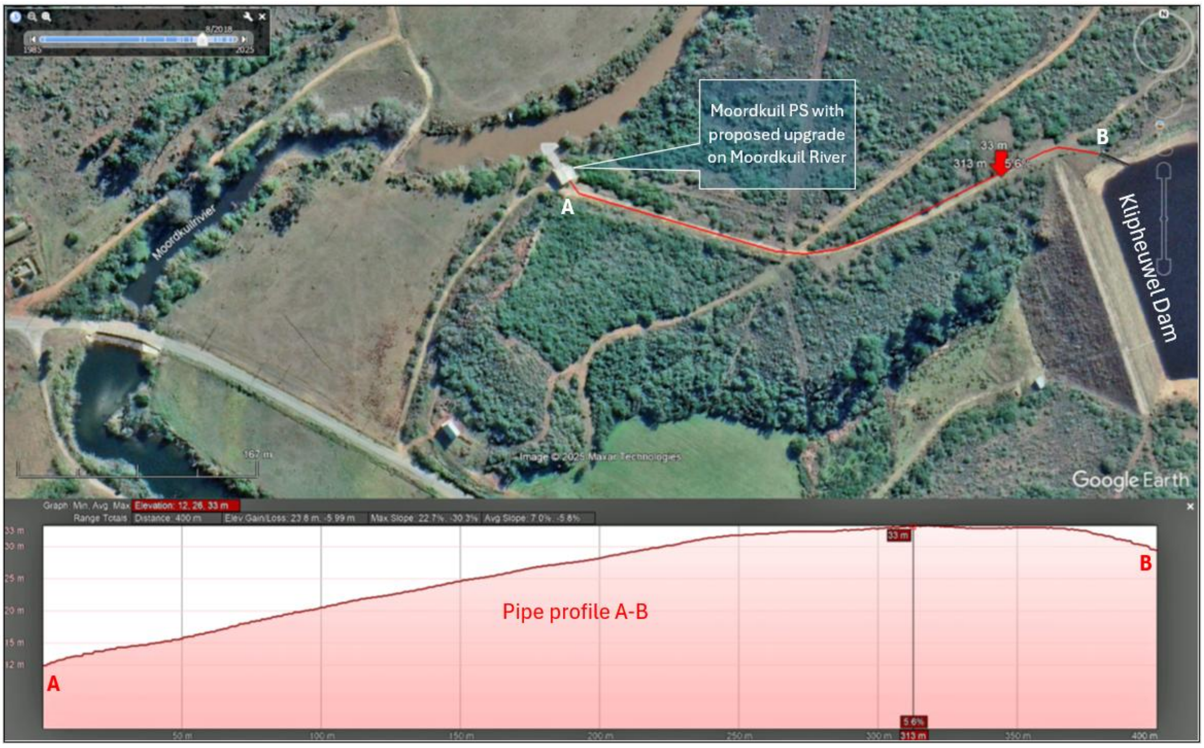

Figure 1-1: Moordkuil Pumpstation with proposed upgrade on the Moordkuil River. The location and profile of the rising main (A-B) are also shown

LYNERS (Pty) Ltd appointed ASP Technology (Pty) Ltd during May 2025 to perform the following tasks:

a) Compare the 2025 and 2014 underwater surveys and make recommendations on the intake location and upstream obstructions in the river (bedrock and man-made wall).
b) Review the proposed hydraulic design of the intake works: pump selection, minimum submergence required, pump bay space required to limit sediment deposition, dividing wall between pumps, jetpump selection with motive pump/or tap-off from high lift pump-pipe system; trashrack openings to protect the jet pumps; hopper design and size to deposit fine sand; floating debris control; cleaning of trashracks, sheetpile soil anchors, etc.
c) 3D CFD modelling with FLOW3D-HYDRO of the hydraulic forces on the sheetpile and piling intake works: 2 scenarios: i) intake with sediment not scoured from between the piling; ii) with sediment scoured from between the piling:

- Setup model: convert bathymetric survey and structure to appropriate format, boundary conditions, allocate reference points for forces
- Simulations: 2D model of large domain followed by hybrid 3D model, repeat for two scenarios
- Post-process data and produce figures/tables of water levels along the sheet piling and piling, pressure plot of sheetpile wall, fluid force values at reference points over time for two scenarios

This report presents the results of the above required tasks. Information available to ASP is first presented followed by relevant results and recommendations.

2. Comparison of the 2025 and 2014 underwater surveys and recommendations on the intake location and upstream obstructions in the river

2.1. Comparison of the 2025 and 2014 underwater surveys

The 2025 and 2014 local surveys at the proposed abstraction works were compared to identify any scour and deposition that occurred in the river within the elapsed period of 11 years, which might affect the original proposed intake location of the upgraded abstraction works. The 2014 survey was subtracted from the 2025 survey, see Figure 2.1-1, the positive values on the legend indicate deposition and the negative values on the legend indicated scour. Scour was observed on the outside bend of the river and deposition on the inside bend of the river as expected, however the upstream rock obstruction at the left bank caused deposition further downstream where the proposed intake works is located. This deposition will influence the effectiveness of the abstraction works negatively, due to the area not being able to scour properly during floods and keep the intake area clean and clear from deposited material. The rock should be removed.

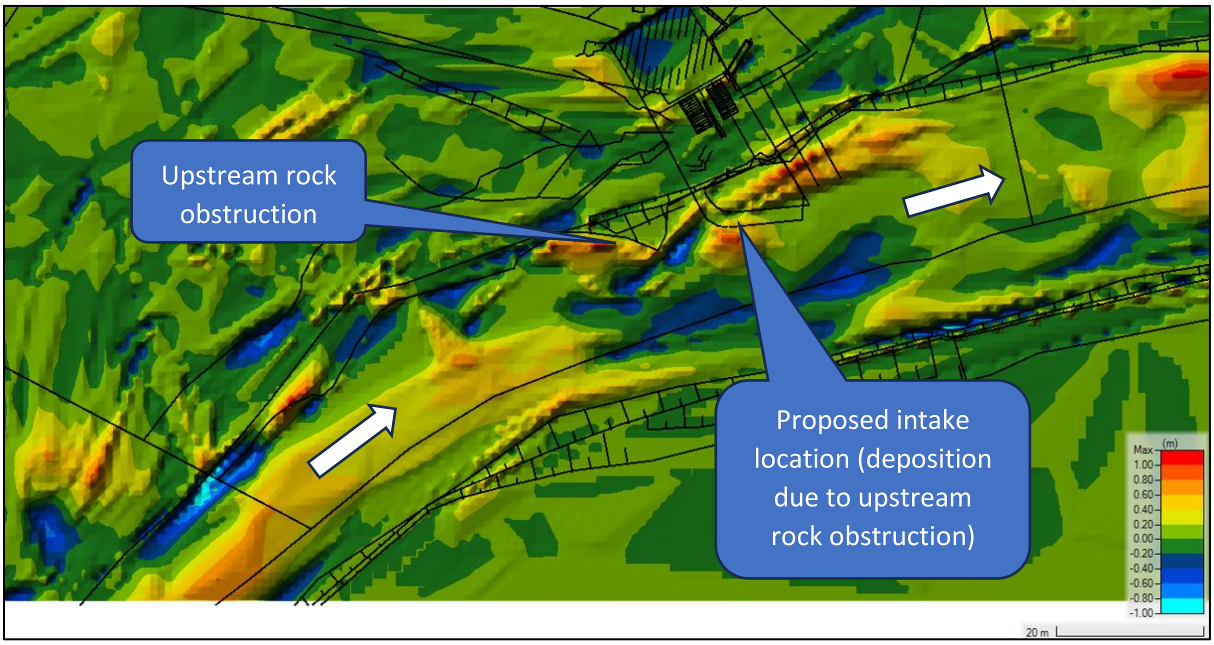

Figure 2.1-1: Difference between 2025 and 2014 surveys of the proposed Moordkuil Pumpstation with proposed upgrade on the Moordkuil River. (Red legend = deposition, blue legend -= scour)

2.2. Recommendations on the intake location and upstream obstruction

The intake location of the proposed abstraction works is located at the lowest point in the river, with the downstream low water bridge (that was closed off to prevent saltwater from pushing back up in the river due to ocean tides) that creates the control in the river at an approximate elevation of 1.950 masl. The intake is located on the outside bend of the river with an angle of approximately 19 degrees to the flow direction (left bank used as reference line). Figure 2.2-1 shows an isometric view looking at the abstraction works from upstream. The invert level of the intake is located at 0.600 masl and the soffit of the intake is located at 1.100 masl, ensuring that the intake is submerged below the minimum operating level.

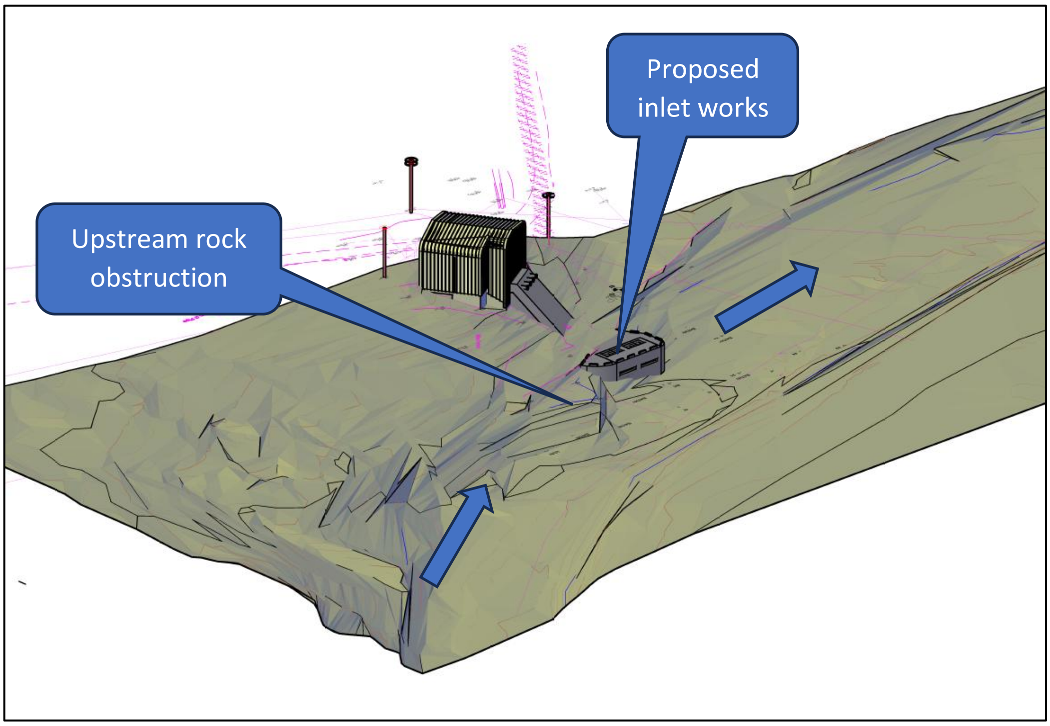

Figure 2.2-1: Isometric view looking at the proposed abstraction works from upstream (2025 survey)

Figure 2.2-2 shows a descriptive cross section summary of the current proposed intake work elevations and key dimensions. The summary of current proposed intake details is shown below:

- Proposed abstraction required = 0.8 m^3/s
- Minimum operating water level (downstream bridge) = 1.950 masl
- Invert level = 0.600 masl
- Soffit level = 1.100 masl
- Level of proposed abstraction deck = 2.500 masl
- Natural ground level @ middle of intake = -1.0 masl
- Intake opening invert above ground level = 1.6 m
- Top of hopper level = 0.400 masl
- Invert level of hopper inside = -1.250 masl
- Intake opening height = 0.500 m
- Width of each opening = 3.0 m
- No of intakes = 2

- Total effective opening inlet area = 3 m^3
- Flow velocity = 0.267 m/s (target = 0.300 m/s with trashracks/screens unblocked)

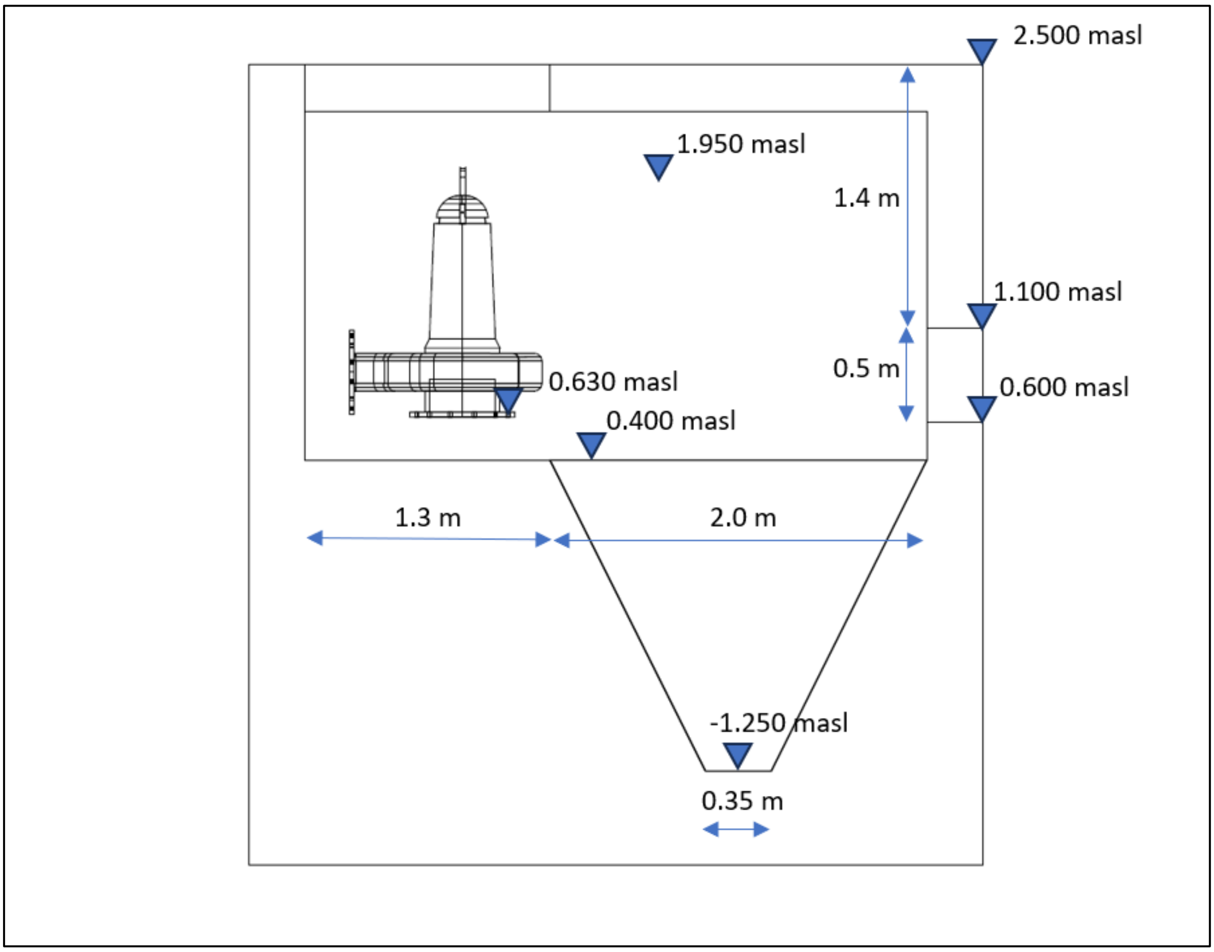

Figure 2.2-2: Example of section view indicating current proposed intake work elevations and key dimensions

The position of the intake is located at a good position with a few minor adjustments that need to be made. The following are recommended at the intake works:

- Proposed abstraction required = 0.8 m^3/s
- Minimum operating water level (downstream bridge) = 1.950 masl
- Enlarge each hopper to 4 x 4 m in plan (approximately 4 m deep)
- Make width of each intake opening 4.0 m from previous 3.0 m
- No of intake openings = 2
- Total width intake openings = 8.0 m
- Intake opening height = 0.527 m
- Assumed open intake area of each opening = 2.0 m^2
- Total open intake area = 4.0 m^2
- Install trashrack with 40 x 40 mm flat grid bars spaced at 50 mm centre to centre
- Closed area factor to be included in velocity calculations = 0.633
- Velocity calculations for opening height of 0.527 m and 4.0 m width each:
    - Q = A x V x Closed Factor

- 0.8 = (8.0 x 0.527) x V = 0.633
- V = 0.300 m/s (blocked scenario = 0.600 m/s)

Figure 2.2-3 shows a descriptive cross section summary of the elevations required for the proposed changes to the inlet works. The summary of proposed abstraction inlet detail requirements is shown below:

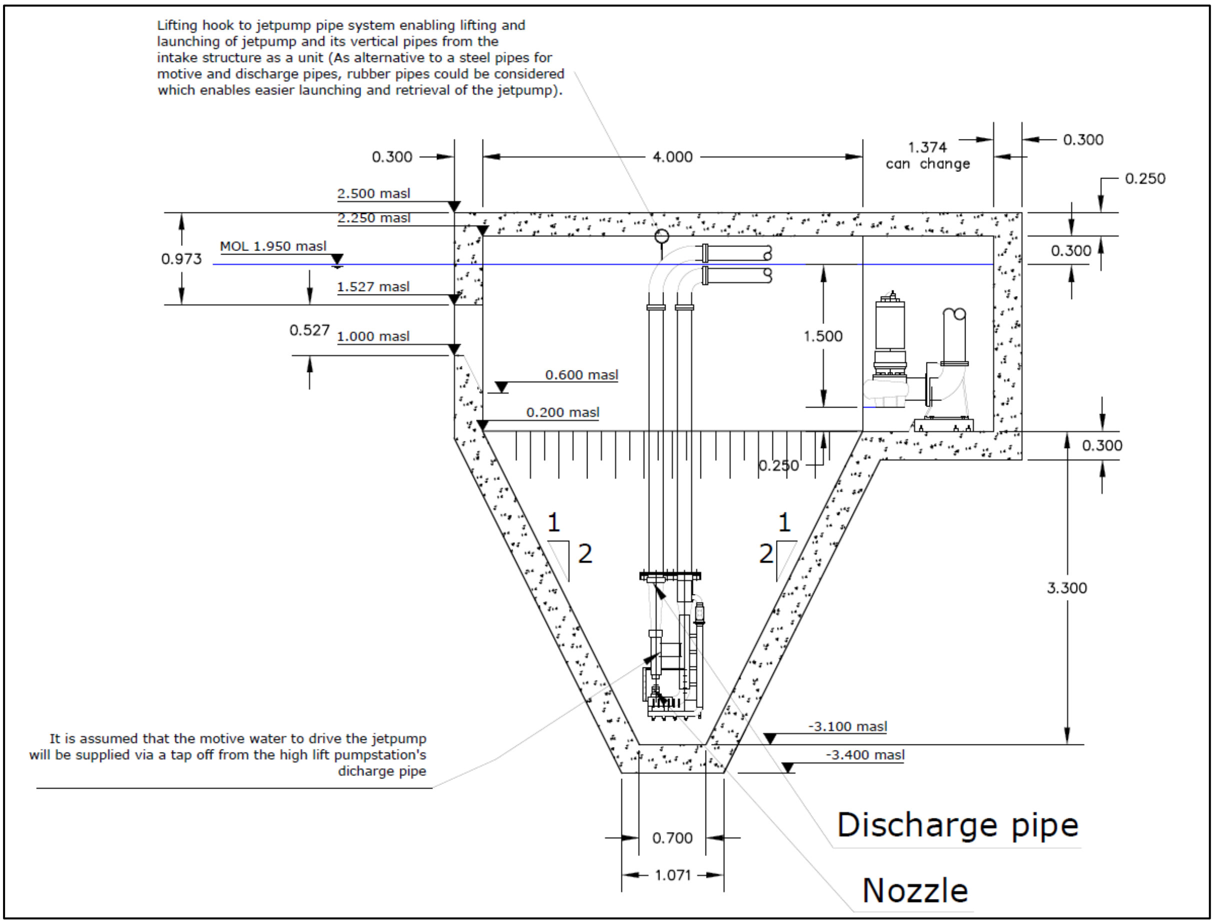

Figure 2.2-3: Example of section view indicating possible changes required on widths and elevations

- Minimum operating water level = 1.950 masl (deck of low water bridge downstream)
- Width of each intake opening = 4.0 m
- Height of each intake opening = 0.527 m
- Screens to be added 40 x 40 mm flat grid bars spaced at 50 mm centre to centre
- Submerge soffit of inlets at least 0.3 m under the MOL (1.650 masl) (0.550 m higher than previous design)
- New intake soffit level = 1.527 masl
- New intake invert level = 1.00 masl
- Top of hopper level = 0.200 masl (0.200 m lower than previous design)
- Invert level of hopper on inside = -3.100 masl (1.850 m lower than previous design)
- Natural ground level @ middle of intake = -1.0 masl
- Intake opening invert above ground level = 2.000 m

The inlet position of the proposed abstraction works should stay the same. The change to the hopper size should be made to the downstream end of the existing proposed design and also moved further inland, which will affect the pipe layout. The upstream rock obstruction should also be removed as indicated in Figure 2.2-4 (up to a level of at least -0.5 masl). The area identified to be excavated will yield a total volume of approximately 300 m^3 material that needs to be removed.

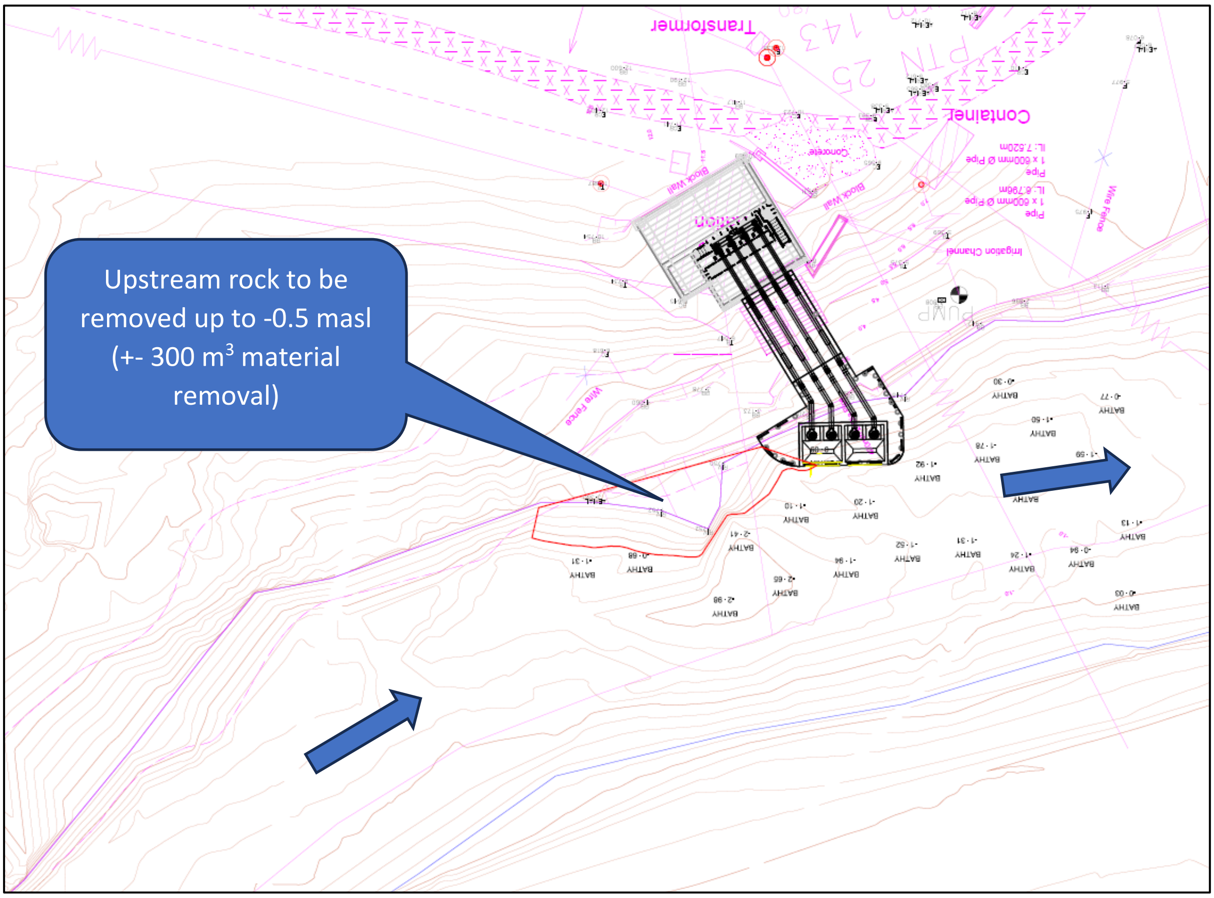

Figure 2.2-2: Plan view of proposed abstraction works on 2025 survey that indicate the upstream rock obstruction that needs to be removed

The minimum operational level of the pumps for the proposed recommended changed intake works is located at 1.950 masl. When the water level in the pool from which the abstraction is done drops to a level below 1.950 masl, the pumps should stop working.

3. Review of the proposed hydraulic design of the intake works

The LYNERS proposed intake works (low level option) in the river (adjacent to the existing pumphouse on the left riverbank) is illustrated in Figures 3-1(a and b) below. The footprint in which the river intake works should be located is shown in Figure 3-2.

Figure 3-1a: Illustration of the low-level intake works (LYNERS, 2016)

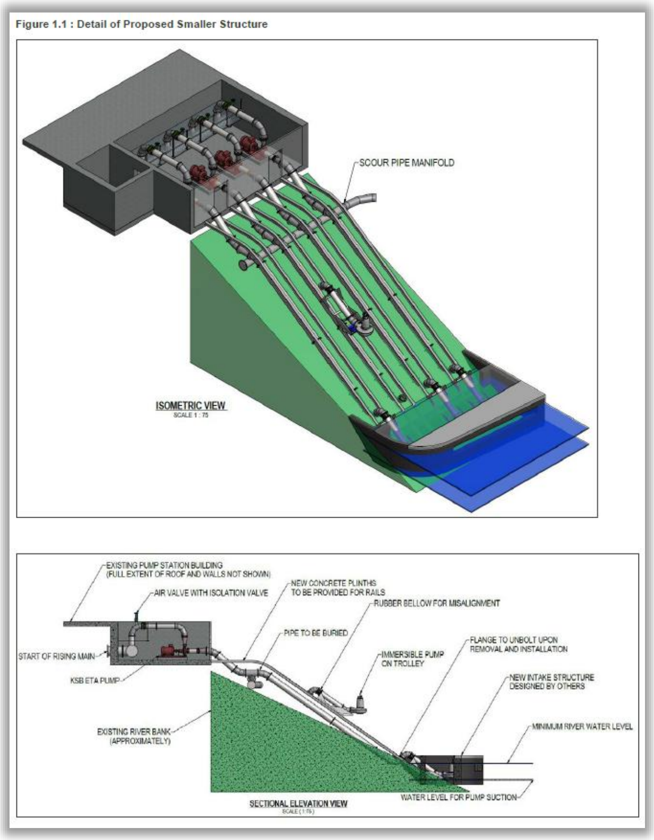

Figure 3-1b: Illustration of the low-level intake works (LYNERS, 2016)

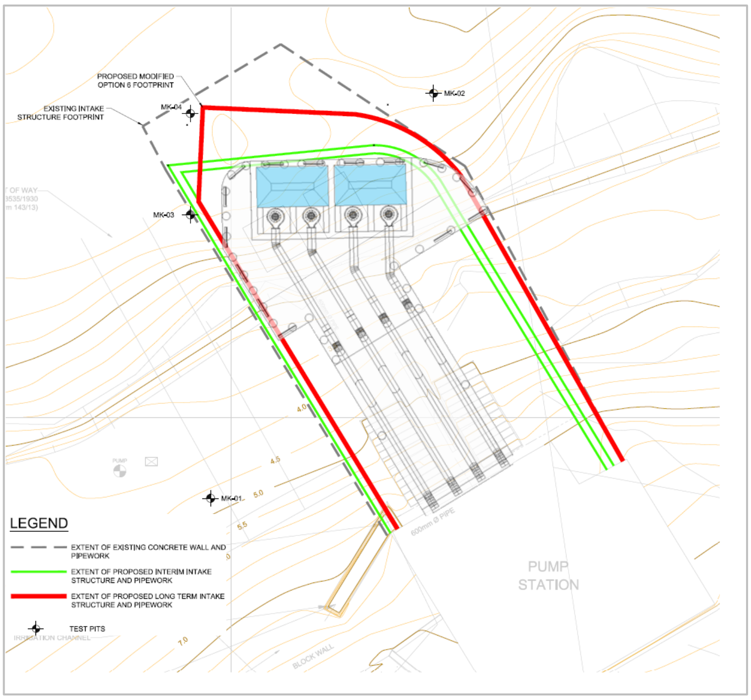

Figure 3-2: Illustration of the low-level intake works (LYNERS, 2016) to be accommodated within the red and green boundaries

Based on the principle of the LYNERS concept design for the river intake works as shown in Figure 3-2, the following modifications are recommended as shown in Figure 3-3:

- The size of the hoppers should be increased to a longer flow path to the pumps enabling the courser fraction (sizes larger than 0.4 mm) to settle in the hopper.
- To provide a larger hopper volume to ensure a less frequent sediment removal from the hopper.

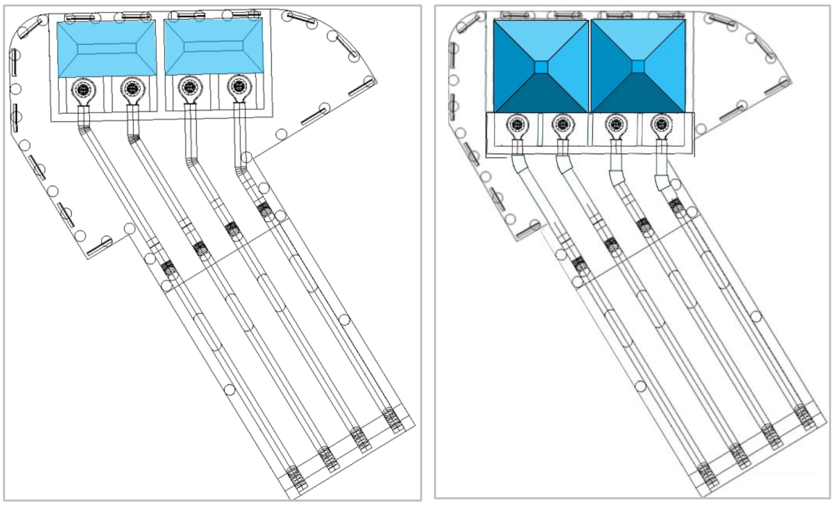

Figure 3-3: Pump and hopper layout by LYNERS (left) and that of ASP (right) and existing intake footprint and possible modification of proposed intake footprint: LYNERS dwg no 13012-c-000-a rev B (right)

It is proposed that the hoppers should be enlarged to 4m x 4m each with hopper slopes of 1H:2V. No dividing wall between hoppers should be present for flexibility of pump operation. The 4m x 4m pump bay should be large enough to also accommodate a motive pump (refer Section 7) to drive the proposed jet-pumps in the hoppers for sediment removal.

4. Selection of raw water pumps

Submersible sewer-type pumps are recommended for the raw water pumps because of their robustness and capability to pump debris and sediment with the appropriated selected impeller material (e.g., grey cast iron). LYNERS indicated that pumps for the proposed works have already been purchased. It is assumed that the submersible pumps were intended to be located in the proposed improved river intake works and that these pumps would deliver to pumps (in the existing pumphouse) which would act as booster pumps to deliver water to the Klipheuwel Dam via an existing 600 diameter pipe about 420 m long (refer Figure 1-1 for the location and profile of the rising main and Table 4-1 and Figure 4-1 for the system curves of the rising main). These purchased pumps are:

<u>Submersible pumps to be accommodated in the proposed improved river intake works</u>:

- Four FLYGT NP 3202 LT by XYLEM (refer **Appendix A** for technical detail)

<u>Booster pumps to be accommodated in the existing pumphouse</u>:

- Four LOWARA NSC 250-315 pumps by XYLEM (refer **Appendix B** for technical detail.

The above submersible type pumps are suitable for pumping raw river water containing some sediment and debris. However, the booster type pumps are less suitable to pumping raw water containing some sediment and debris. In addition, for a pumping system with a relative low pumping head (such as for this project with a static head of about 30m) it is considered that the pumping capacity of the raw water pumps in the river works should be selected such that they can pump directly (without the assistance of a booster pump) to the Klipheuwel Dam. This will significantly simplify the operation and consequently increase the reliability of the pumping system.

Based on the latter reasoning, four KSB KRT 200-402 pumps are recommended with two alternative cases (refer Figure 4-2 for the performance curves of the recommended KSB pumps):

- Case 1: Four duty pumps with no standby with each delivering 200 l/s for a total discharge of 4 x 200 l/s = 800 l/s. Figure 4-3 shows the duty flow rates per pump and head range for this case.
- Case 2: Three duty and one standby (for improved reliability) with each delivering 266 l/s for a total of 3 x 266 l/s = 800 l/s. Figure 4-3 also shows the duty flow rates per pump and head range for this case.

Cooling jackets are recommended for the recommended submersible pumps for additional protection. Variable frequency motors are recommended to operate at their best efficiency both during normal river flow and flood flow conditions as shown in Figure 4-3.

Table 4-1: Calculations of the system curves of the rising main as presented in Figure 4-1

Waterlevel at intake end of pipe 1 =	2	(masl)
Waterlevel at discharge end of last pipe =	33	(masl)
Static head =	31	(m)

Q	Dens, ρ	Dyn Visc., μ	Kin viscos, υ	Dia, D	Length, L	ε	ε/D	Veloc., V	Re= V*D/υ	Darcy Weisb. f	hL in pipe	Turb. Shear stress	sum of k's ; minor losses	hL minor	Loss Sub-total 1	Total head
m³/s	kg/m³	kg/(m.s)	m²/s	m	m	m	-	m/s	-	-	m	N/m²	Number	m	m	m
0.100	1000	0.00112	0.00000112	0.6000	420	0.0000450	0.000075	0.35	1.89E+05	0.016	0.07	0.26	3	0	0	31.00
0.200	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	0.71	3.79E+05	0.015	0.26	0.92	3	0.08	0.34	31.34
0.300	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	1.06	5.68E+05	0.014	0.56	1.96	3	0.17	0.73	31.73
0.400	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	1.41	7.58E+05	0.013	0.96	3.37	3	0.31	1.27	32.27
0.500	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	1.77	9.47E+05	0.013	1.47	5.15	3	0.48	1.95	32.95
0.600	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	2.12	1.14E+06	0.013	2.08	7.30	3	0.69	2.77	33.77
0.700	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	2.48	1.33E+06	0.013	2.80	9.80	3	0.94	3.74	34.74
0.800	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	2.83	1.52E+06	0.013	3.62	12.66	3	1.22	4.84	35.84
0.900	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	3.18	1.71E+06	0.013	4.54	15.89	3	1.55	6.09	37.09
1.000	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	3.54	1.89E+06	0.012	5.56	19.47	3	1.91	7.47	38.47

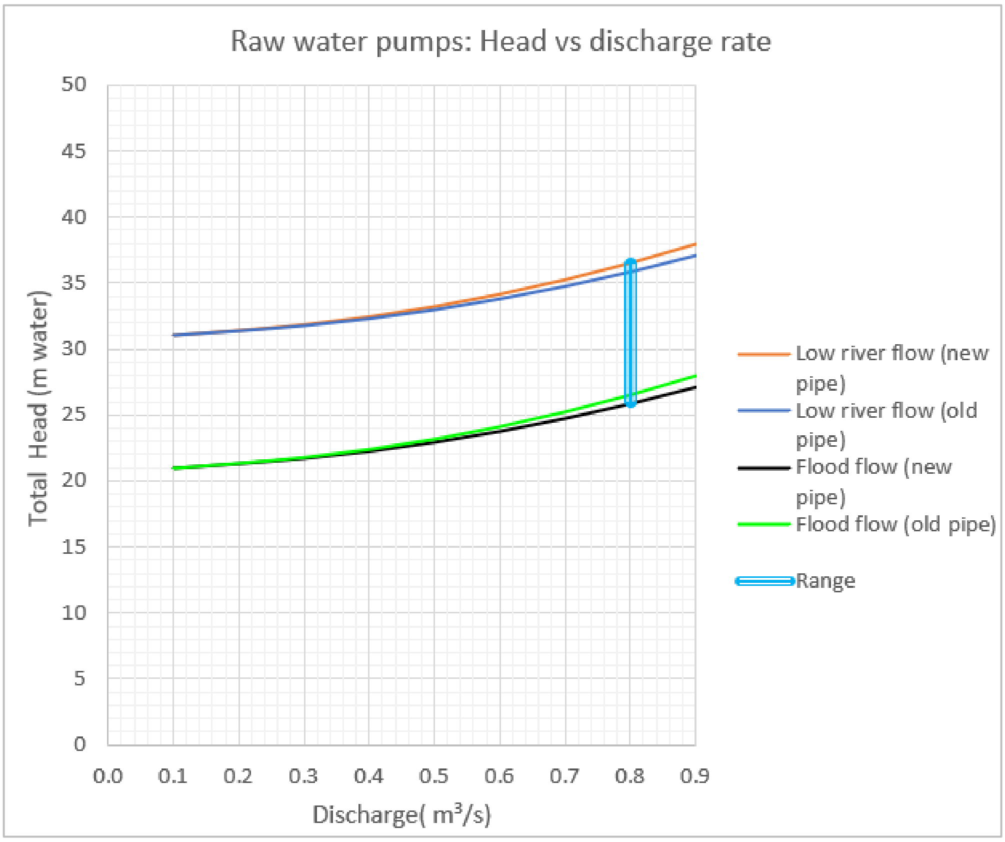

Figure 4-1: System curves of the rising main during low river flow and floods with the required operating range at 800 l/s

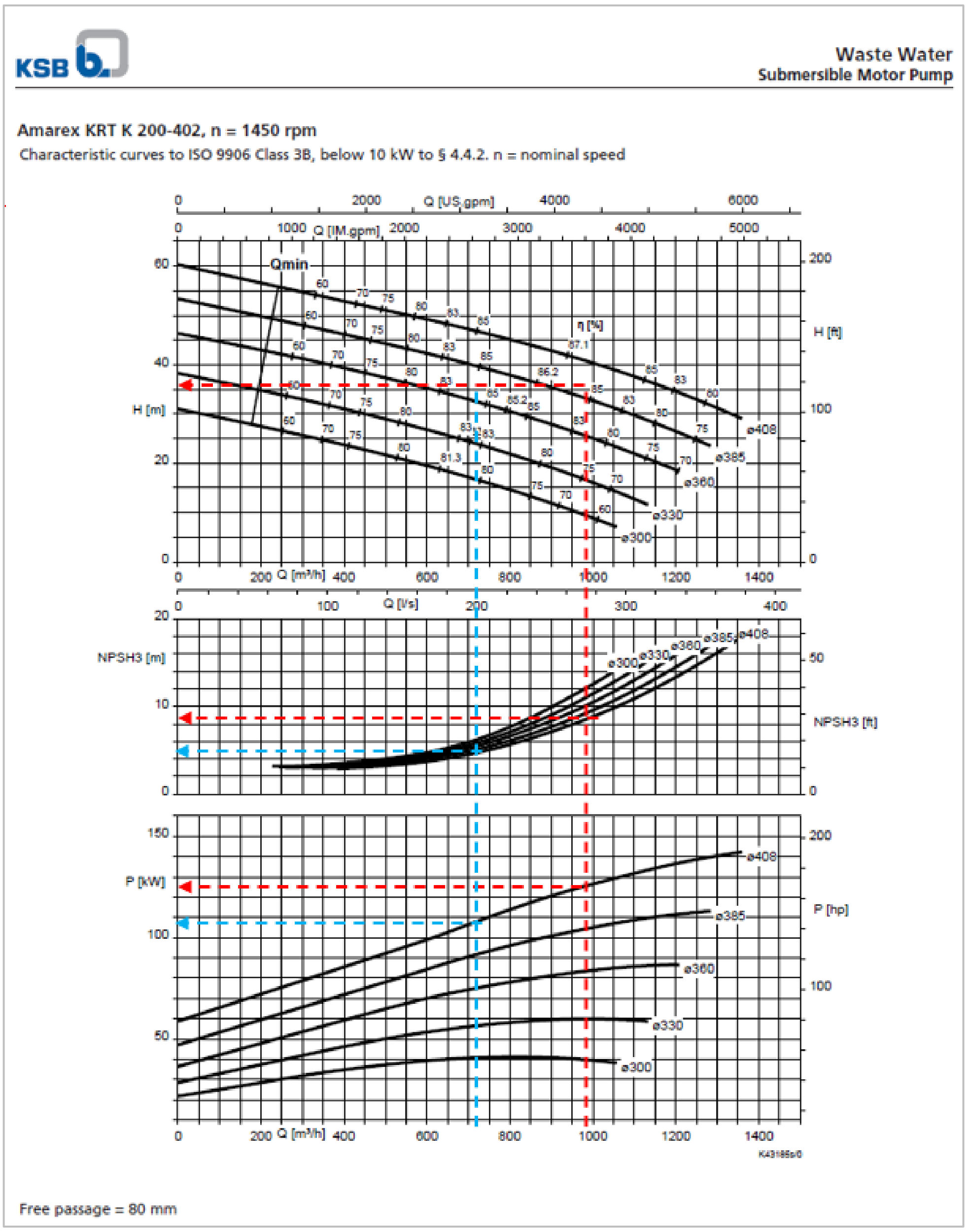

Figure 4-2: Performance curves of the recommended KSB raw water pumps indicating duty conditions for the case of 3 duty and 1 standby at 266 l/s per pump and for the case of 4 duty and no standby at 200 l/s per pump for a total delivery of 800 l/s for both cases.

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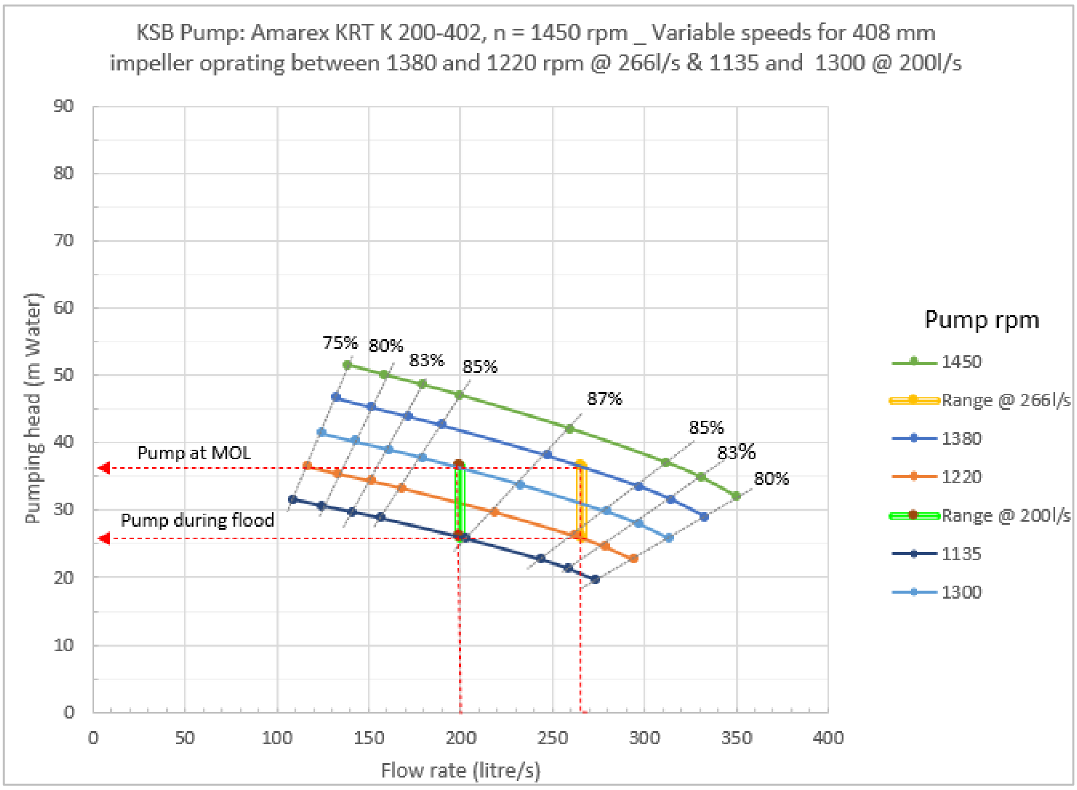


Figure 4-3: Operating ranges for the two possible duty ranges of the proposed KSB pumps to provide for variable river water levels.

5. Required submergence for raw water pumps

To prevent air entrainment (due to the tendency of vortex formation) at the pump intakes, sufficient submergence is required. The relevant excerpts from The American National Standard for Pump Intake Design, (ANSI/HI 9.8, 1998), which are shown in Figure 5-1, were used as a guideline together with the recommended KSB pump's inlet diameter of 200mm (refer **Appendix C**). It is assumed that the pump's body diameter of 0.735m (refer **Appendix C**) above the pump's intake, will dampen vortex formation and that the body diameter will have at least the same effect as the required bellmouth diameter of 0.45 m according to Figure 5-1. With these assumptions a minimum submergence of 1.5 m is required according to Figure 5-1.

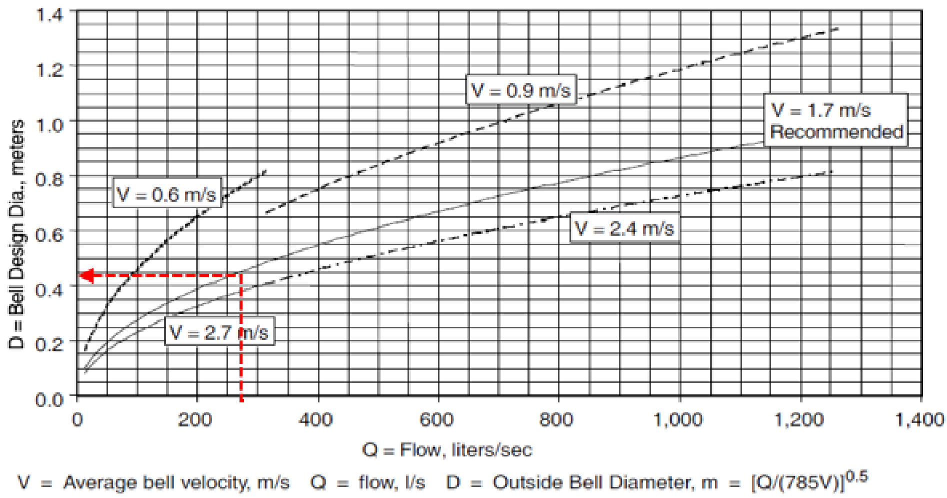


Figure 9.8.25A — Recommended inlet bell design diameter (OD)

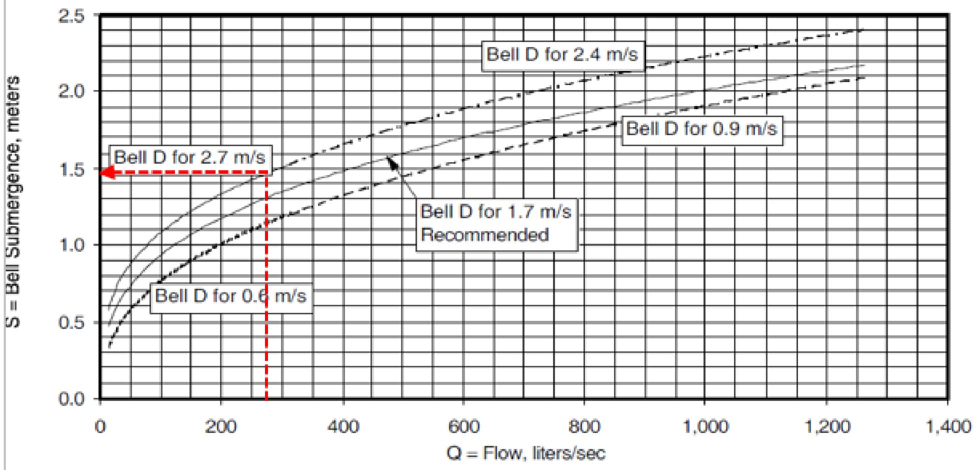


Figure 9.8.26A — Recommended minimum submergence to minimize free surface vortices

Figure 5-1: Required bellmouth diameter and submergence (ANSI/HI-9.8, 1998)

6. Required and available Nett Positive Suction Head (NPSH) for raw water pumps

The required Net Positive Suction Heads (NPSHreq) for the recommended KSB pumps are 4.5m for Case 1 and 8.5m for Case 2 according to Figure 4-2 and the available NPSH (NPSHA) values are calculated according to ANSI/HI-9.8 (1998) as shown in Excerpt 6-1. Table 6-1 shows the calculation of the Available Net Positive Suction Heads (NPSHA) for the two cases and indicates that the NPSHreq values are satisfied with the NPSHA values.

Excerpt 6-1: NPSH available after ANSI/HI-9.8 (1998)

8.3.4 Net Positive Suction Head Available

Net positive suction head available (NPSHA) is the head available above vapor pressure head to move a liquid into the impeller unit of the pump. It is necessary to ensure that the NPSHA exceeds the NPSHR to prevent cavitation. The following equation is used to compute NPHSA:

$$NPSHA = H_{pa} + H_s - h_f - H_{vp} \qquad (8\text{-}11)$$

where:

H_{pa} = the atmospheric pressure head on the surface of the liquid in the sump – m (ft).

H_s = static suction head of liquid. This is height of the surface of the liquid above the centerline of the pump impeller – m (ft)

h_f = total friction losses in the suction line – m (ft)

H_{vp} = the vapor pressure head of the liquid at the operating temperature – m (ft)

Table 6.1: Calculation of available Net Positive suction head for the two cases

Duty per pump = 200 l/s

Elevation of intake	2	masl	
Atmospheric head (Hpa)	10.3	m H2O	
Vapour pressure of water at 20°C (Hvp)	0.24	m H2O	
Friction head loss incl fall through screens	0.15	m	CHECK (assumed 0.2m, to confirm)
Submergence (S) of PUMP intake end at MOL - from ANSI	**1.5**	m	Based on ANSI/HI 9.8
Height of impeller suction face above intake face	0.20	m	See dimensions of selected pump
NPSHavailable= Hpa + Hs - hf - Hvp	11.2	m	
NPSHrequired from pump curves	4.5	m	See selected pump's curves
NPSHavailable / NPSHrequired	2.5	>1.3 OK	

Duty flow rate per pump = 266 l/s

Elevation of intake	2	masl	
Atmospheric head (Hpa)	10.3	m H2O	
Vapour pressure of water at 20°C (Hvp)	0.24	m H2O	
Friction head loss incl fall through screens	0.15	m	CHECK (assumed 0.15m, to confirm)
Submergence (S) of PUMP intake end at MOL - from ANSI	**1.5**	m	Based on ANSI/HI 9.8
Height of impeller suction face above intake face	0.20	m	See dimensions of selected pump
NPSHavailable= Hpa - Hs - hf - Hvp	11.2	m	
NPSHrequired from pump curves	8.5	m	See selected pump's curves
NPSHavailable / NPSHrequired	1.32	>1.3 OK	

7. Jet-pump selection for removal of settled sediment in hoppers

It is expected that sediment sizes of 0.4 mm and larger will settle in the hopper in the pump forebay. A jet-pump, which is permanently installed at the invert of each of the two hoppers, is proposed to remove the settled sediment intermittently. The sediment level in the hopper can be automatically monitored by means of a sensor such as the ultrasonic-type bed level sensor used in wastewater treatment works for the continuous sludge/water interface monitoring in sludge thickeners. The advantage of a jet-pump is that it has no moving parts and operates at its best when it is buried in sediment. An example of the type of jet-pump recommended is shown in Figure 7-1 and its operation in Figure 7-2 – refer also to **Appendices E and F**.

COMPONENT PARAMETER	TECHNICAL DETAIL
Production Rate	30 t/h (approx 18 m^3 bulk volume sand per hour
Supply head of motive pump	700 kPa
Supply flow rate to main jet	10 litres/second
Supply flow rate to fluidizer nozzle	2.5 litres/second
Induced suction flow rate	9 litres/second
Head at diffuser outlet	11.7 m
Mixing Chamber diameter	44 mm
Jet pump grid size at suction end	30 mm x 30 mm
Main jet diameter	19 mm
Motive pipe length	50 m
Motive pipe diameter	100 mm
Motive pipe velocity	1.5 m/s
Motive pipe friction head loss	2.5 m per 100 m pipe length
Jet pump depth below water surface	3 m
Discharge pipe length	50 m
Discharge line diameter	100 mm
Discharge pipe flow velocity	2.5 m/s

Figure 7-1: Example of a jet-pump by GENFLO used in a WRC field evaluation research project (WRC, 2002).

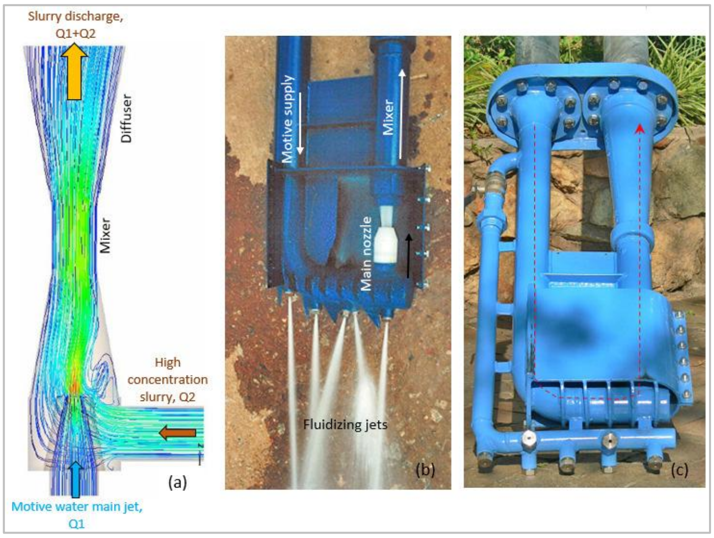


Figure 7-2: Basic operation of a jet-pump. Motive water is supplied in the form of a high velocity jet through a large main nozzle across a gap between the main nozzle and a receiving mixer where the sediment (entrained at the gap) and motive water is mixed and flows via a diffuser to a discharge pipe. The motive water also supplies fluidization water via smaller nozzles to liberate the settled sediment near the suction end of the jet-pump.

The main feature of the jet-pump recommended for the Moordkuil raw water intake is as follows:

- Sediment production rate: 38 t/hr (approx. 23 m^3/hr)
- Mixer inside diameter: 62.5 mm
- Main nozzle diameter: 31 mm
- Fluidization nozzle diameter: 5 mm
- Required motive head: 65 m
- Motive water supply rate (including 10% fluidization water): 27 + 3 = 30 l/s
- Slurry discharge rate: 37 l/s
- Discharge pipe inside diameter: 125 mm
- Motive water supply pipe inside diameter: 100 mm
- Head loss through 20 m long, 100 mm diameter motive pipe: approx. 3 m
- Head loss trough 30m long, 125 mm diameter slurry pipe: approx. 4.5 m (refer **Appendix H**)

The performance curves of a jet-pump with the features are presented in **Appendix D**. A supplier of jet-pumps in South Africa is GENFLO Dredging (https://www.genflo.co.za/). GENFLO Dredging has the software to refine the specifications of a jet-pump based on the required performance and the configuration of the civil works of a project and it is recommended that GENFLO Dredging be approached during the detail design phase for further refinement of the design and a description of its operation.

It is proposed that the motive water be supplied by a submersible pump (located in the river works with the main pumps) at a head of 65 m at the jet-pump inlet (point X in **Appendix G**). The pump selected for the jet-pump's motive pump is the submersible KSB KRT 080-315 pump – refer **Appendix J** for the technical detail of this pump. This pump should also be provided with a cooling jacket and variable frequency motor. The motive water should pass through a screen filter to ensure that the 5 mm diameter fluidization nozzles do not clog. The filter could be of the screen-type which has a relatively small head loss, similar to the type as shown in **Appendix I** and used in farm irrigation systems. The filter system can be accommodated in the existing pump house.

The motive and slurry pipes of the jet-pump could be of either rigid or flexible (hose) type. The latter is for practical reasons considered more appropriate as demonstrated in **Appendix G**. The hose-type alternative has the advantage that the jet-pump could be launched while active (to dig itself into the settled sediment towards the hopper invert). Also, it enables the jet-pumps to be inspected/serviced by lifting the jet-pump to the top of the pump bay without disconnecting the motive supply and slurry discharge pipes. Inspection of a jet-pump from time to time is necessary to establish possible clogging of the fluidization nozzles, and inspection/replacement of the main nozzle and mixer which are subjected to wearing.

8. 2D Hydrodynamic modelling with the proposed new low abstraction works intake located on the left bank protruding bedrock upstream of the existing pumpstation

Before a final decision could be made on the proposed new intake location on the protruding bedrock at the left bank upstream of the existing pumpstation, 2D hydrodynamic modelling was required with movable bed conditions to evaluate whether the proposed intake location will be self-scouring during floods, and to evaluate possible other impacts of the new low intake on the flow patterns and sediment dynamics.

8.1. Model setup

A two-dimensional model Mike 21C of the DHI group was used to simulate the flow patterns and sediment deposition and erosion near the proposed pumpstation intake.

The model was set up based on the new (2025) topographical and underwater survey data. The low water causeway and the bridge downstream of the pumpstation were included in the model. The annual recurrence interval floods shown in Figure 8-1 were routed through the river to simulate scour and deposition patterns. The model was set up considering the following:

a) Downstream boundary conditions were taken from the ASP (2014) study where the downstream water levels considered the year 2060 sea level rise. The backwater effect downstream of the 2D model bathymetry was also simulated by 1D model in the 2014 study and for low river flow conditions tidal effects were considered. The corresponding downstream water levels are shown in Figure 8-2.

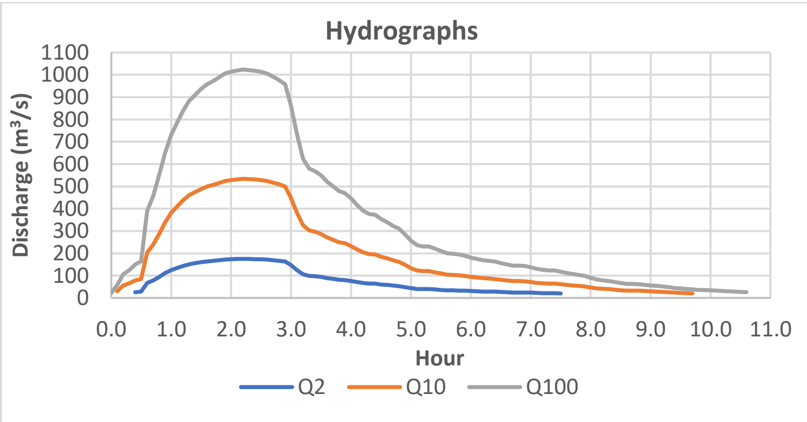


Figure 8-1: Flood hydrographs used in the model

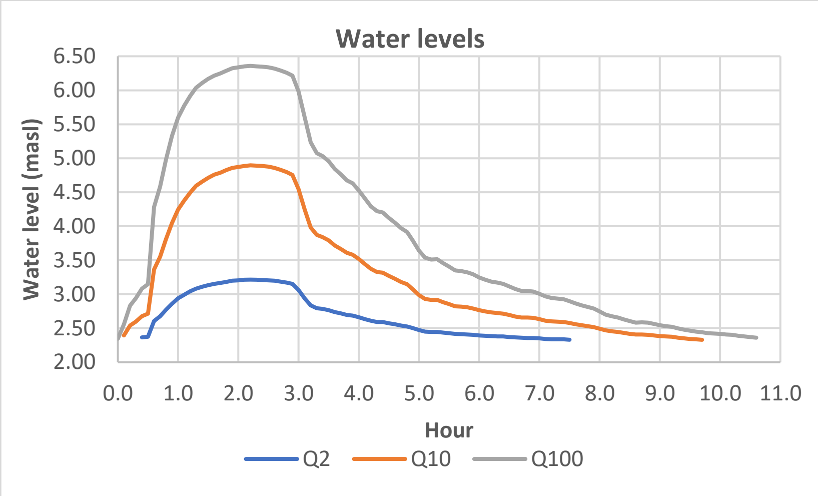


Figure 8-2: Downstream tailwater levels used in the model

b) The riverbed sediment grading was obtained from bed grab sampling and laboratory analysis (2014 study) as shown in Table 8-1.

Table 8-1: Sediment fractions used in the model

fraction no.	mm	% bed	Description
d1	0.002	7	clay
d2	0.03	23	silt
d3	0.11	40	sand
d4	0.45	20	sand
d5	4.00	10	sand

d) Manning roughness n = 0.045 was used in the main channel and n = 0.055 on the floodplains.

The surveyed 2025 bathymetry of the 2D model is shown in Figure 8-3. The elevations in the figure are shown as masl (refer to legend). The existing pumpstation and the proposed intake with a top of structure elevation of 2.5 masl are shown in the bathymetry. The surveyed reach has a bed level in the main channel below mean sea level.

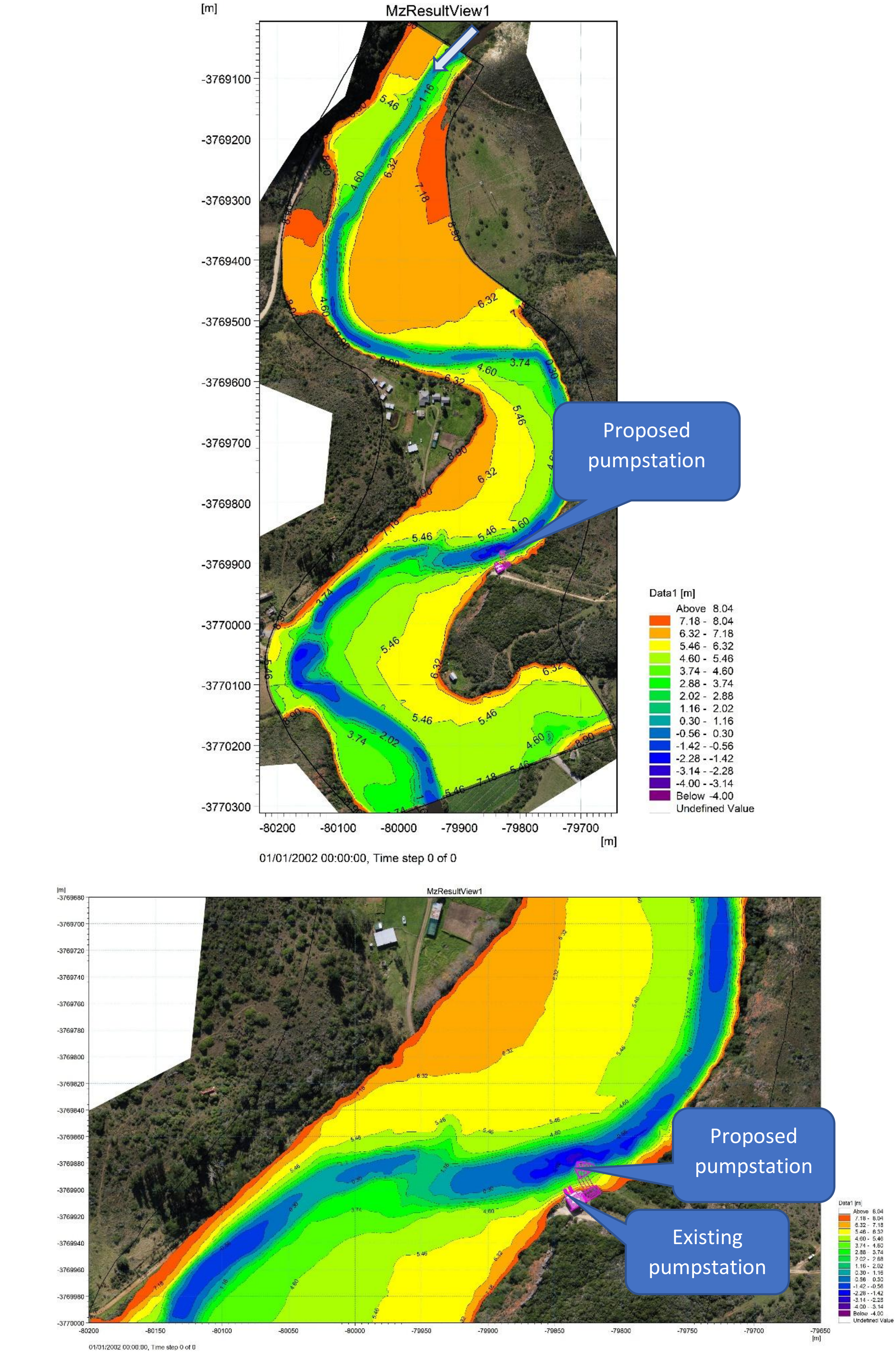


Figure 8-3: 2D model bathymetry for the current scenario based on the topographical survey (masl) (bottom picture: zoomed view near the proposed pumpstation)

The new survey shows that there is bedrock protruding from the left bank upstream of the existing intake. The proposed pump station is located on the above-mentioned protruding rock upstream of the existing pump station.

Simulations were carried out for the current scenario with the proposed new pumpstation intake added. The left bank of the river upstream of the pumpstation was made non-erodible (bedrock) in the model.

8.2. Current bathymetry with proposed new upstream intake on rock scenario

The simulated flow patterns (velocities, flow depths and water levels) are shown in Figures 8-4 to 8-12 for the 2-year, 10-year and 100-year floods respectively. The simulated velocities indicate that the pumpstation location is in a good position, with high flow velocities near the left bank which would help to scour the future pump intake during floods.

During the 2-year flood and 100-year flood the water levels 10 m upstream of the pumpstation in the are 4.2 masl and 7.6 masl, respectively.

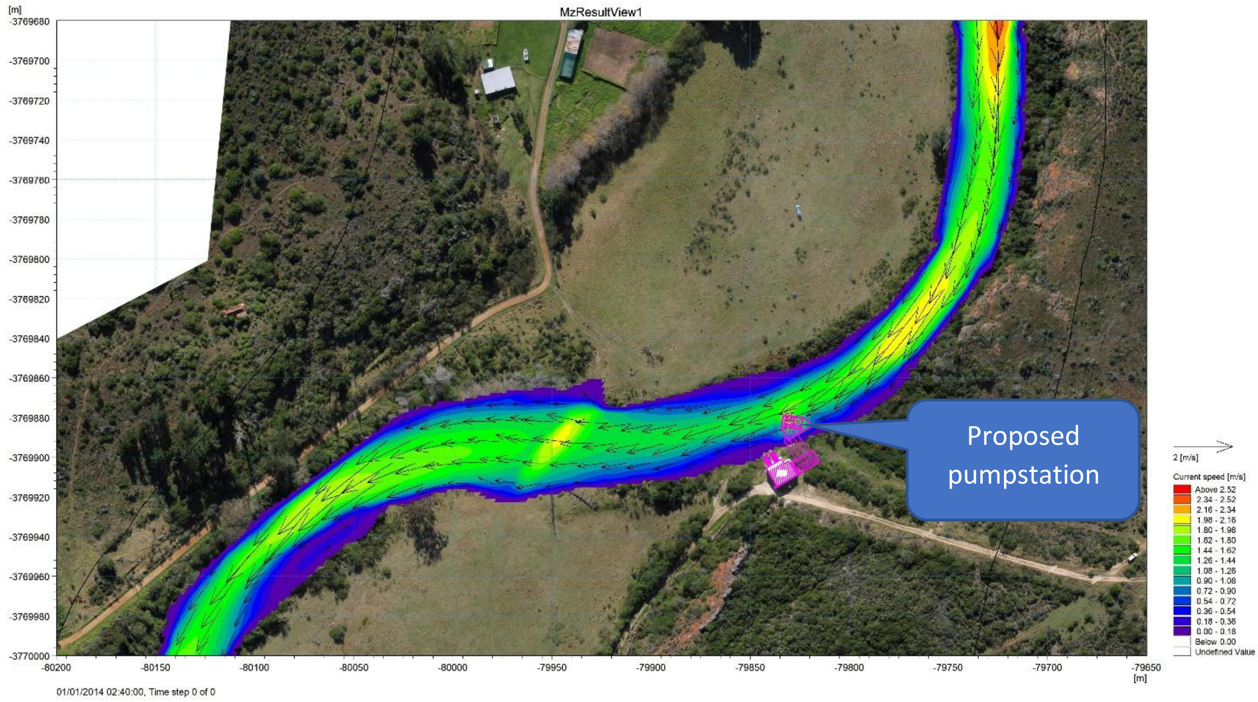


Figure 8-4: Simulated flow velocities during the peak of the 2-year flood

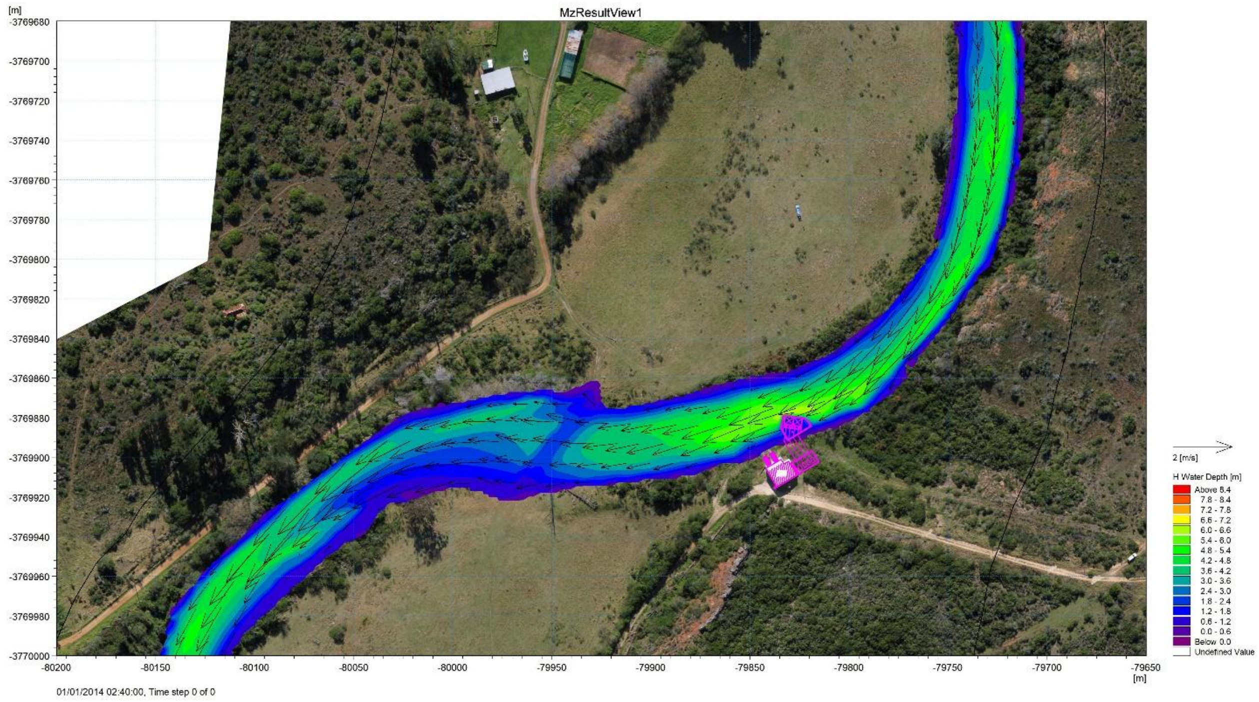


Figure 8-5: Simulated flow depths during the peak of the 2-year flood

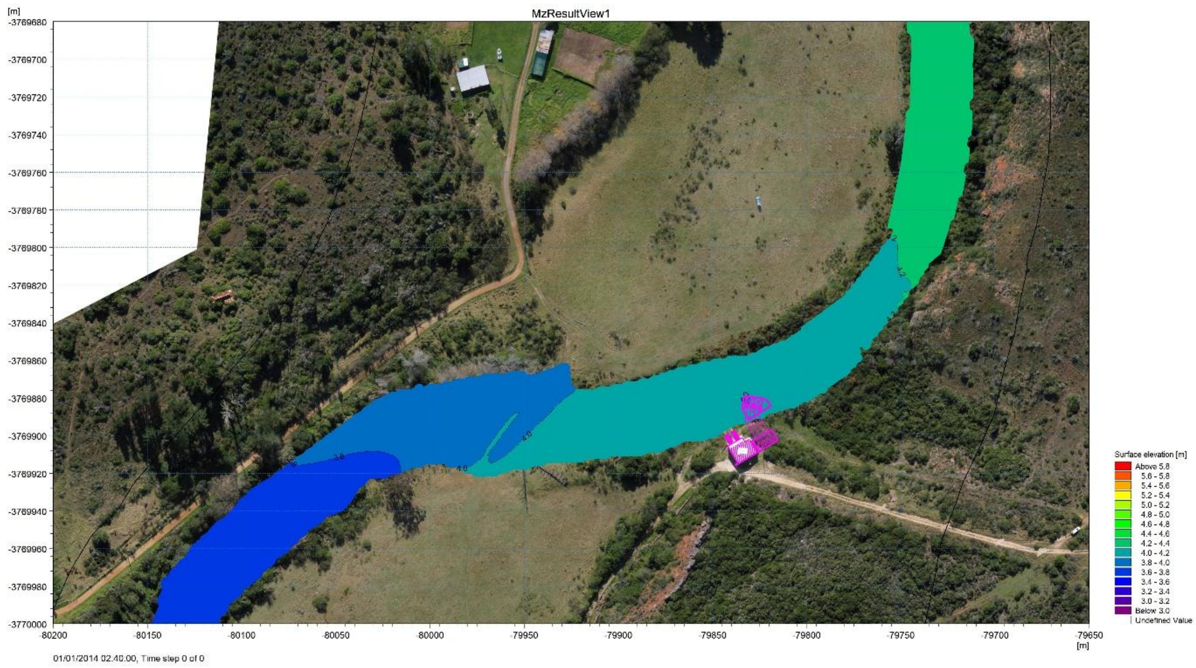


Figure 8-6: Simulated water levels during the peak of the 2-year flood (masl)

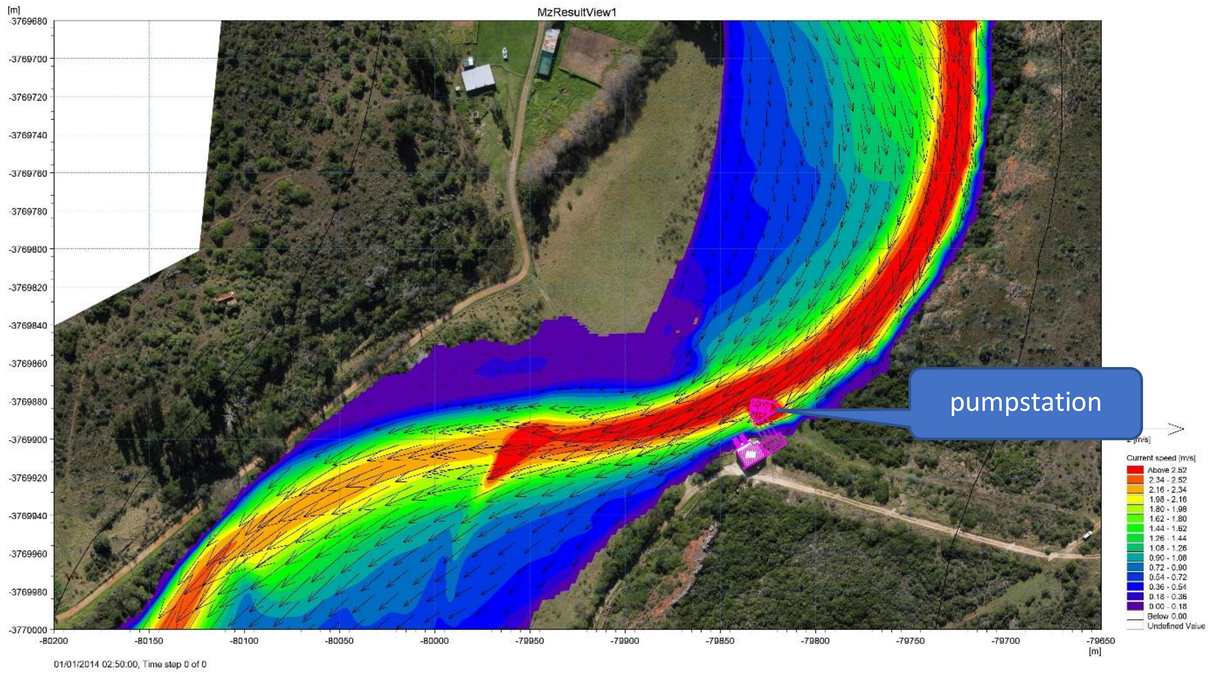


Figure 8-7: Simulated flow velocities during the peak of the 10-year flood

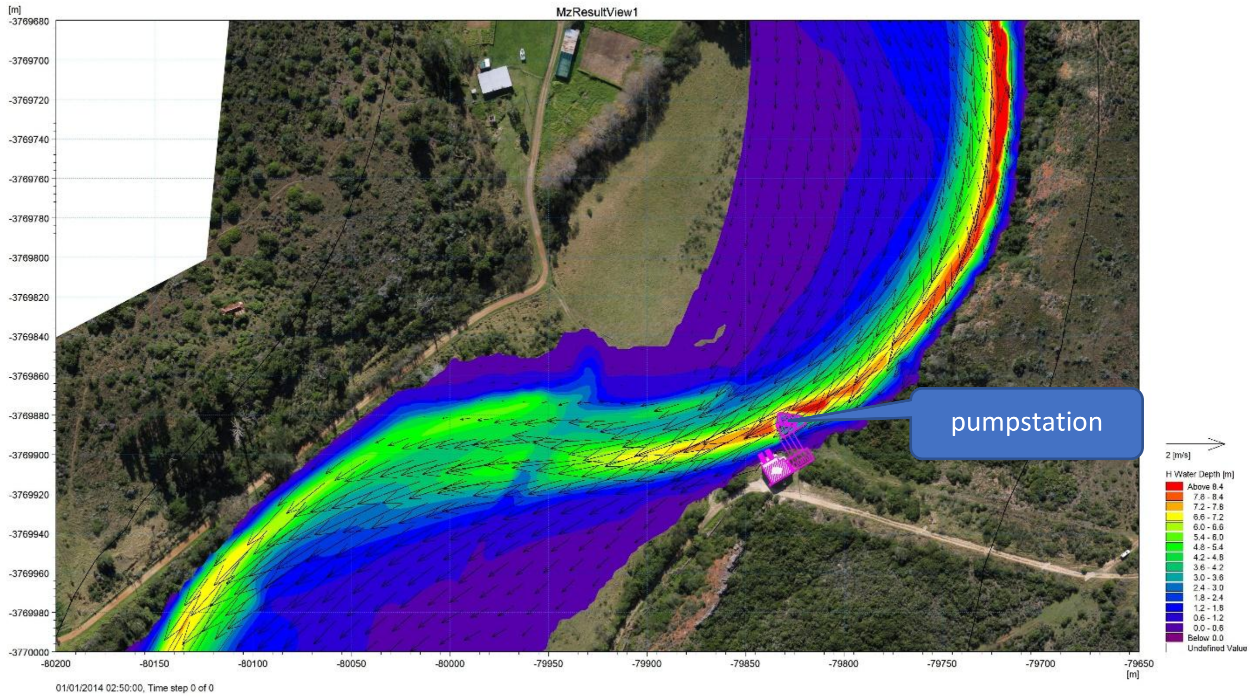


Figure 8-8: Simulated flow depths during the peak of the 10-year flood

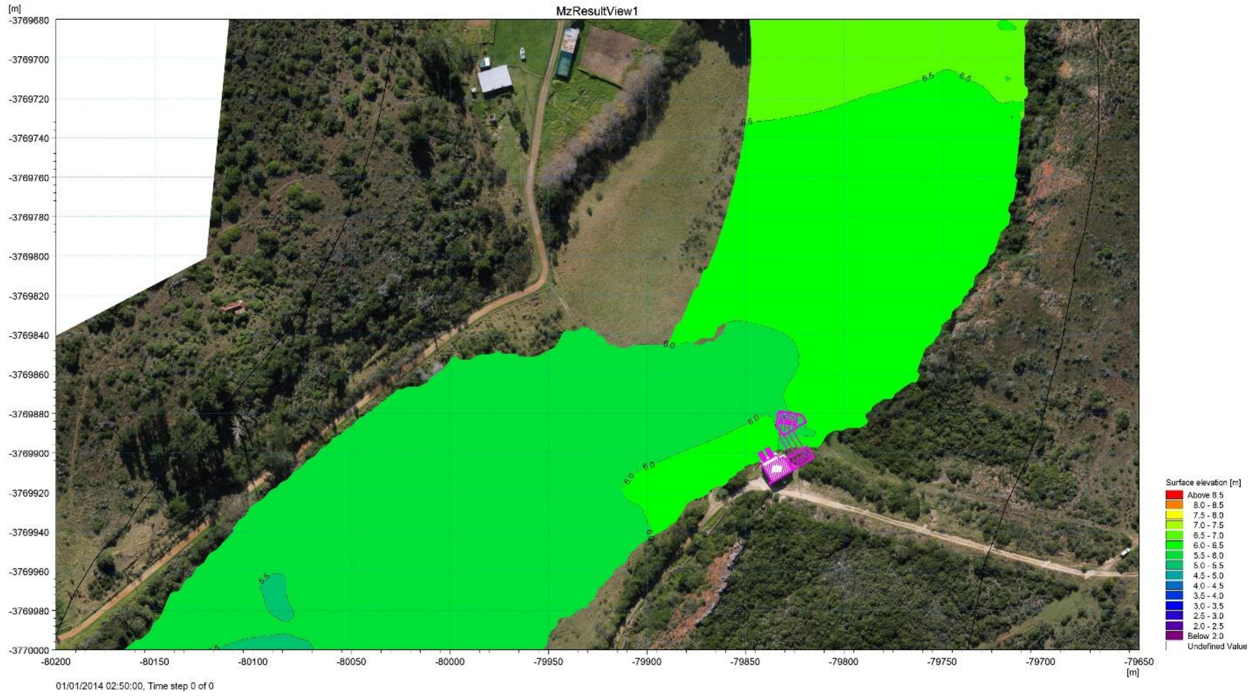


Figure 8-9: Simulated water levels during the peak of the 10 year flood

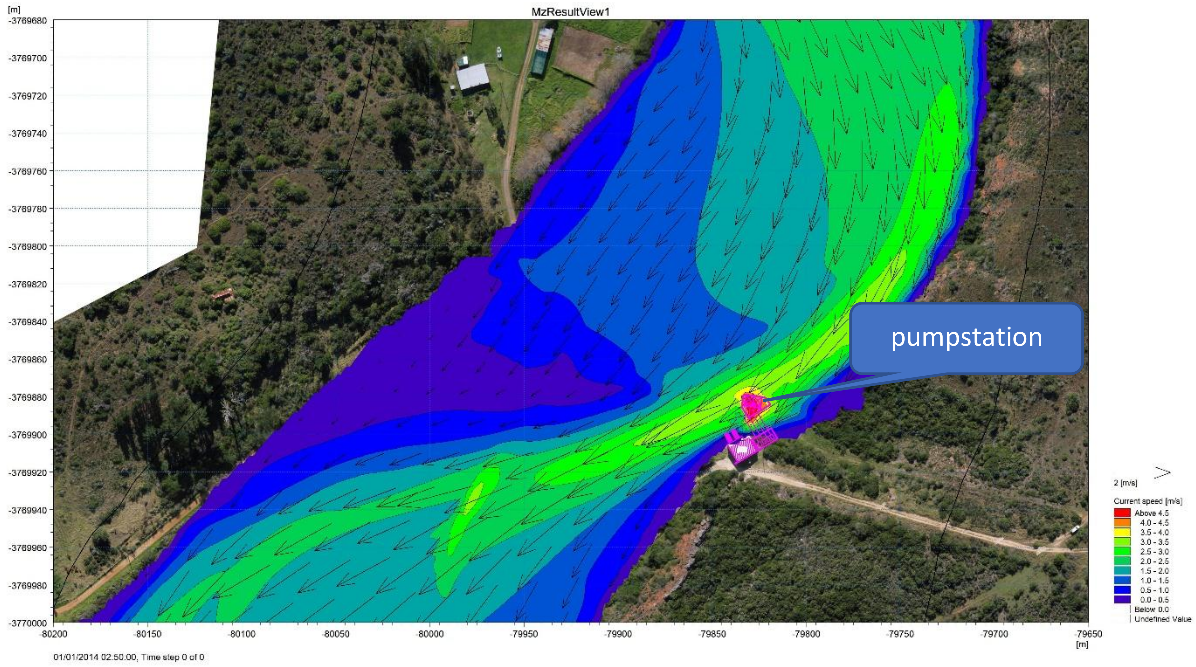


Figure 8-10: Simulated flow velocities during the peak of the 100-year

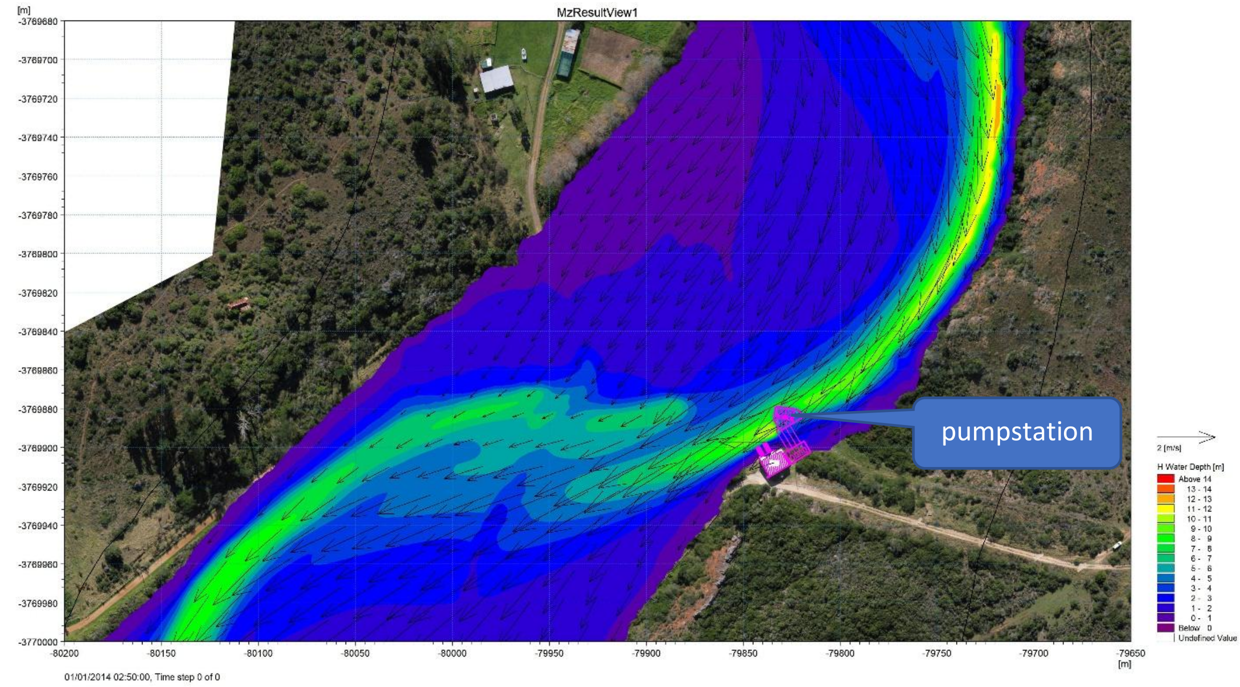


Figure 8-11: Simulated flow depths during the peak of the 100 year flood

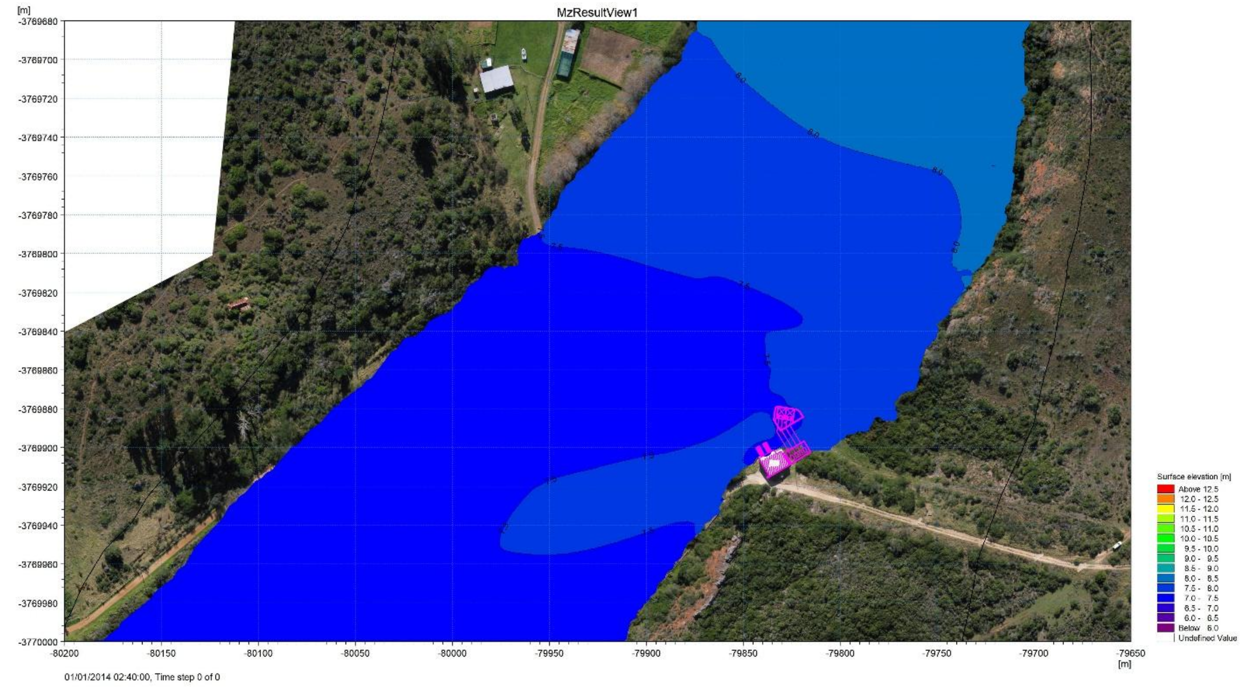


Figure 8-12: Simulated water levels during the peak of the 100 year flood

Figures 8-13 to 8-18 show the simulated bed levels and bed level change following the 2-year, 10 year and 100-year floods, respectively. In these simulations the causeway and the approach roads were specified as non-erodible in the model. The left bank of the river upstream of the pumpstation was also specified as non-erodible (bedrock). During all the floods sediment deposits (positive depths) at the inside of the river bend opposite the pumpstation. At the proposed pump intake during a 2-year flood the bed scoured between 1 m to 1.8 m deep. During larger floods (10-year flood and 100-year flood) the model simulated a large amount of sediment deposit opposite to the abstraction intake closer to the right bank. However, the area near the proposed abstraction intake scoured 1.30 m deep during 10-year flood and 0.90 m deep during the 100 year flood. More simulation results are provided in **Appendix K**.

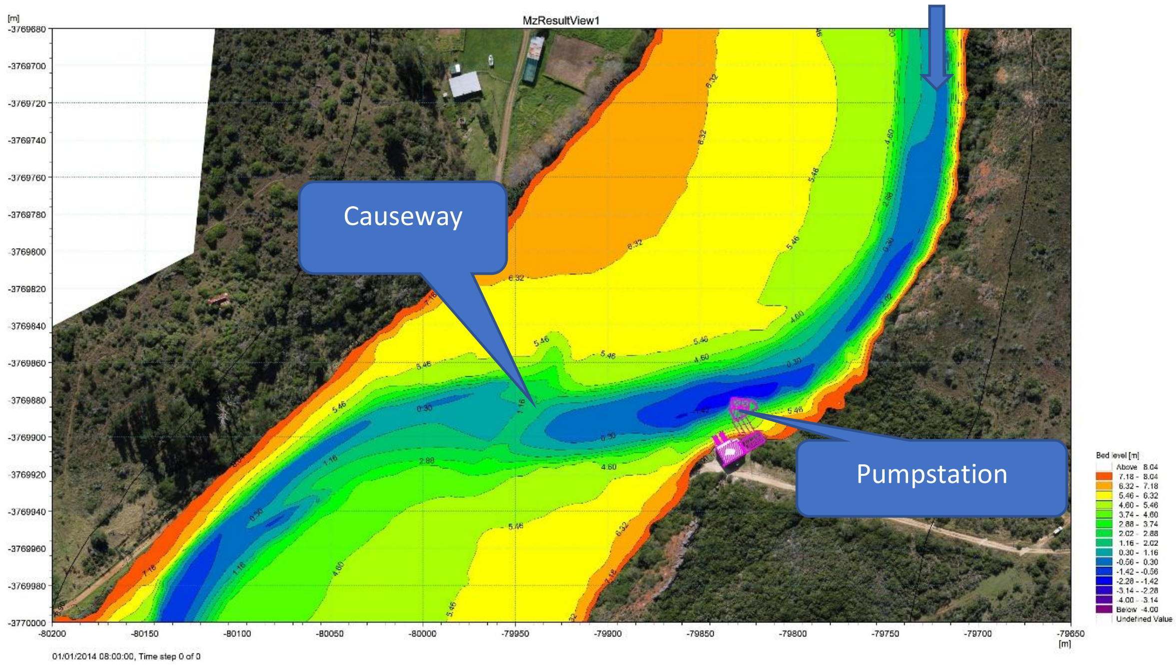


Figure 8-13: Simulated bed level following the 2-year flood

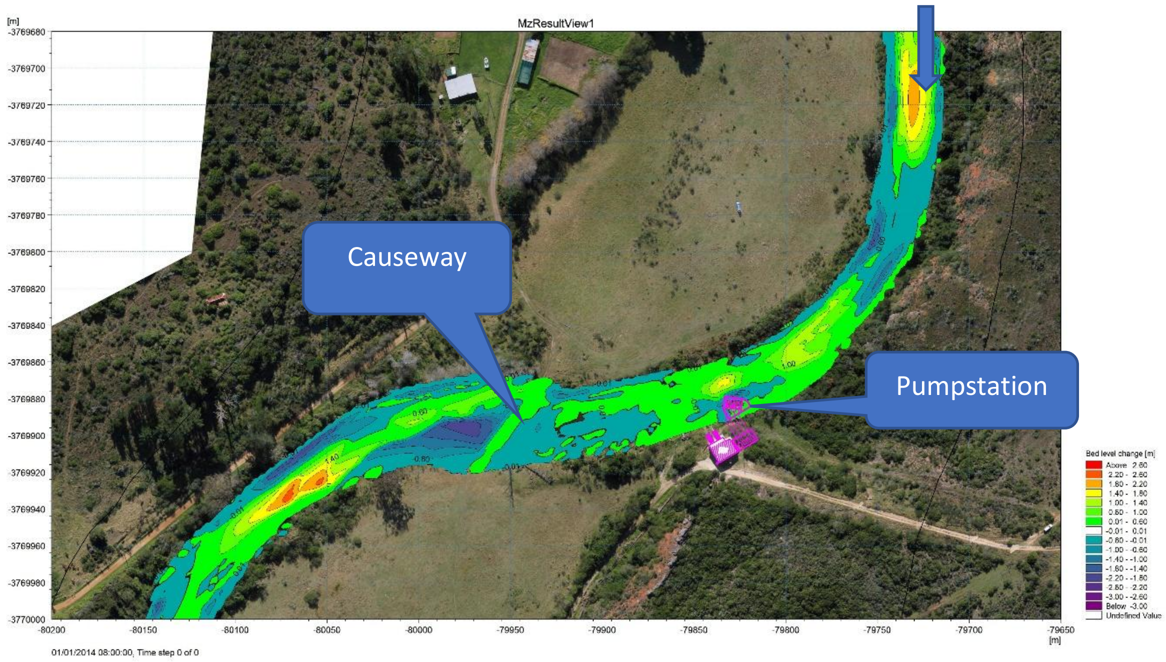


Figure 8-14: Simulated bed level change after the 2-year flood (positive values = deposition; negative values = erosion)

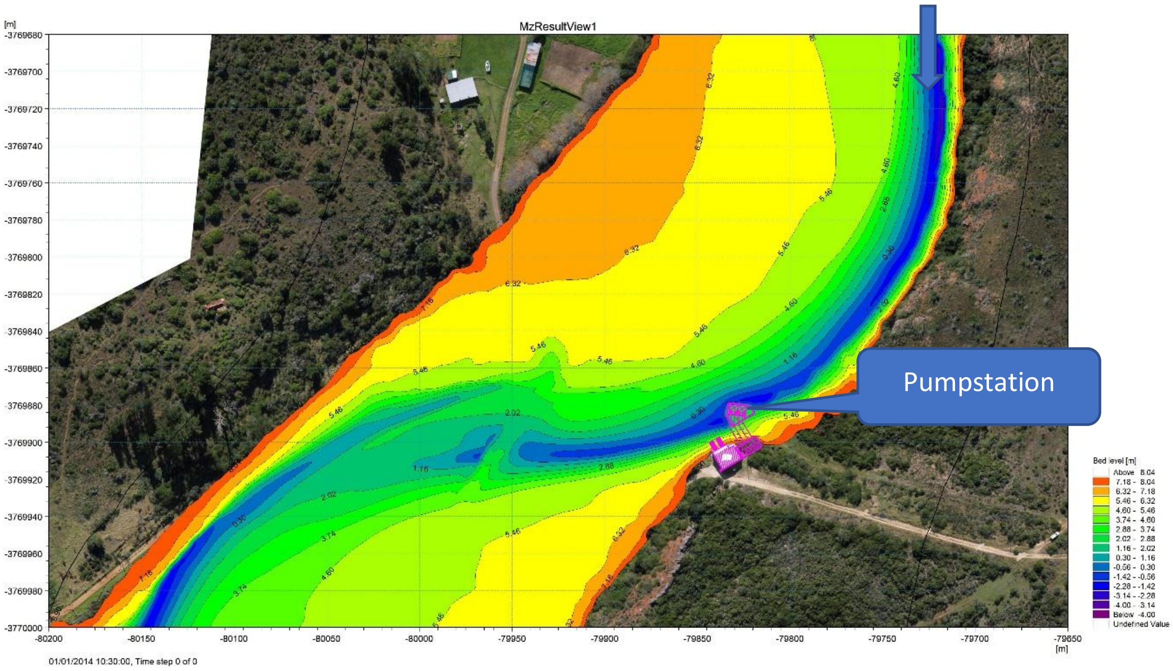


Figure 8-15: Simulated bed level following the 10-year flood

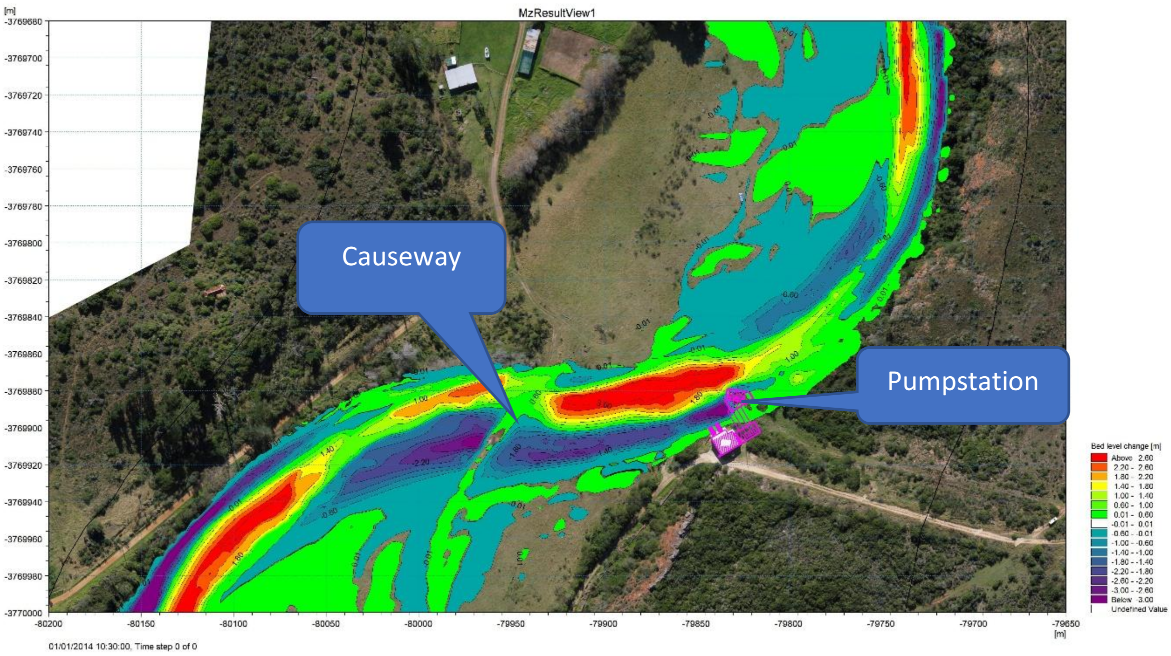


Figure 8-16: Simulated bed level change after the 10-year flood

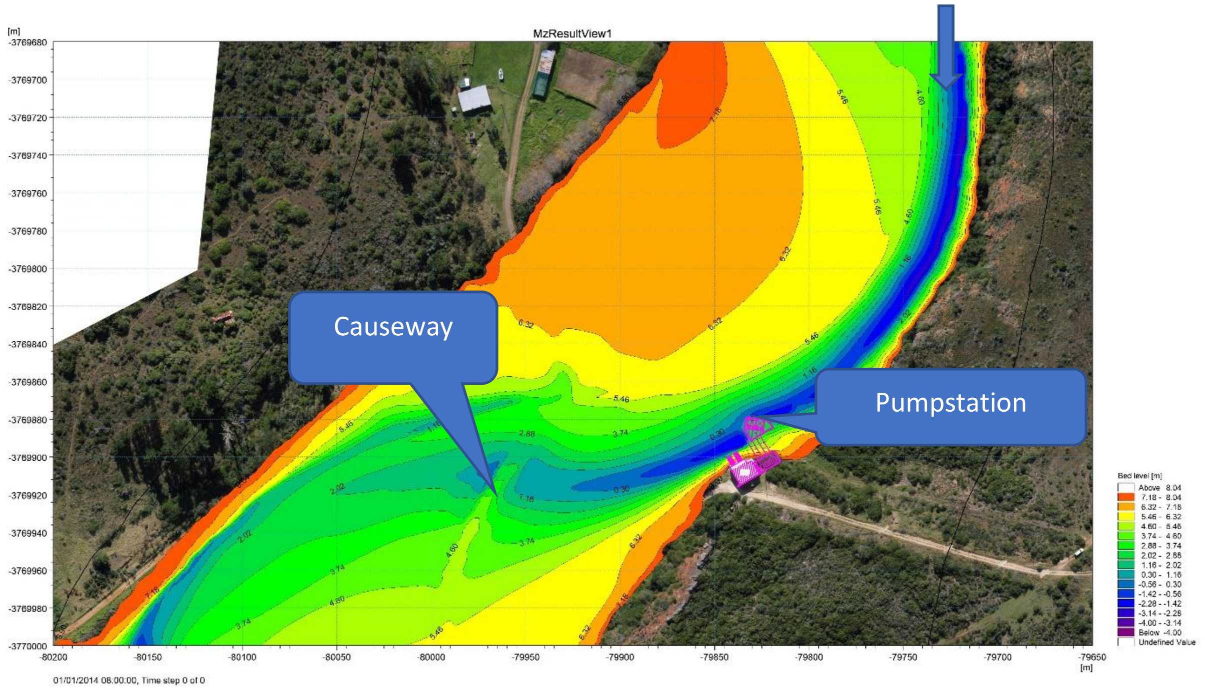


Figure 8-17: Simulated bed level following the 100-year flood

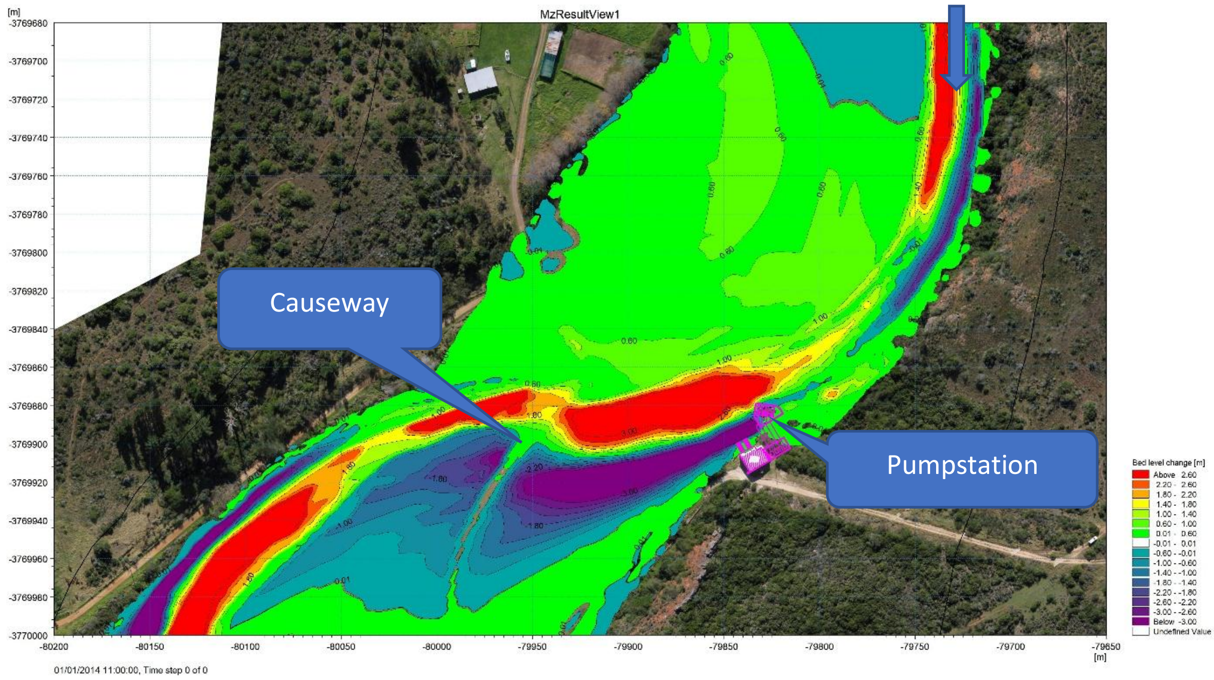


Figure 8-18: Simulated bed level change after the 100-year flood

8.3.Summary of findings

The key findings from the 2D hydrodynamic modelling of the sediment dynamics with the proposed intake located at the left bank on the bedrock upstream of the existing pumpstation are:

- The proposed intake is in scour zone at the outside of the bend and from small to large floods the proposed intake is self-scouring.
- The proposed intake is relatively low and submerged during the floods and therefore does not deflect the flow towards the right bank. The simulations for all the floods indicated that the

inside of the bend (right bank opposite the proposed intake) is a sediment deposition zone. Therefore, no erosion protection is required at the right bank.

- The left bank and floodplain between the proposed intake and the causeway, is scoured during the 10-year and 100-year floods. The possible erosion should be monitored, and critical infrastructure should be protected against scour.
- The proposed intake location on the left bank bedrock upstream of the existing pumpstation should be used for the detailed design of new river abstraction works.

9. 3D CFD modelling with FLOW3D-HYDRO of the hydraulic forces on the sheetpile and piling intake works

(To be included in the follow-up version of this report)

10. Concluding Remarks

The following are main concluding remarks to this investigation:

a) The 2025 and 2014 local surveys at the proposed abstraction works were compared to identify any scour and deposition that occurred in the river within the elapsed period of 11 years, which might affect the original proposed intake location of the upgraded abstraction works. Scour was observed on the outside bend of the river and deposition on the inside bend of the river as expected, however the upstream rock obstruction at the left bank caused deposition further downstream where the proposed intake works is located.

b) The proposed dimensions of the LYNERS proposed hoppers should be increased to at least 4 m x 4 m in plan with side slopes of 1H:2V (implying a hopper depth of about 4m). The reasons for the recommended increase are for the effective settlement of the coarser fraction of suspended sediment (sediment larger than about 0.4 mm) and to increase the volume of the hopper for less frequent sediment removal by jet-pumps. Each hopper should be provided with a permanently installed jet-pump for the intermittent removal of settled sediment from the hoppers. The proposed installation method for the jet-pumps is presented in **Appendix G**.

c) The purchased submersible type pumps are suitable for pumping raw river water containing some sediment and debris. However, the purchased booster type pumps are less suitable to pumping raw water containing some sediment and debris.

d) For a pumping system with a relative low pumping head (such as for this project with a static head of about 30 m) it is considered that the pumping capacity of the raw water pumps in the river works should be selected such that they can pump directly (without the assistance of a booster pump) to the Klipheuwel Dam. This will significantly simplify the operation and consequently increase the reliability of the pumping system.

e) Based on the reasoning under Item (d) above, four KSB KRT 200-402 pumps are recommended with two alternative cases:
   - Case 1: Four duty pumps with no standby with each delivering 200 l/s for a total discharge of 4 x 200 l/s = 800 l/s.
   - Case 2: Three duty and one standby (for improved reliability) with each delivering 266 l/s for a total of 3 x 266 l/s = 800 l/s

f) Cooling jackets are recommended for the proposed submersible pumps for additional protection and variable frequency motors are recommended to operate at their best efficiency both during normal river flow and flood flow conditions.

g) A separate submersible pump (KSB 80-315 – refer **Appendix J**) to be accommodated in the river pump bay with the main pumps is recommended for the jet-pumps' motive pump and the motive water from it should pass through a filter to prevent clogging of the fluidization nozzles of the jet-pumps. The filter system can be accommodated in the existing pump house.

h) The key findings from the 2D hydrodynamic modelling of the sediment dynamics with the proposed intake located at the left bank on the bedrock upstream of the existing pumpstation are:
   - The proposed intake is in scour zone at the outside of the bend and from small to large floods the proposed intake is self-scouring.
   - The proposed intake is relatively low and submerged during the floods and therefore does not deflect the flow towards the right bank. The simulations for all the floods indicated that the inside of the bend (right bank opposite the proposed intake) is a sediment deposition zone. Therefore, no erosion protection is required at the right bank.

- The left bank and floodplain between the proposed intake and the causeway, is scoured during the 10-year and 100-year floods. The possible erosion should be monitored, and critical infrastructure should be protected against scour.
- The proposed intake location on the left bank bedrock upstream of the existing pumpstation should be used for the detailed design of new river abstraction works.

i) Results on forces on the river intake works derived by 3D modelling will be reported on in follow-up versions of this report.

11. References

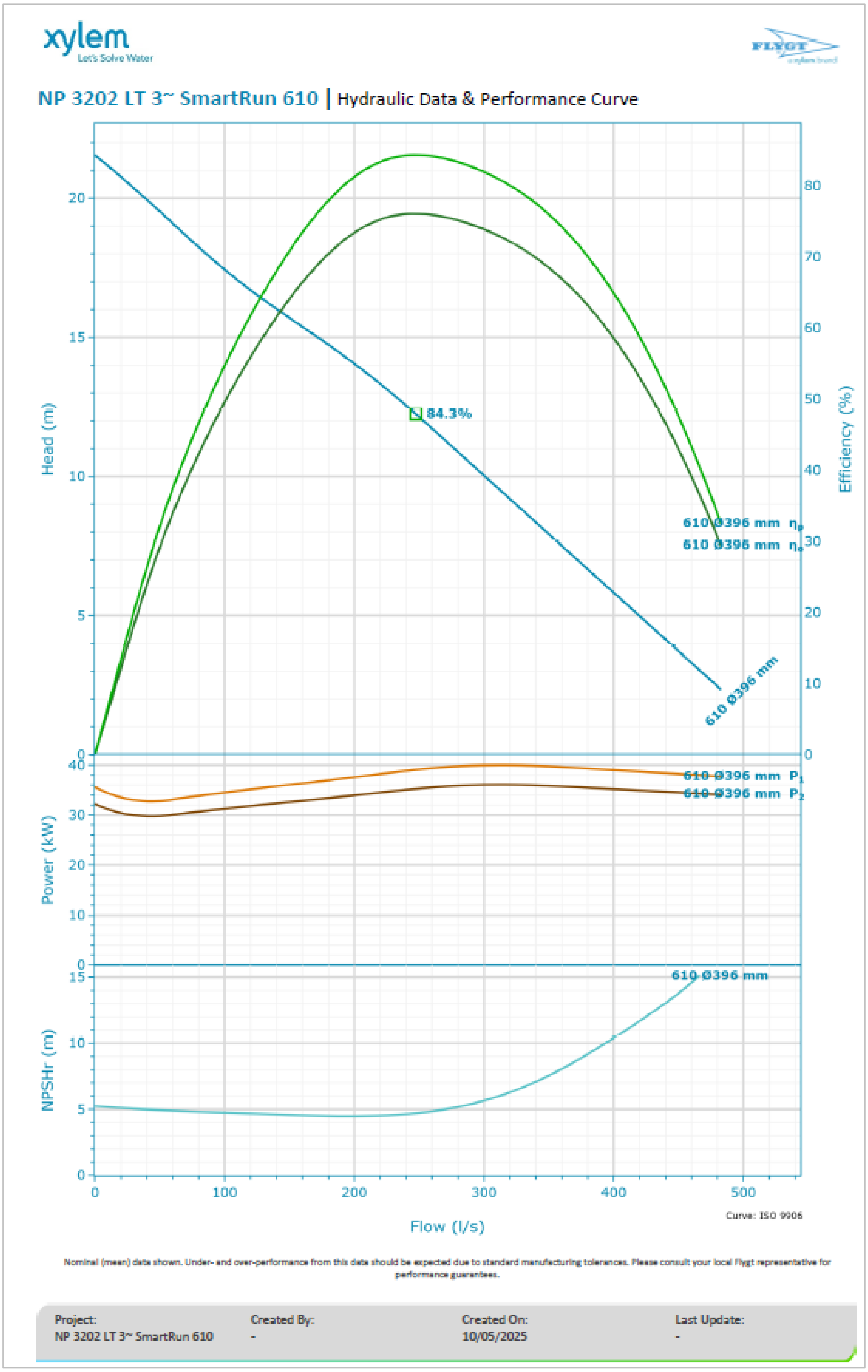
ASP (2014). Moordkuil raw water pumpstation rehabilitation - hydrology, hydraulics and sediment dynamics. Report to Lyners.

LYNERS (2016). Moordkuil Raw Water Intake and Pumping Station: Implementation Report.

Miedema, S., & Ramsdell, RC. (Ed.) (2016). Slurry Transport: Fundamentals, a Historical Overview & The Delft Head Loss & Limit Deposit Velocity Framework (DHLLDV). (1st ed.) SA Miedema / Delft University of Technology.

WRC (2002). AN INVESTIGATION INTO THE REMOVAL OF SEDIMENTS FROM WATER INTAKES ON RIVERS BY MEANS OF JET-TYPE DREDGE PUMPS. Report prepared for the Water Research Commission as a contract research project with Prestedge Retief Dresner Wijnberg. Authors: Bosman D.E., Prestedge G.K., Rooseboom A, Slatter P.T. https://www.wrc.org.za/wp-content/uploads/mdocs/1187-1-021.pdf

Appendix A: Flygt NP 3202 LT pump – Technical detail



xylem
Let's Solve Water

FLYGT
a xylem brand

Selection

Series
Experior

Name
NP 3202 LT 3~ SmartRun 610

Frequency
50 Hz

System Type
Single Pump

Operating Pumps
1

Standby Pumps
No Standby Pump

Curve Code
610

Impeller Diameter
396 mm

Inlet Diameter
300 mm

Outlet Diameter
300 mm

Number Of Vanes
2

Fluid

Fluid Type
Water

Fluid Temperature
4 °C

Specific Gravity
1

Density
1,000 kg/m³

Dynamic Viscosity
0.001567 Pa·s

Fluid Vapor Pressure
8.135 mbar

Design Curve

Rated Speed
50 Hz

Max Flow
482.92 l/s

H@QMin
21.55 m

H@QMax
2.33 m

BEP
84.3 %

BEP Flow
247.52 l/s

BEP Head
12.24 m

Max P2
36.06 kW

Project:	Created By:	Created On:	Last Update:
NP 3202 LT 3~ SmartRun 610	-	10/05/2025	-

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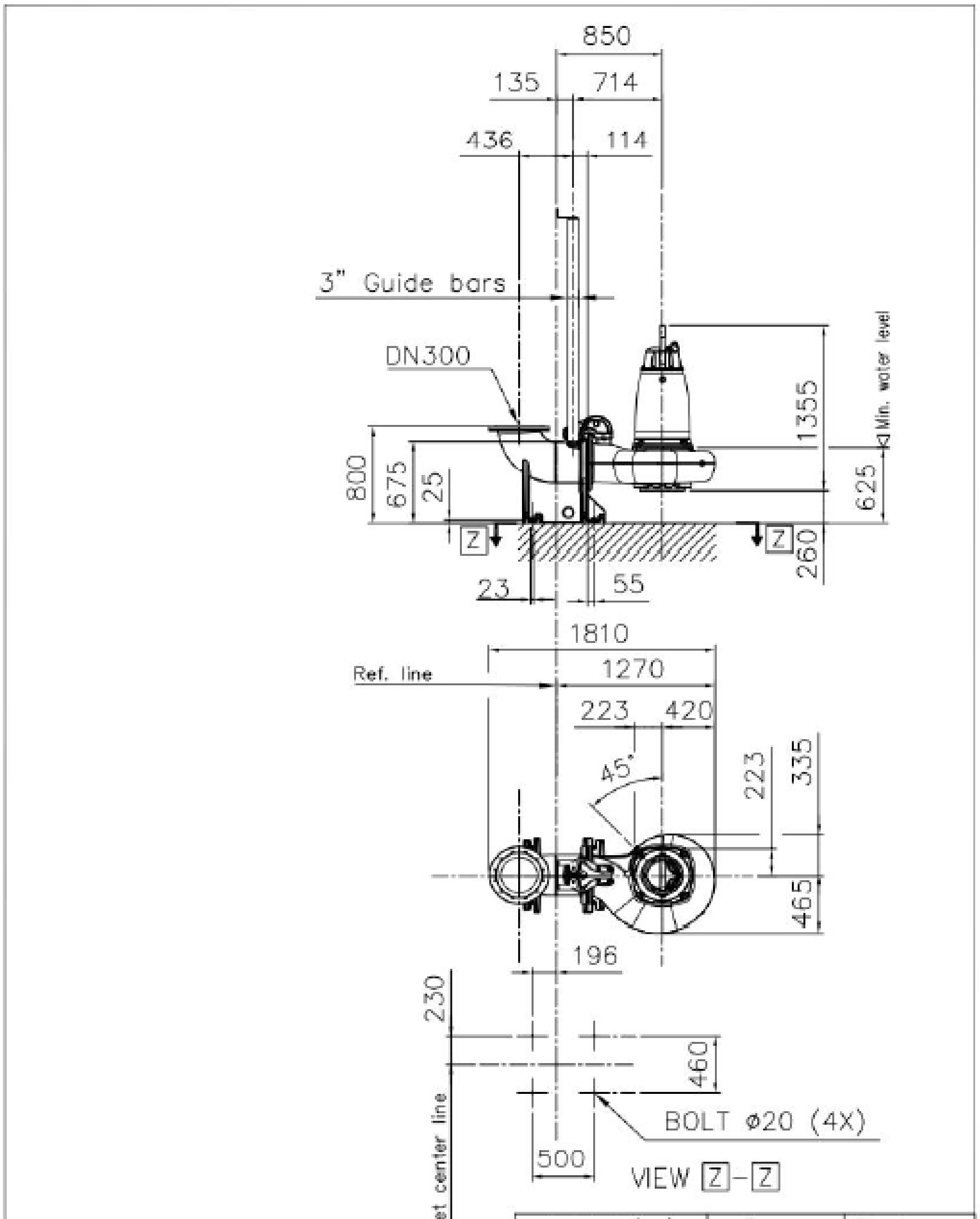
NP 3202 LT 3~ SmartRun 610 | Dimensional Data & Drawing



Weight (kg)	Pump	Discharge
with cooling jacket	790	291
without cooling jacket	740	291

NP	3202	LT	Discharge outlet DN 300	Pump outlet DN 300	Scale 1:40	Date 210312	
090,095,180,185,660,670					Drawing number 6841500	Revision 7	

Project:	Created By:	Created On:	Last Update:
NP 3202 LT 3~ SmartRun 610	-	10/05/2025	-

xylem
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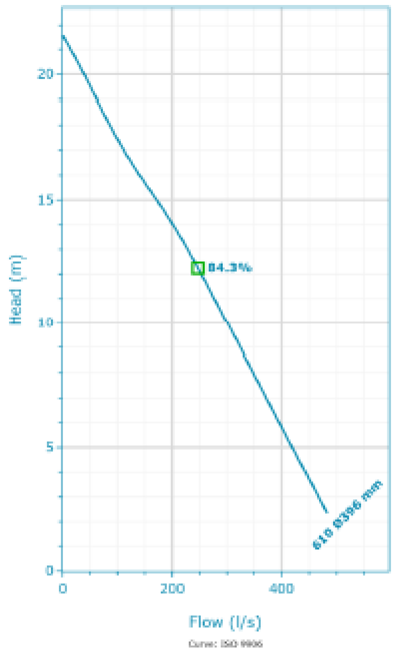
NP 3202 LT 3~ SmartRun 610 | Configuration Summary



Patented self cleaning semi opened channel impeller, ideal for pumping in wastewater applications. Modular based design with high adaptation grade. Equipped with a user-friendly intelligent control unit, pre-programmed for wastewater pumping used for pumping station with one or two pumps.

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances. Please consult your local Flygt representative for performance guarantees.

Installation

Installation Type
P - Semi-Permanent, Wet

Motor

Rated Voltage	**Motor Efficiency Class**
400 V	Standard
Coupling	**Rated Power**
D	37 kW

Materials

Impeller Material
Grey Cast Iron
Volute Material
Grey Cast Iron

Performance

Explosion Proof	**Impeller Diameter**
No	396 mm
Max. Pumped Media Temp.	
40 °C	

Project:	Created By:	Created On:	Last Update:
NP 3202 LT 3~ SmartRun 610	-	10/05/2025	-

xylem
Let's Solve Water

FLYGT
a xylem brand

NP 3202 LT 3~ SmartRun 610 | Product Details

Description

N 3202

Flygt Experior

Patented self cleaning semi opened channel impeller, ideal for pumping in wastewater applications. Modular based design with high adaptation grade. Equipped with a user-friendly intelligent control unit, pre-programmed for wastewater pumping used for pumping station with one or two pumps.
The Flygt N-series are equipped with the Flygt invented N-technology with its innovative self-cleaning impeller. Solid objects entering the pump will pass through the impeller between the impeller vanes. If an object gets caught on the leading edge of one of the vanes, it will slide along the backswept shape towards the perimeter of the inlet where it will be guided by a relief groove through the pump housing. This ensures a high sustained total efficiency over time. Due to the mechanical self-cleaning design, a sludge concentration of solids up to 8% can easily be pumped. The pump can easily be installed in either permanently or temporary submerged, or horizontally or vertically dry installations.

Flexible and Modular Design

The modular hydraulic design enables customization of the hydraulics to meet the requirements of many applications.
- Replaceable wear ring in two materials, gray iron or Hard Iron, for different operation conditions
- Hardened gray iron impeller for typical wastewater applications
- Hard-iron impeller for heavy duty wastewater applications containing abrasive and corrosive content
- Chopper ring intended for tough wastewater applications where cutting is required due to long fibres and solid concentrations up to 10-12%
- Stainless steel impeller for special applications that require duplex stainless steel

Robust and Reliable

- Short shaft overhang reduces shaft deflection and increases seal and bearing life
- Class H Motor designed for submersible use. Heat is concentrated to the stator core for improved cooling properties.
- The Plug-in seal with Active Seal system eliminates the risk associated with incorrect installation and careless handling. All in one unit. Available in Tungsten carbide (WCCR) or Silicone carbide (SiC) depending on pumped media.
- Motor cable SUBCAB® specially developed for submersible use
- Offers flexible cooling systems, e.g. closed-loop cooling system, media cooled or external cooling that allows full motor potential in dry installations.
- Premium brand bearings, greased for life, ensures a minimum of 50 000 hours of duty
- Leakage sensor and motor temperature sensor as standard

The N 3202 is available with the following options
- ATEx, FM, CSA-approvals
- Premium efficiency motors
- Hard Iron hydraulic design
- Stainless Steel hydraulic design
- Vibration-sensor, extended motor temperature sensors, additional leakage-sensor, current-sensor and pump memory
- Compatible with SmartRun® Wastewater pump controller
- Compatible with MAS 801 monitoring system

Product Features

- State-of-the-art wastewater pump with N-technology
- Sustained high efficiency pumping with energy savings up to 25%
- Flexible and modular design
- Robust and reliable

Construction Materials

Impeller Material	Volute Material	Stator Cover Material
Grey Cast Iron	Grey Cast Iron	-

Motor

Rated Power	**Number Of Phases**	**Start Current Ratio**	**Motor Issue**
37 kW	3	5.69	11
Motor Denomination	**Rated Motor Speed**	**Insulation Class**	**Locked Rotor Code**
30-29-6AA	980 RPM	H	G
Motor Efficiency Class	**Rated Voltage**	**Approval**	**Max starts per hour**
Standard	400 V	Standard	30
Version Code	**Rated Current**	**Total moment of inertia**	**Power Factor 100%**
180	71 A	0.7737 kgm²	0.83
Frequency	**Start Current**	**Type of duty**	**Power Factor 75%**
50 Hz	405 A	S1	0.79
Max P2 (1x)	**Starting Current, Direct Starting**	**Stator Variant**	**Power Factor 50%**
36.06 kW	405 A	1	0.68
Number Of Poles	**Starting Current, Star Delta**	**Motor Module**	**Efficiency 100%**
6	135 A	170	90 %
			Efficiency 75%
			90.5 %
			Efficiency 50%
			90.5 %

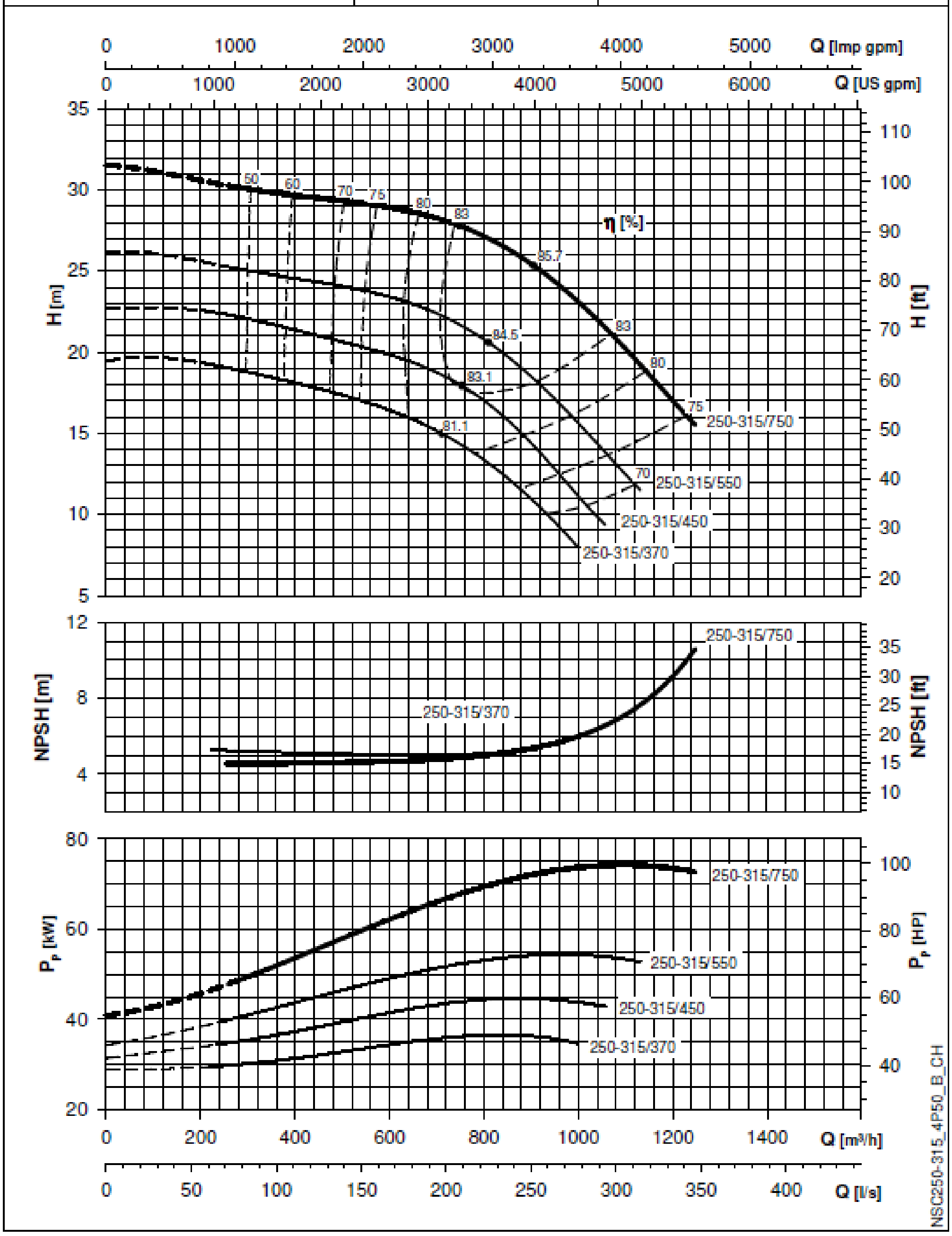
Project:	Created By:	Created On:	Last Update:
NP 3202 LT 3~ SmartRun 610	-	10/05/2025	-

Appendix B: LOWARA NSC 250-315 pump – Technical detail

LOWARA
a xylem brand

e-NSC SERIES
OPERATING CHARACTERISTICS AT 50 Hz, 4 POLES

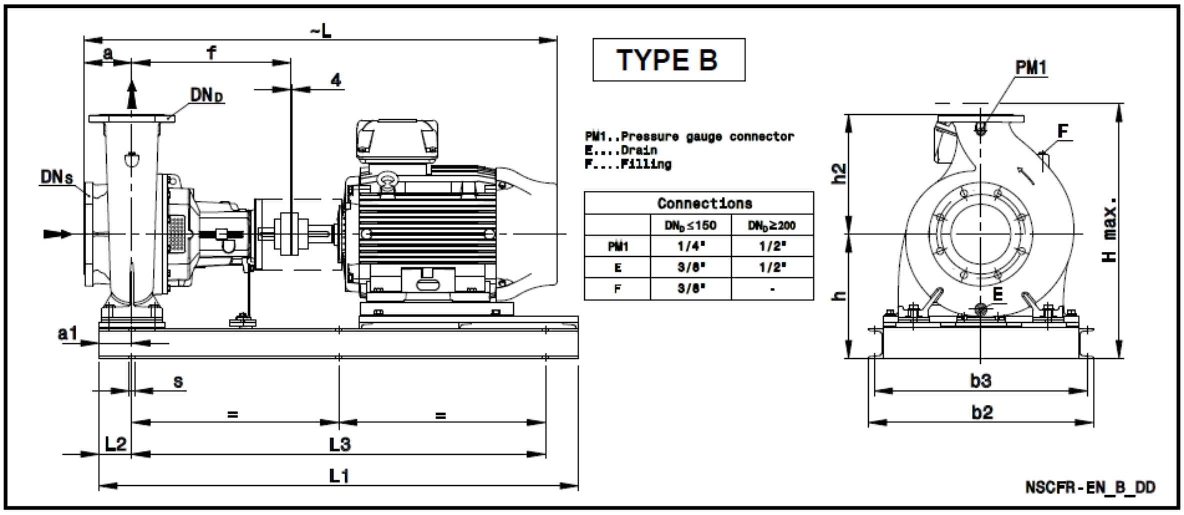
NSC 250-315	~ 1475 [rpm]	ISO 9906:2012 - Grade 3B



The NPSH values are laboratory values; for practical use we suggest increasing these values by 0,5 m.
These performances are valid for liquids with density ρ = 1,0 Kg/dm³ and kinematic viscosity ν = 1 mm²/sec.

LOWARA
a xylem brand

NSCF 150 SERIES (MOUNTED ON BASE)
DIMENSIONS AND WEIGHTS AT 50 Hz, 4 POLES



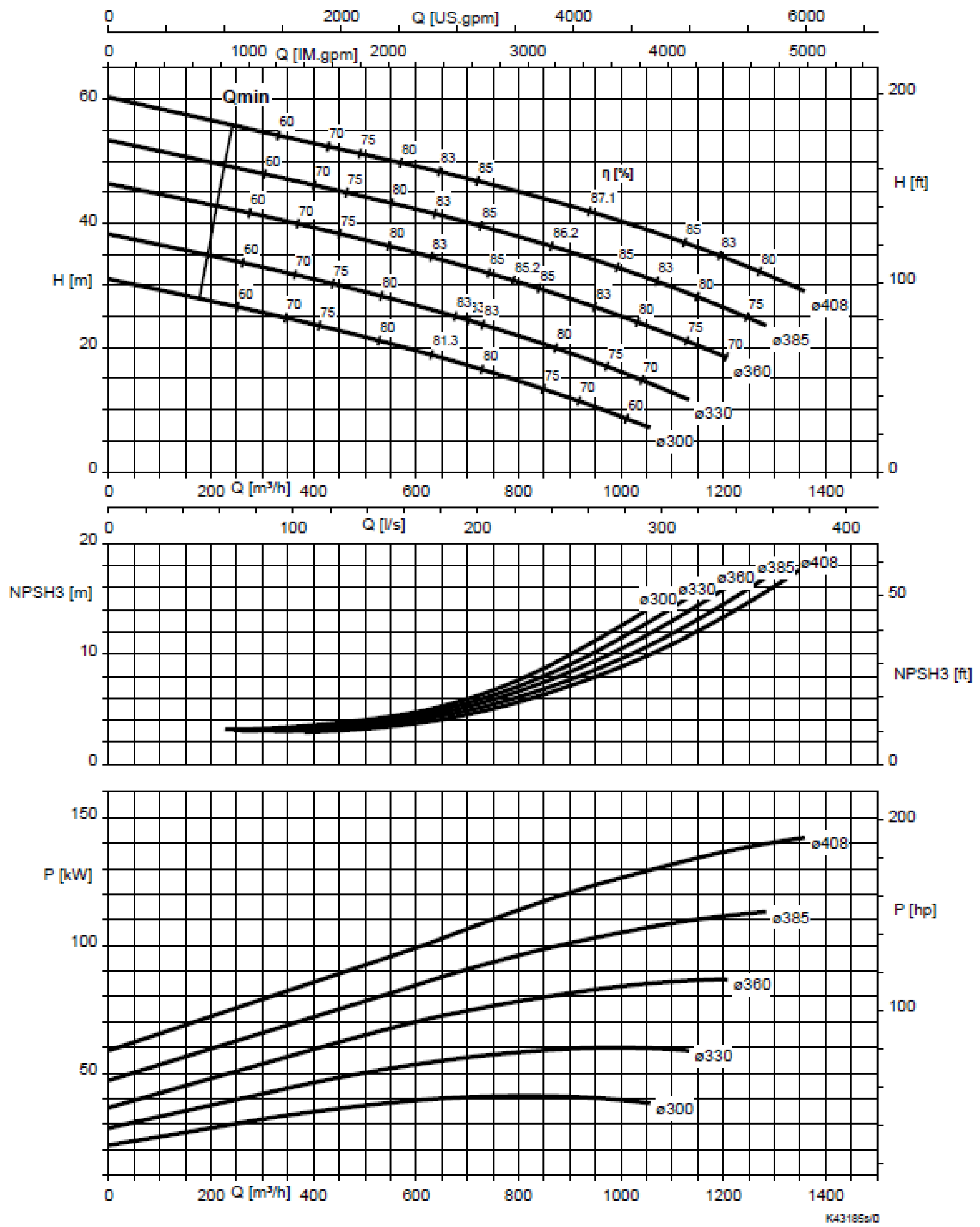
PUMP TYPE NSCF..4	TYPE	DIMENSIONS (mm) DNS	DND	a	a1	b2	b3	f	h	h2	L	L1	L2	L3	H max	s FOR SCREWS	WEIGHT (kg) G	COUPLING TYPE
250-315/370/W	B	300	250	250	165	850	810	530	525	500	1670	1700	165	1370	1025	6xØ19 (M16)	905	B140B

Appendix C: KSB KRT K 200-402 pump – Technical detail

Waste Water
Submersible Motor Pump

Amarex KRT K 200-402, n = 1450 rpm

Characteristic curves to ISO 9906 Class 3B, below 10 kW to § 4.4.2. n = nominal speed



Free passage = 80 mm

Waste Water
Submersible Motor Pump

General arrangement drawing S5, stationary on duckfoot bend, guide rail arrangement, single-level foundation, without foundation rail, small upper holder, motor version N

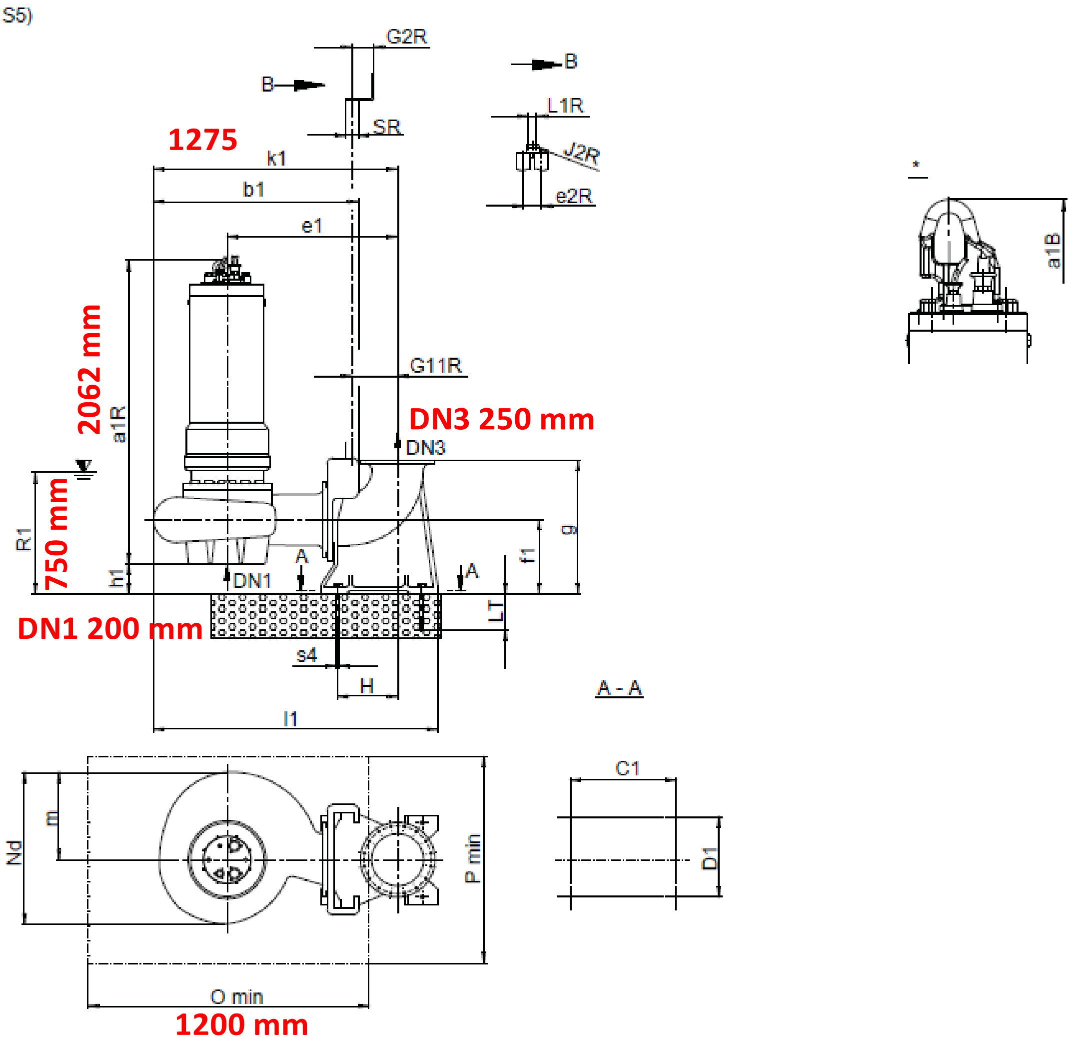


Fig. 6: General arrangement drawing S5, stationary on duckfoot bend, guide rail arrangement, single-level foundation, without foundation rail, small upper holder, motor version N

*: Optional

Size	Impeller type 50 Hz	Impeller type 60 Hz	Motor	Installation types K/S with guide wire arrangement Figure see page	Installation types K/S with guide rail arrangement Figure see page	DN1	DN3	Nominal diameter of bend	b1 [mm]	e1 [mm]	a1B [mm]	f1 [mm]	g [mm]	h1 [mm]	k1 [mm]	l1 [mm]
200-402	K	K	130 4	24	14	200	250	200/250	1120	955	2062	400	745	216	1275	1478

Size	R1 (Motor version UN/UE/WN/WE/XN/XE/YN/YE) [mm]	R1 (Motor version ZN/ZE) [mm]	E12 [mm]	H [mm]	W1 [mm]	LT [mm]	s4 [mm]	A11 [mm]	E21 [mm]	m [mm]	Nd [mm]	C1 [mm]	D1 [mm]	C11 [mm]	C2 [mm]	O min [mm]	P min [mm]
200-402	750	750	-	350	-	125	18	-	-	410	735	490	300	450	-	1200	800

Appendix D: Jet-pump performance curves

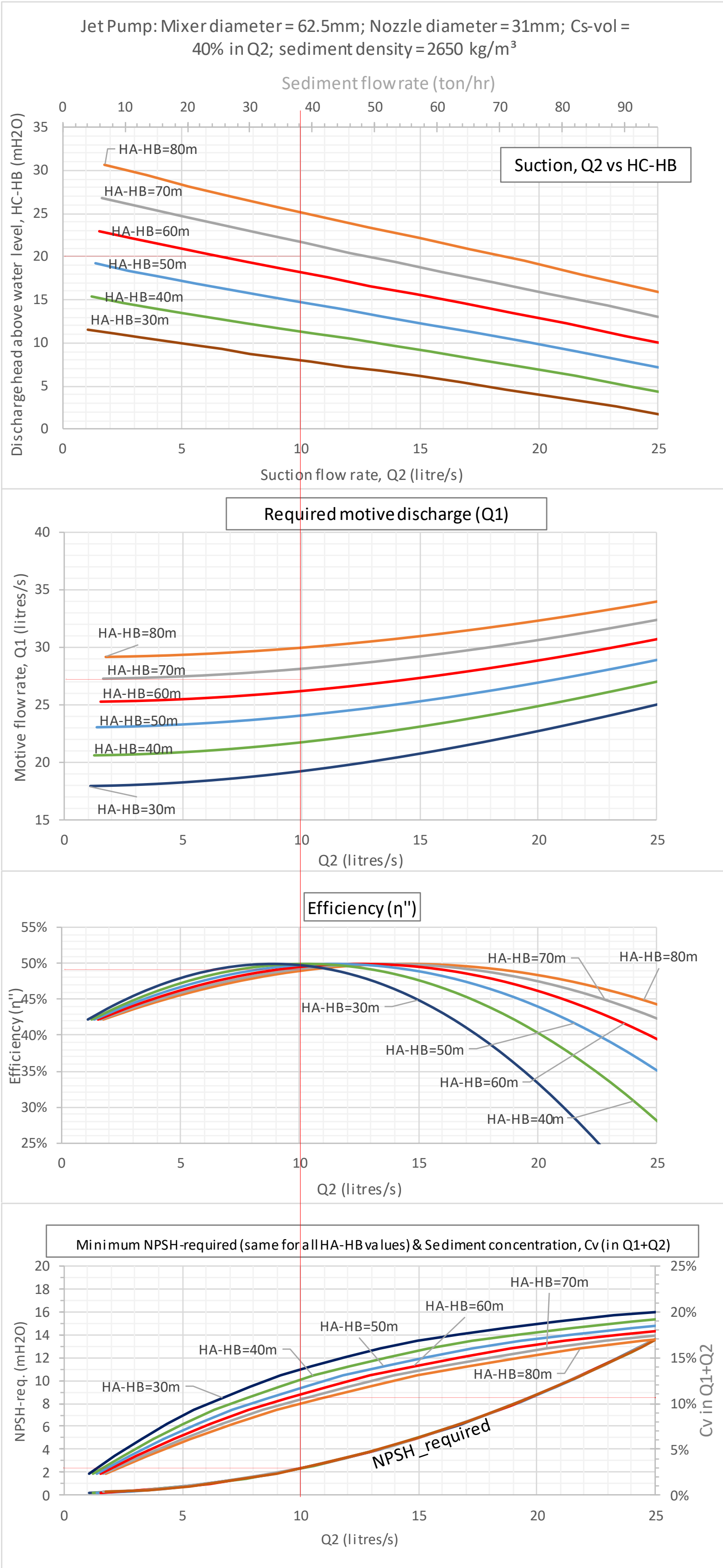


Figure D1: Performance curves of jet-pump with 62.5 mm diameter mixer and 31 mm diameter main nozzle (refer to Figure C2 for declaration of symbols)

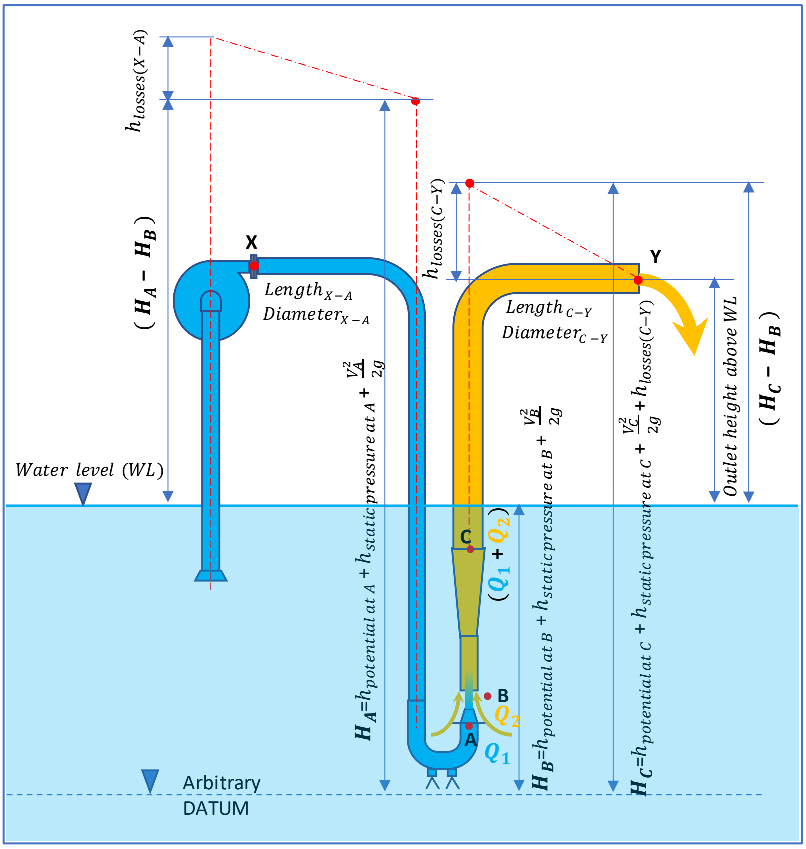
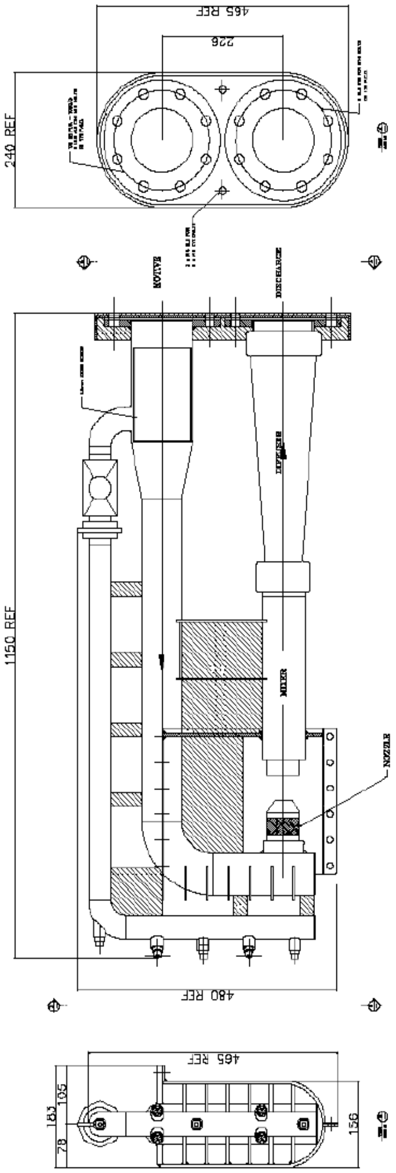


Figure D2: Declaration of symbols used in Figure C1.

Appendix E: Jet-pump examples

Example of GENFLO SANDBUG jet-pump



Half cowl removed with main jet and mixer visible

Appendix F: Mobile Jet-pump in action during WRC field evaluation research project (WRC, 2002)

EXAMPLE OF MOBILE JET PUMP REMOVING SAND FROM A RIVER INTAKE PUMPSTATION

Appendix G: Alternative arrangement for a permanently installed jet-pump in a hopper of as pump forebay

Possible alternative of using rubber supply and discharge pipes for jet pump instead of rigid supply and discharge pipes

OPTION FOR INSTALLATION AND RETRIEVAL CONFIGURATION OF DREDGED TYPE JET PUMP IN HOPPER OF RIVER INTAKE PUMP STATION EMPLOYING RUBBER HOSE PIPES

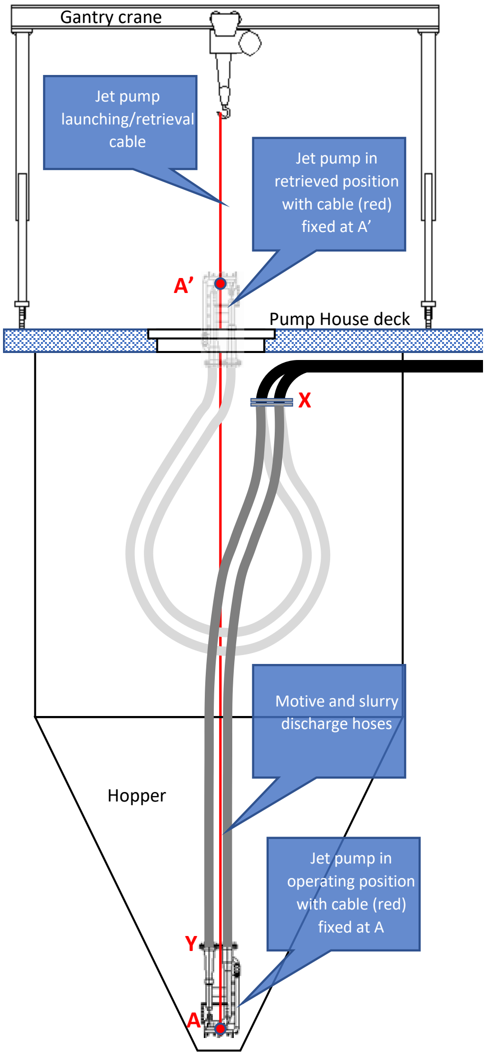


Figure 1: Schematic sketch of installed jet pump (dark grey) in hopper and retrieved for maintenance (light grey) with dredge type hoses between X and Y.

Description of rubber hoses

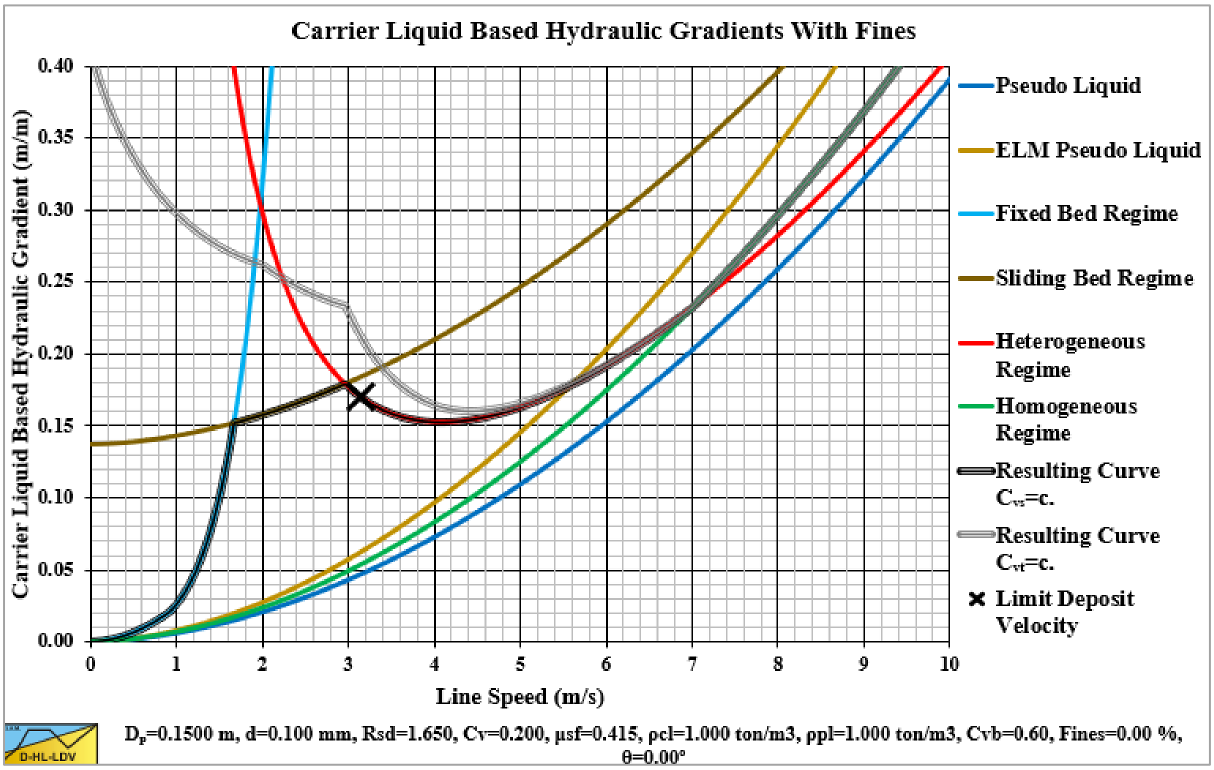
As an alternative to steel pipe for the vertical pipes in the hopper compartment of the intake pump station, dredge type rubber hoses could be considered. This will enable easier retrieval of the jet pump when maintenance on it is required. Also, the jet pump can be activated before reaching the invert of the hopper during launching so that it could dredge itself into position at the hopper invert. Figure 1 demonstrates the installed position of the jet pump on the invert of the hopper (with rubber hose indicated in dark grey) and at its retrieved position at the pump house operating deck (in light grey).

A suitable rubber hose is the dredge type rubber hoses with a LINATEX weir resistance lining as manufactured by WEIR (refer Figure 2 - https://www.global.weir/industries/mining/), or similar type of dredge hose.

The hose flanges should be of swivel flange ends enabling rotation of the backing flange for ease of alignment of the bolt holes as shown in Figure 2. The flange ends should be of the type that can resist significant *axial* stresses to ensure robustness and should therefore be appropriately integrated with the hose material.

Figure 2: Swivel flange ends of LINATEX dredge type hose manufactured by WEIR (https://www.global.weir/industries/mining/)

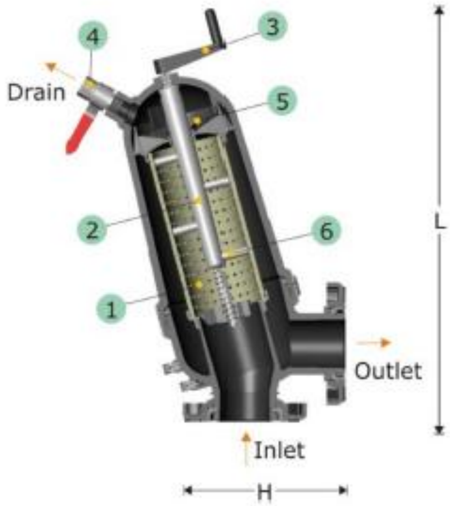
Appendix H: Head loss and sediment limit deposit velocity graph for jet-pump slurry pipe (Miedema et al, 2016)



Appendix I: Example of a screen filter for jet-pump motive water

Operation Description - Filtration Mode

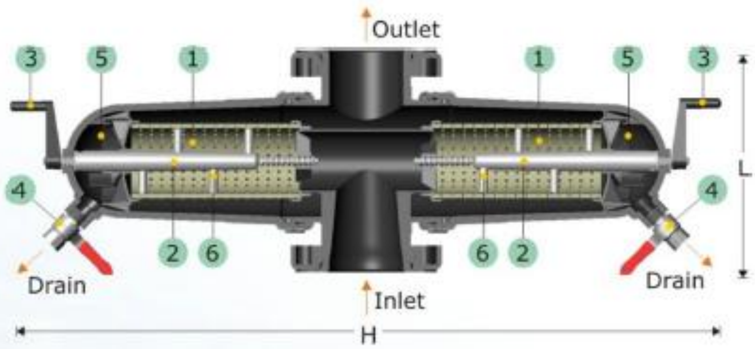
Water enters the filter through the "Inlet", reaching the screen 1, which purifies the water by taking out smaller particles. As more water flows through impurities build up on the screen creating differential pressure between the internal section of the screen 1 and the external section of the screen.



Description

Arkal's 3", 4" and 6" Semi-Automatic filters consist of the following major parts:

1 Screen
2 Dirt Collector
3 Handle
4 Flush Valve
5 Flush Chamber
6 Nozzles



Cleaning Process in Suction Type Filter

When the differential pressure (ΔP) reaches the determined value (no more than 0.8 bar), or according to the timetable specified by the operator, a series of events occurs as the water continues to flow into the system's units.

Before opening the flush valve 4 the operator should check that the dirt collector 2 is fully withdrawn by turning the handle 3 clockwise until it stops. When the flush valve 4 opens, water flows out.

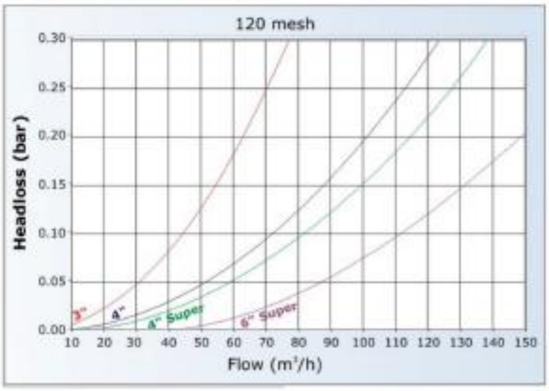
The operator should also check that the pressure in the flush chamber 5 and the dirt collector 2 is significantly reduced, which will create suction through the collector's nozzles 6. Rotating the dirt collector 2 by the handle will cause full scanning of the screen 1 by the nozzles 6. The combination of linear movement and rotation cleans the whole internal screen 1 surface. The flushing cycle will take only a few seconds.

General Technical Data

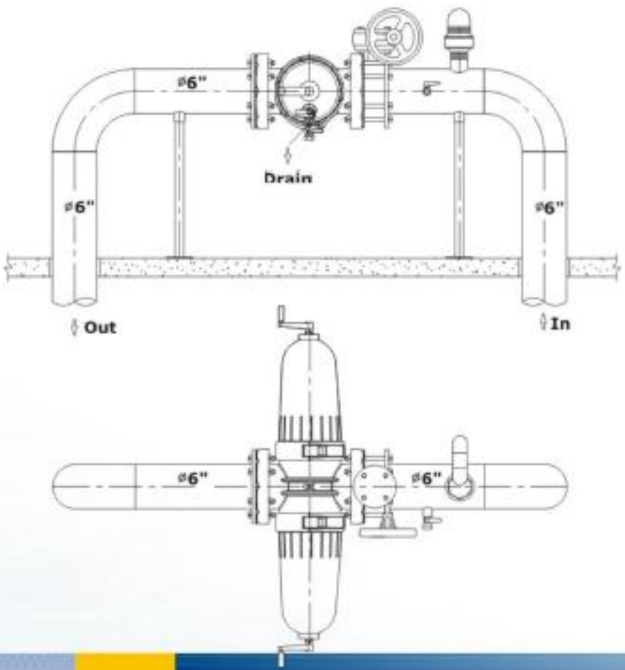
- Max. Pressure: 10 bar / 145 psi
- Max. Temperature: 60°C / 140°F
- Filtration Grades (currently): 400µ, 200µ, 120µ

Model Number	Connection Size (inch)	Screen Area (cm^2)	Max. Flow Rate (M^3/h)	L (mm)	H (mm)	Weight (kg)
AKSP3LT	3	1250	60	825	509	12
AKSP3LV	3	1250	60	789	509	12
AKSP3LF	3	1250	60	789	509	13
AKSP4LV	4	1250	90	789	509	13
AKSP4LF	4	1250	90	789	509	14
AKSP4S	4	2500	110	445	1368	26
AKSP6S	6	2500	140	415	1368	28

Headloss Chart



AKSP = Arkal Semi Automatic Polypropylene
L = Angel filter connection
T = Threaded filter connection
V = Victaulic filter connection
F = Flanged filter connection
S = Super leader filter (inline filter connection)



Appendix J: Proposed motive pump to drive jet-pumps

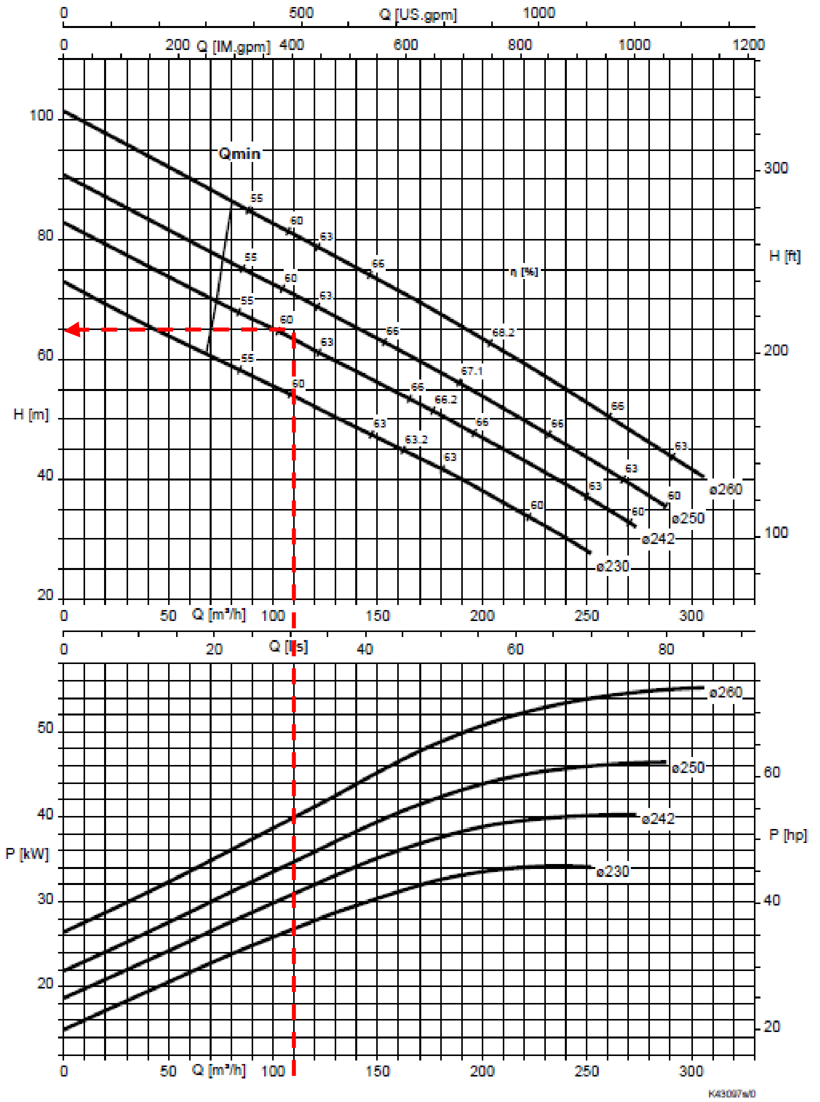
KSB

Waste Water
Submersible Motor Pump

D impeller

Amarex KRT D 80-315, n = 2900 rpm

Characteristic curves to ISO 9906 Class 3B, below 10 kW to § 4.4.2. n = nominal speed



Free passage = 65 mm

KSB

Waste Water
Submersible Motor Pump

General arrangement drawing S14, stationary on duckfoot bend, guide rail arrangement, foundation with step, without foundation rail, motor version E

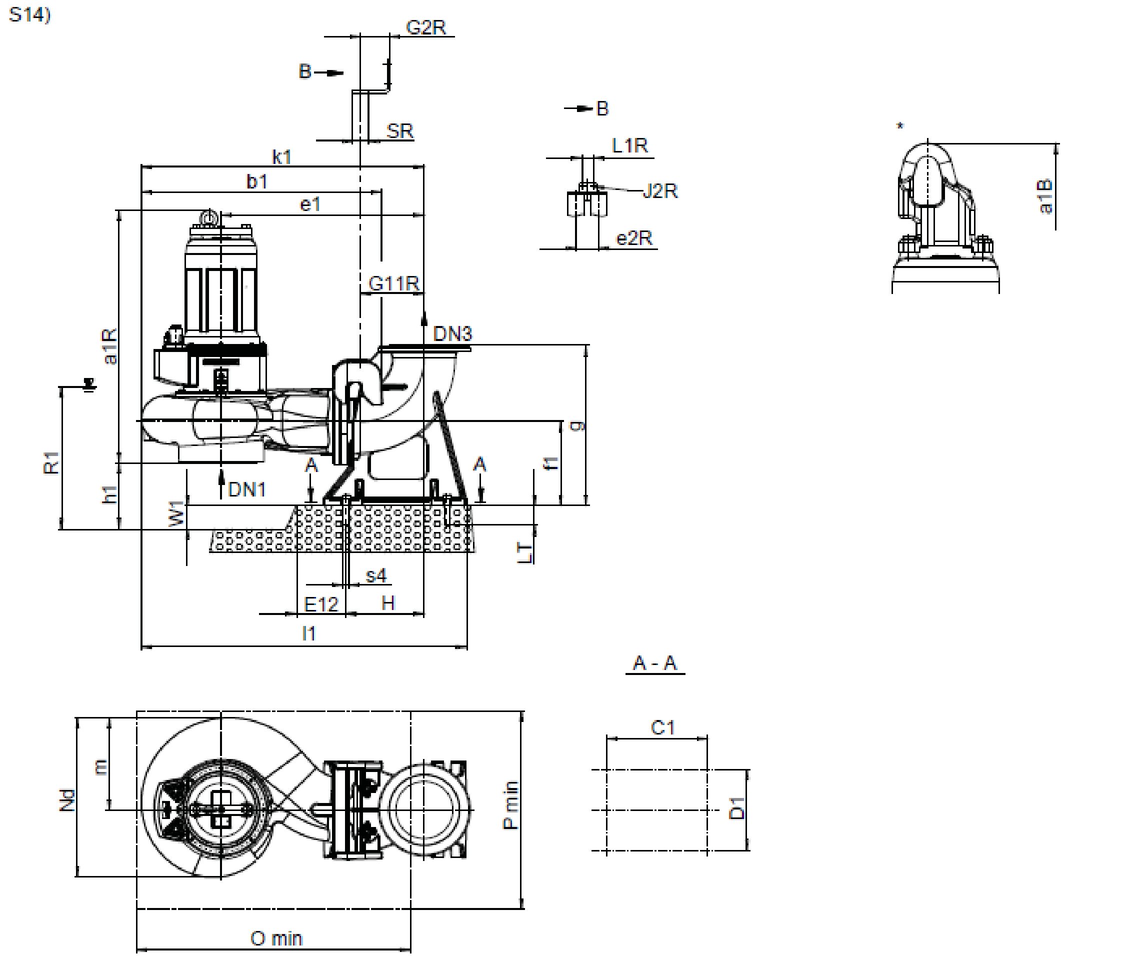


Fig. 9: General arrangement drawing S14, stationary on duckfoot bend, guide rail arrangement, foundation with step, without foundation rail, motor version E

*: Optional

080-315, installation types K/S

080-315, installation types K/S, dimensions and weights depending on the material variant, part 1

Size	Impeller type 50 Hz	Impeller type 60 Hz	Motor	Installation types K/S with guide wire arrangement Figure see page	Installation types K/S with guide rail arrangement Figure see page	DN1	DN3	Nominal diameter of bend	b1 [mm]	e1 [mm]	a1B [mm]	f1 [mm]	g [mm]	h1 [mm]	k1 [mm]	l1 [mm]	R1 (Motor version UN/UE/WN/WE/XN/XE/YN/YE) [mm]	R1 (Motor version ZN/ZE) [mm]	E12 [mm]	H [mm]	W1 [mm]	LT [mm]	s4 [mm]	A11 [mm]	E21 [mm]	m [mm]	Nd [mm]	C1 [mm]	D1 [mm]	C11 [mm]	C2 [mm]	O min [mm]	P min [mm]
080-315	D	-	55 2	27	17	100	100	100/100	771	530	1633	240	430	102	885	1011	590	765	200	139	80	125	18	-	-	203	383	220	150	-	-	900	500

Appendix K: 2D Hydrodynamic modelling results

2-year flood scenario

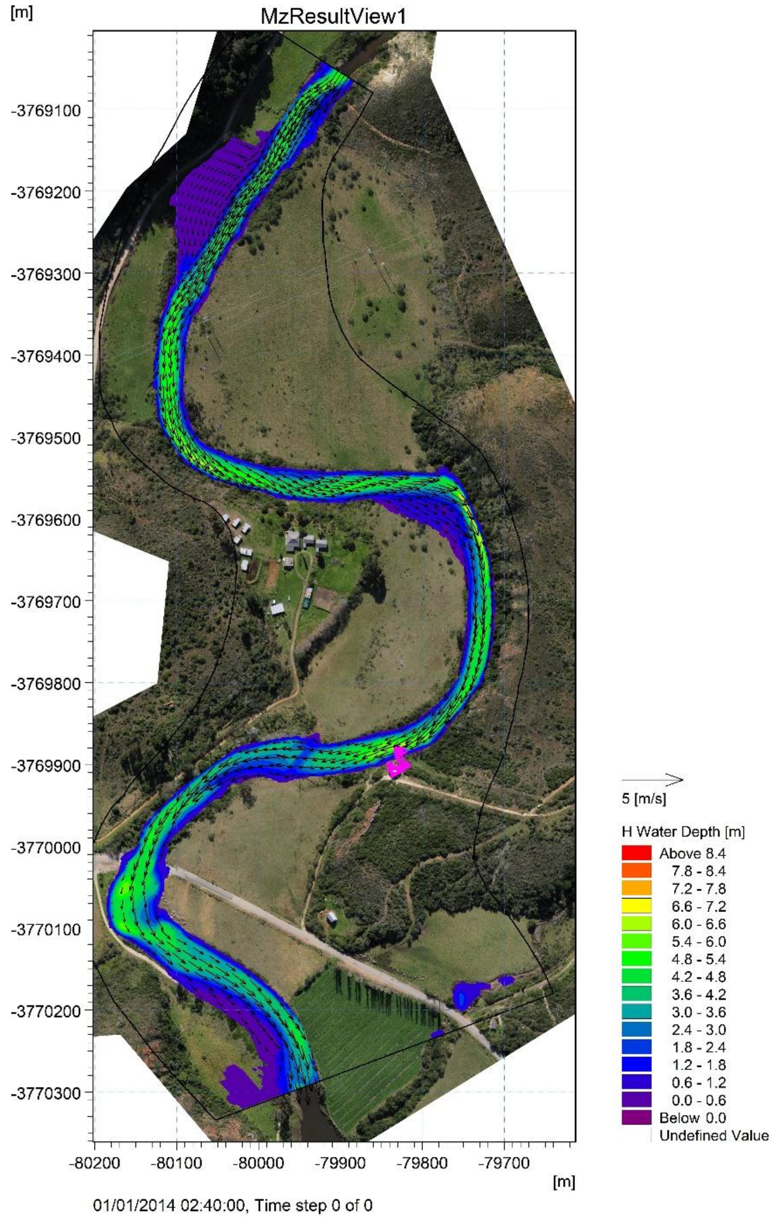


Figure J-1: Simulated flow depths during 2-year flood peak

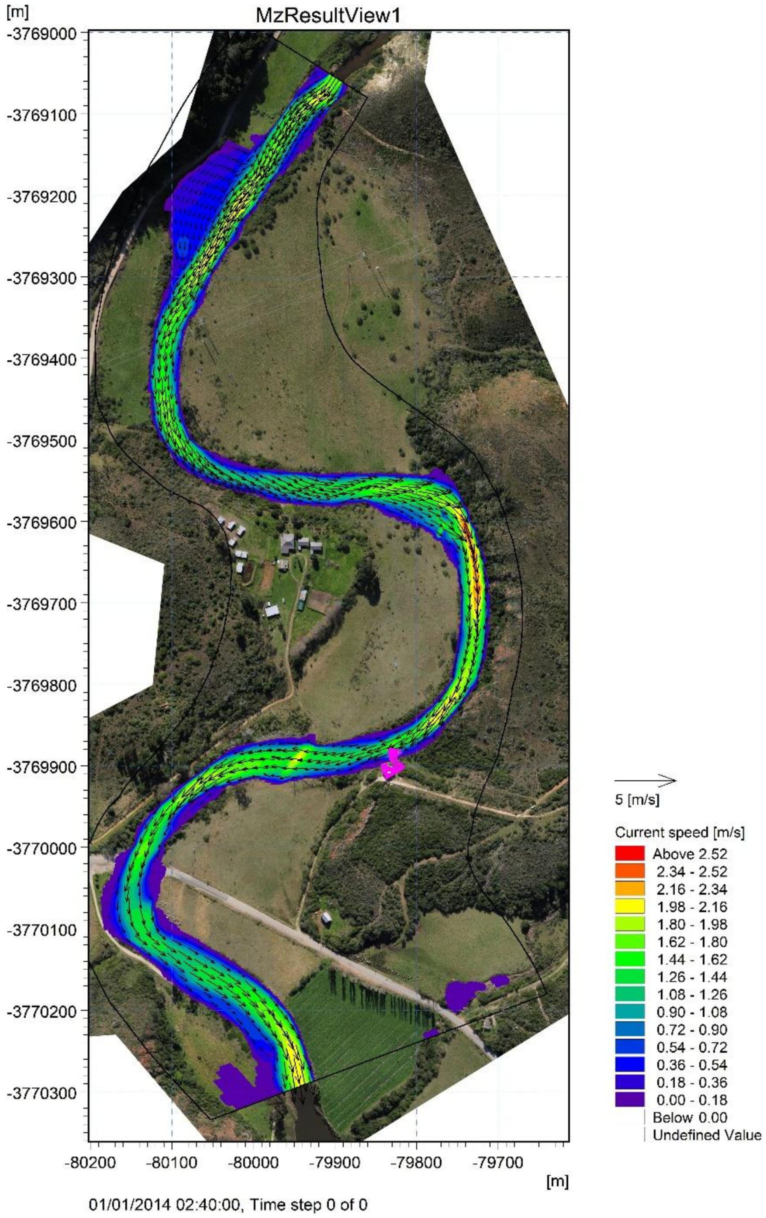


Figure J-2: Simulated flow velocities during 2-year flood peak

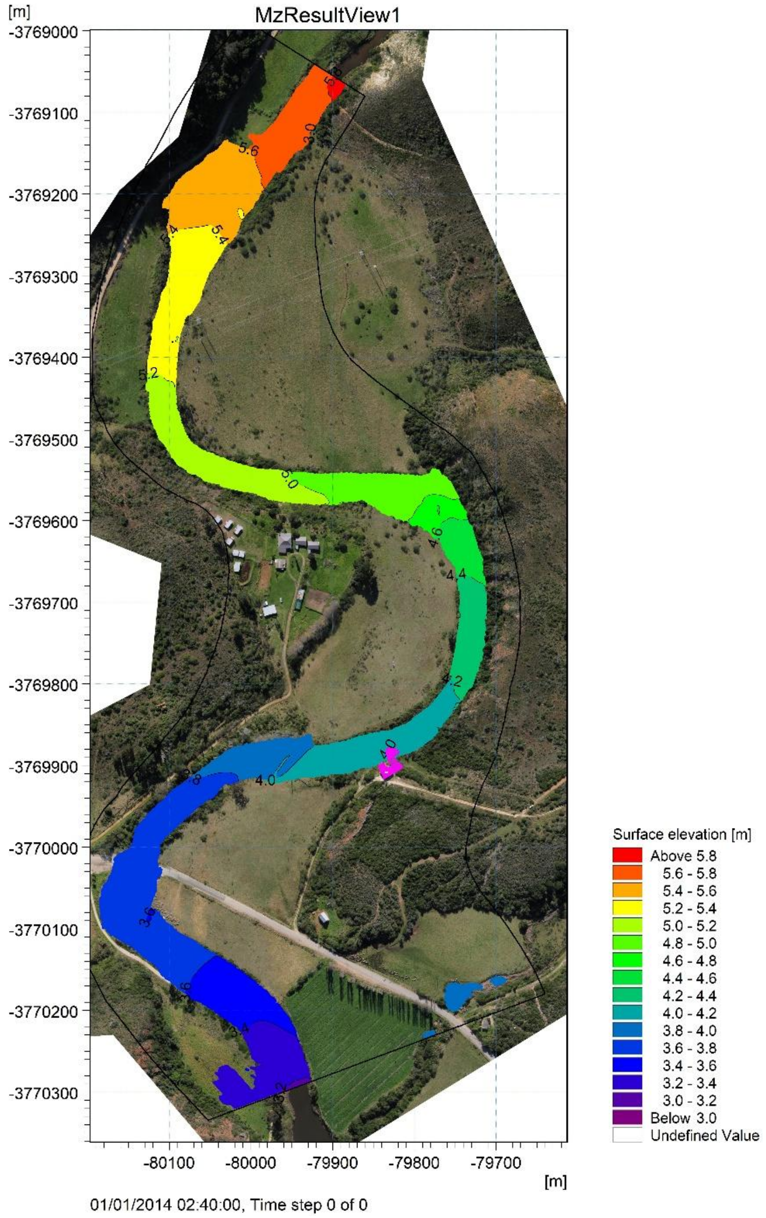


Figure J-3: Simulated water levels during 2-year flood peak

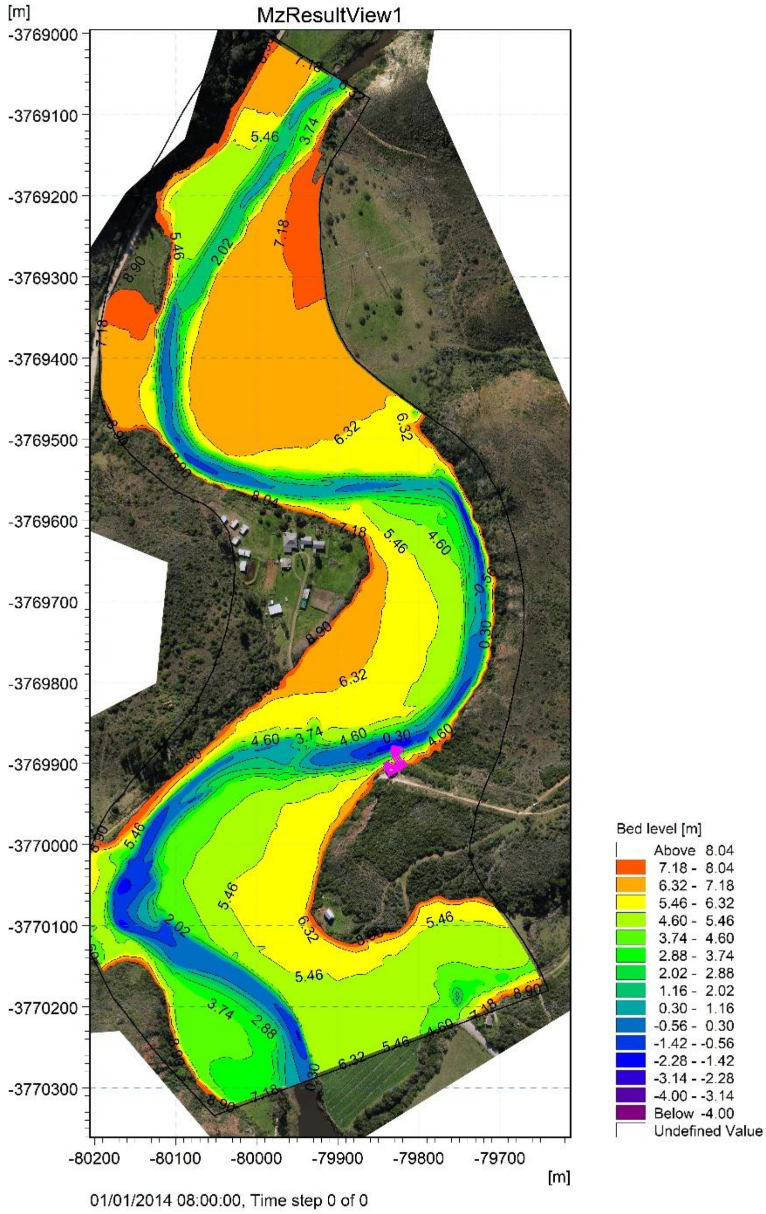


Figure J-4: Simulated bed levels at the end of 2-year flood peak

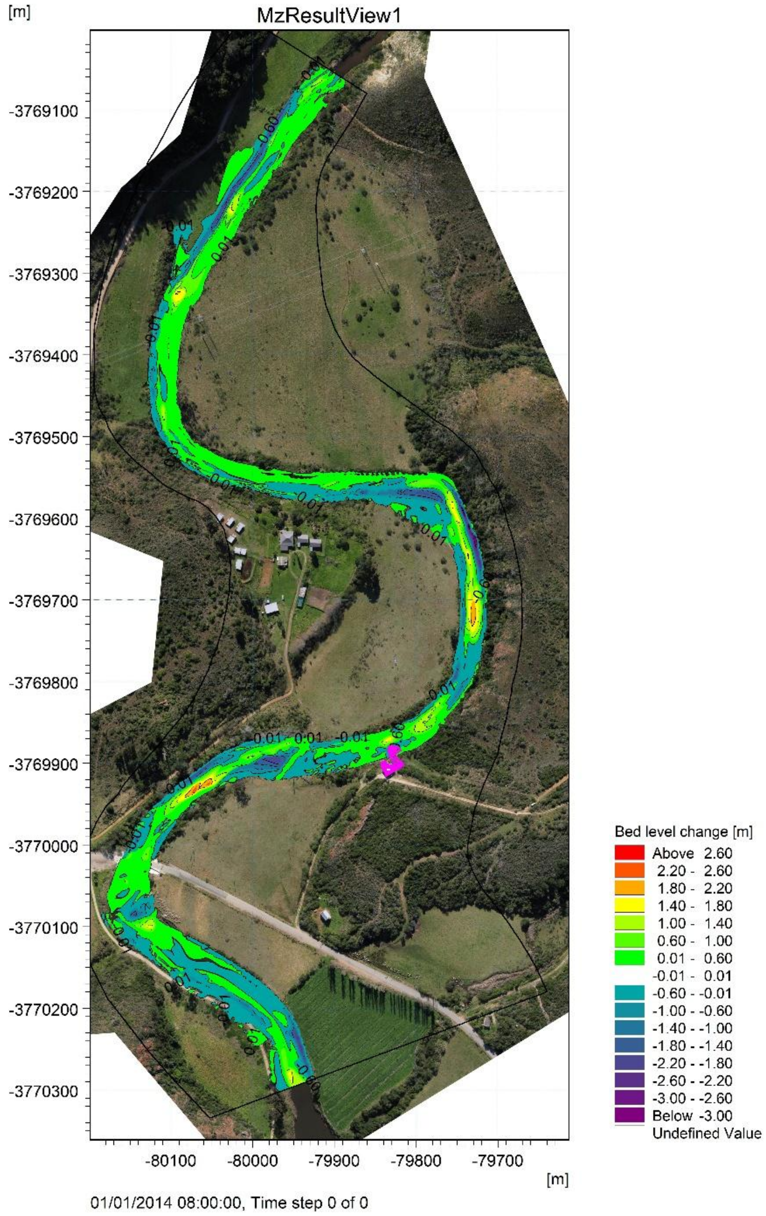


Figure J-5: Simulated bed level change at the end of 2-year flood peak

10-year flood scenario

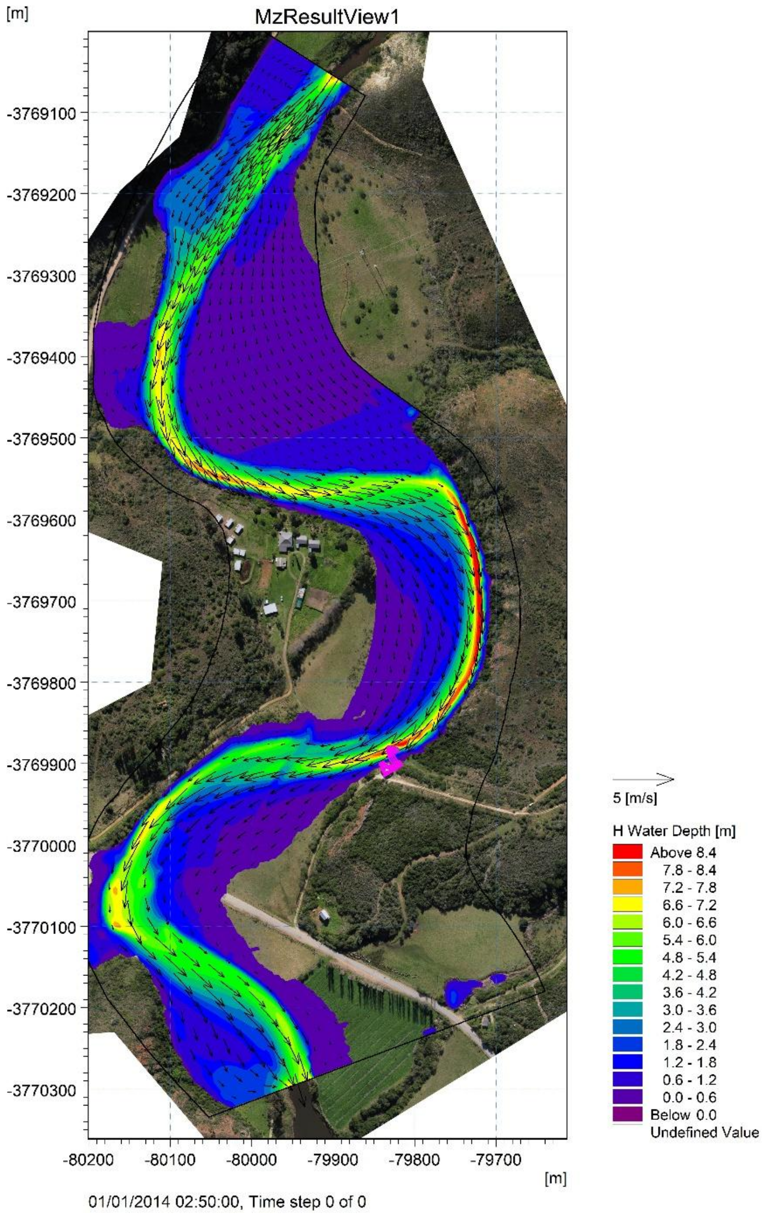


Figure J-6: Simulated flow depths during 10-year flood peak

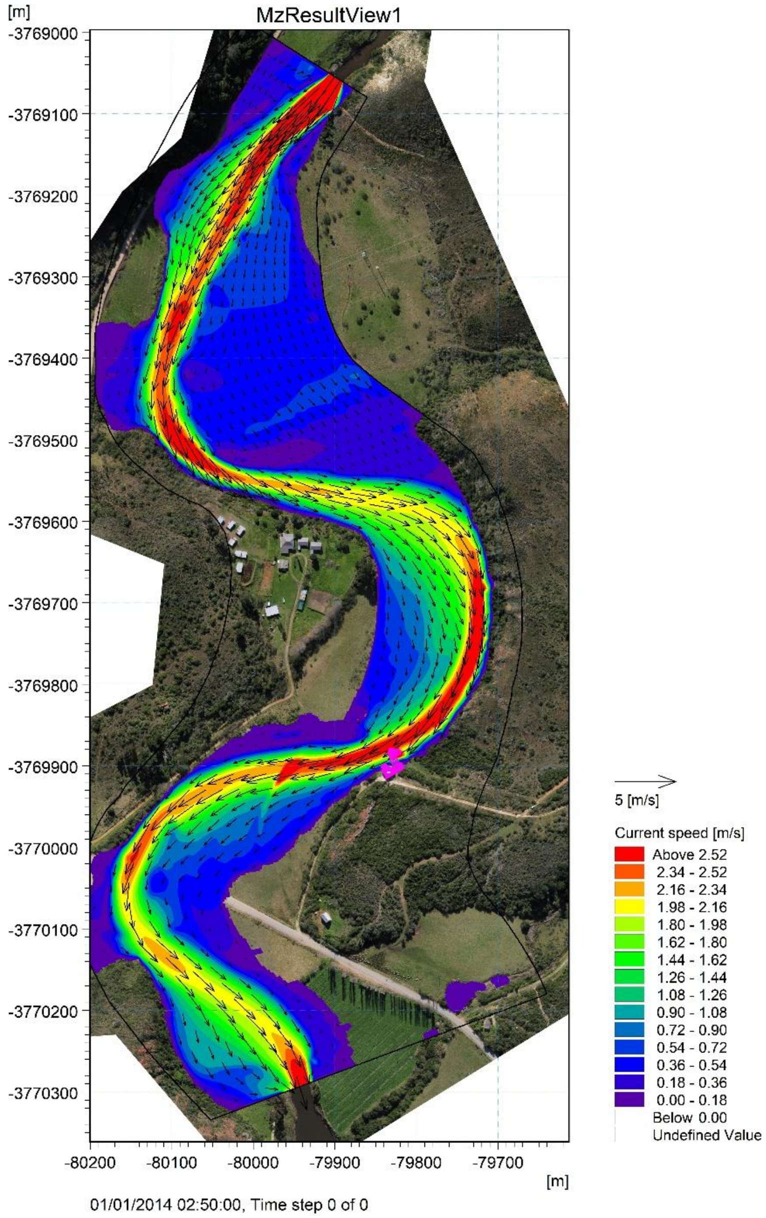


Figure J-7: Simulated flow velocities during 10-year flood peak

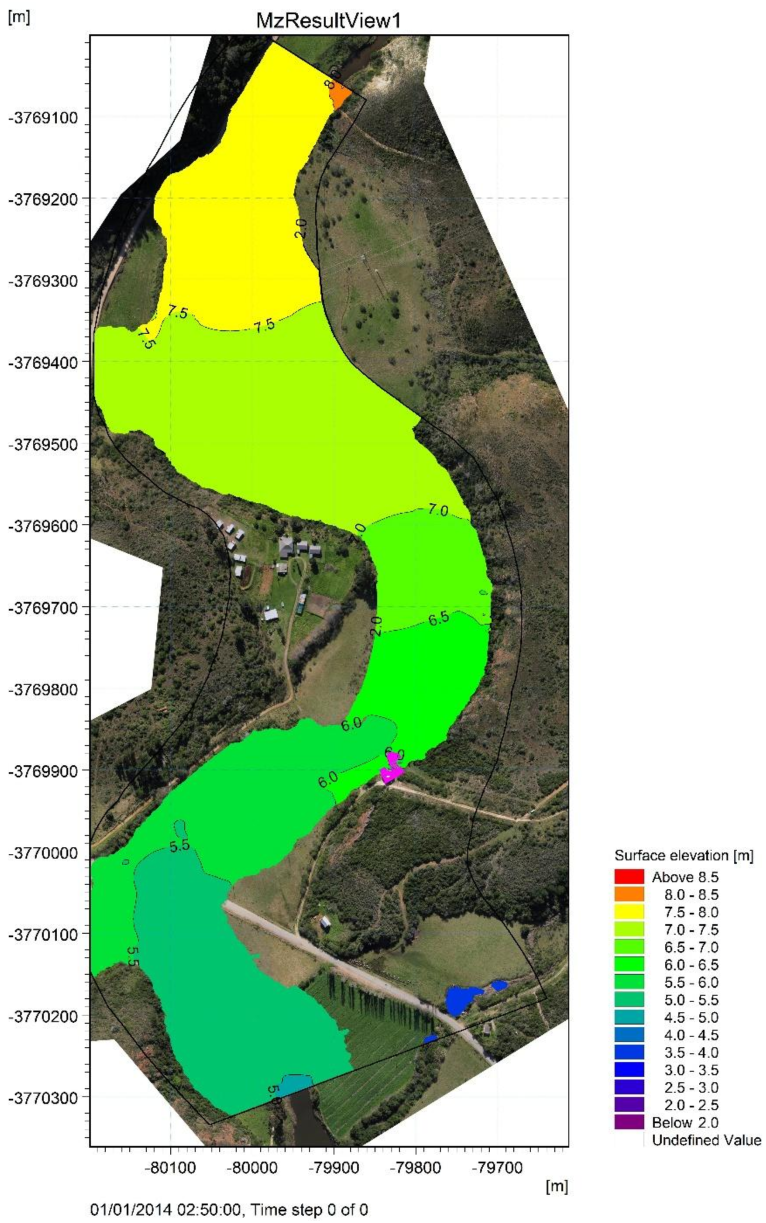


Figure J-8: Simulated water levels during 10-year flood peak

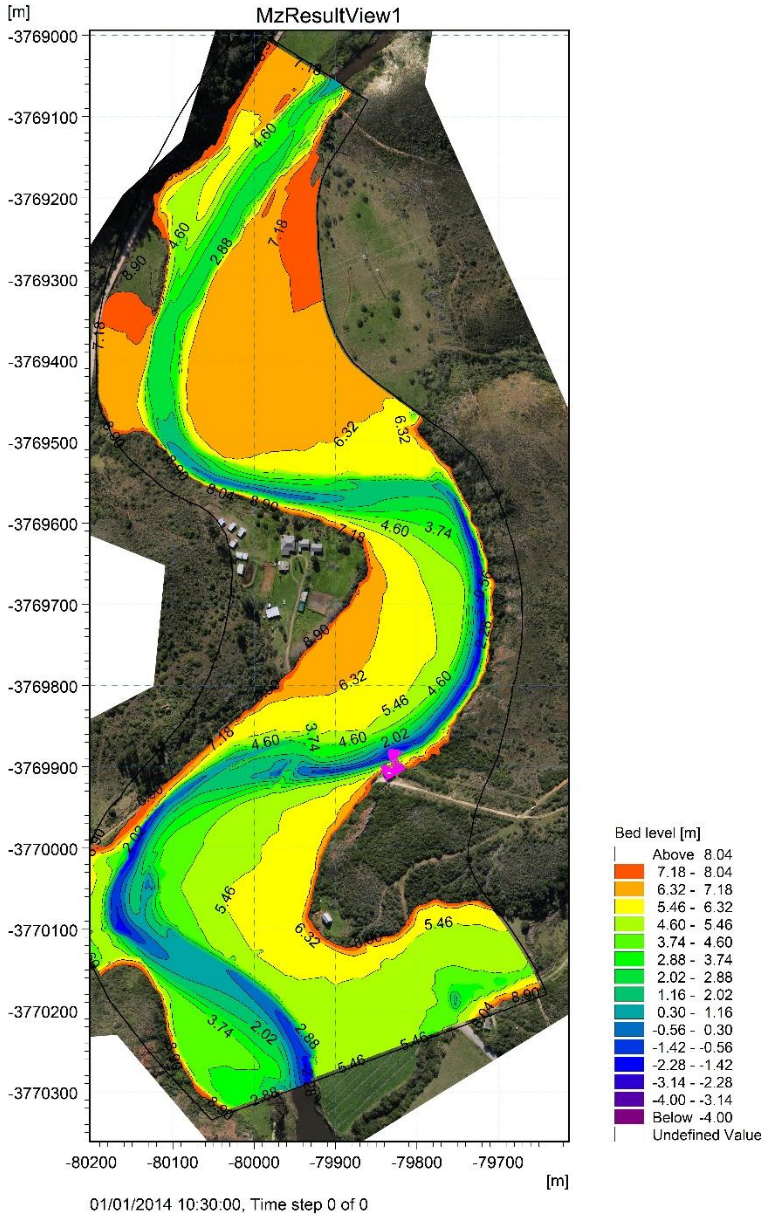


Figure J-9: Simulated bed levels at the end of 10-year flood peak

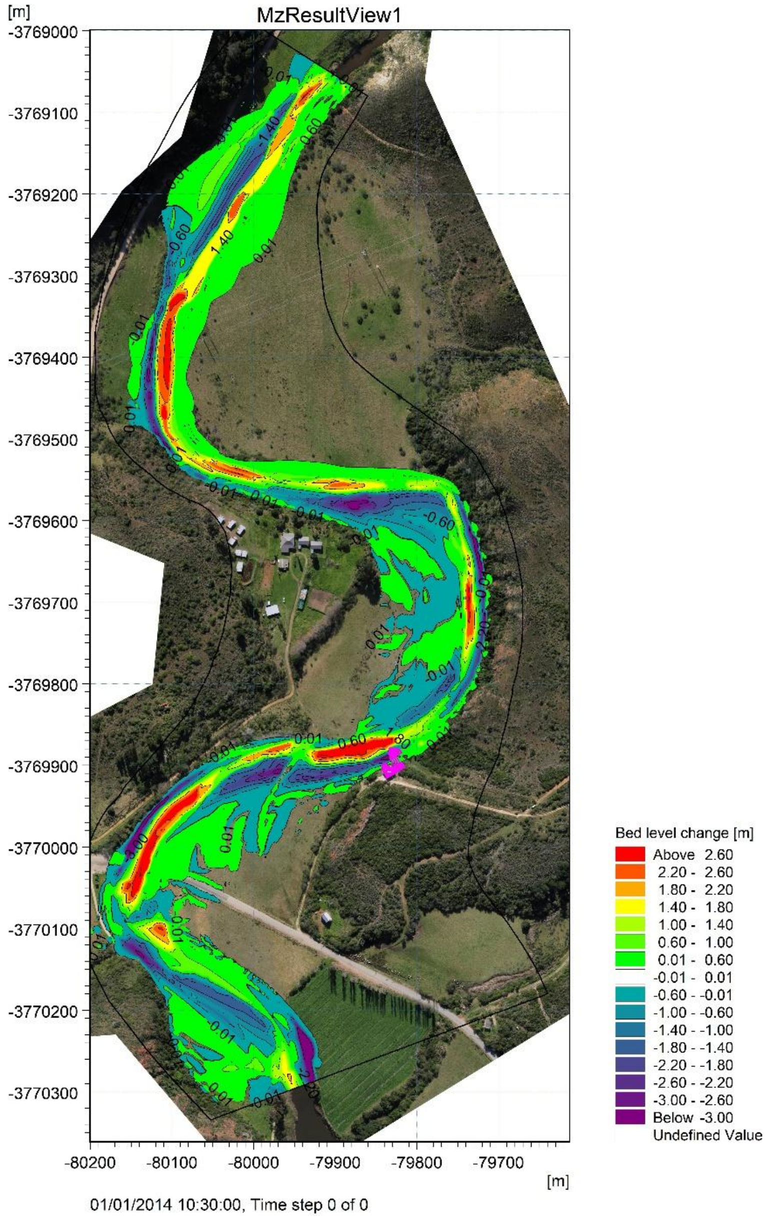


Figure J-10: Simulated bed level change at the end of 10-year flood peak

100-year flood scenario

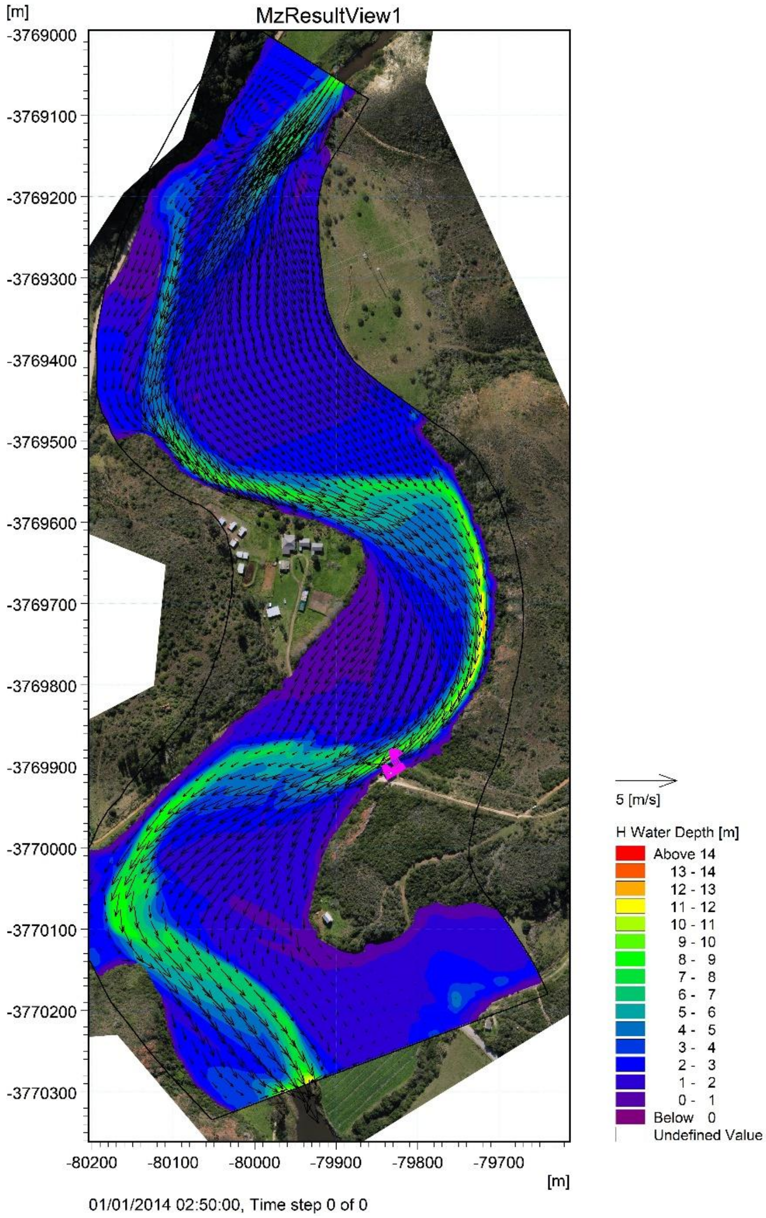


Figure J-11: Simulated flow depths during 100-year flood peak

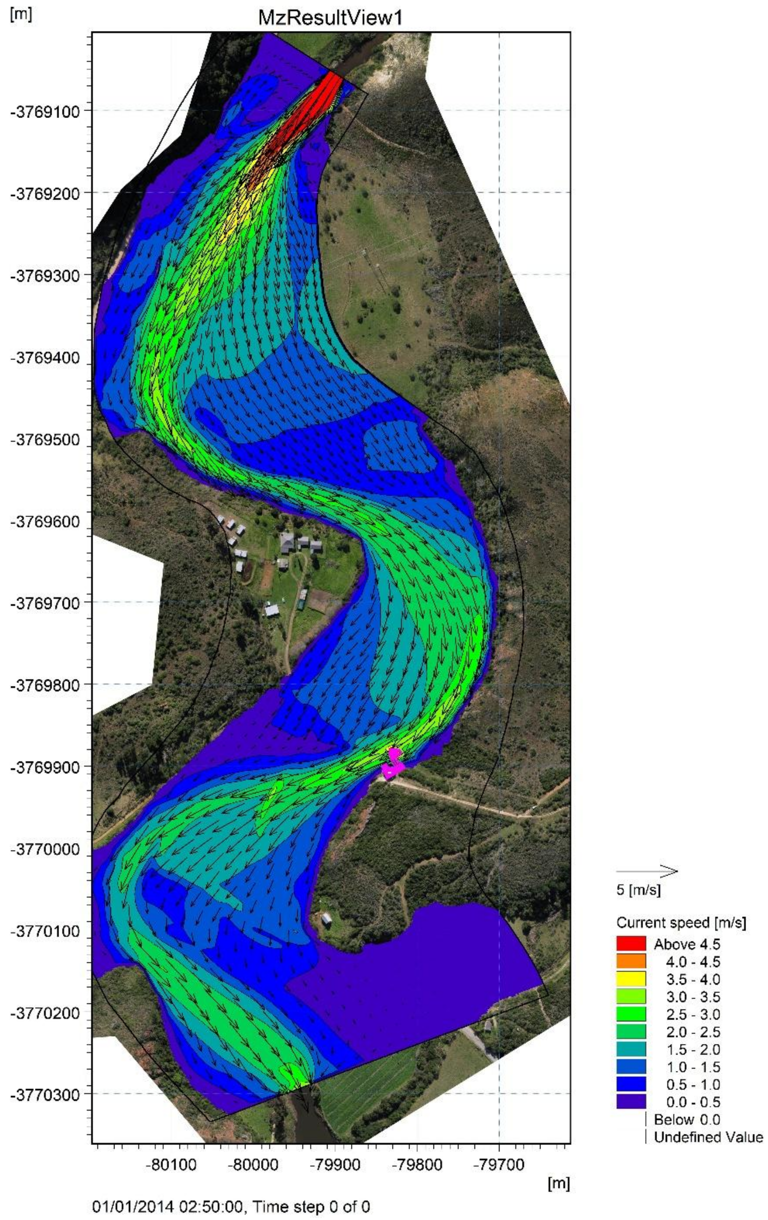


Figure J-12: Simulated flow velocities during the 100-year flood peak

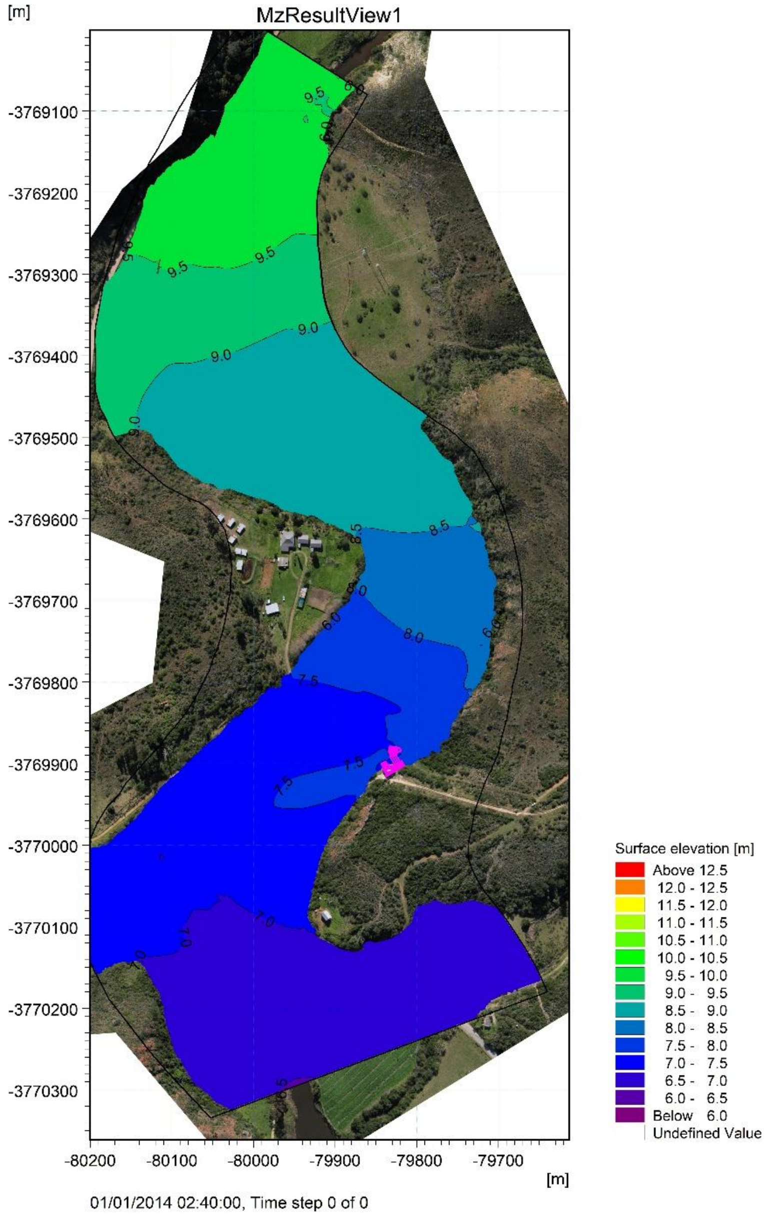


Figure J-13: Simulated water levels during the 100-year flood peak

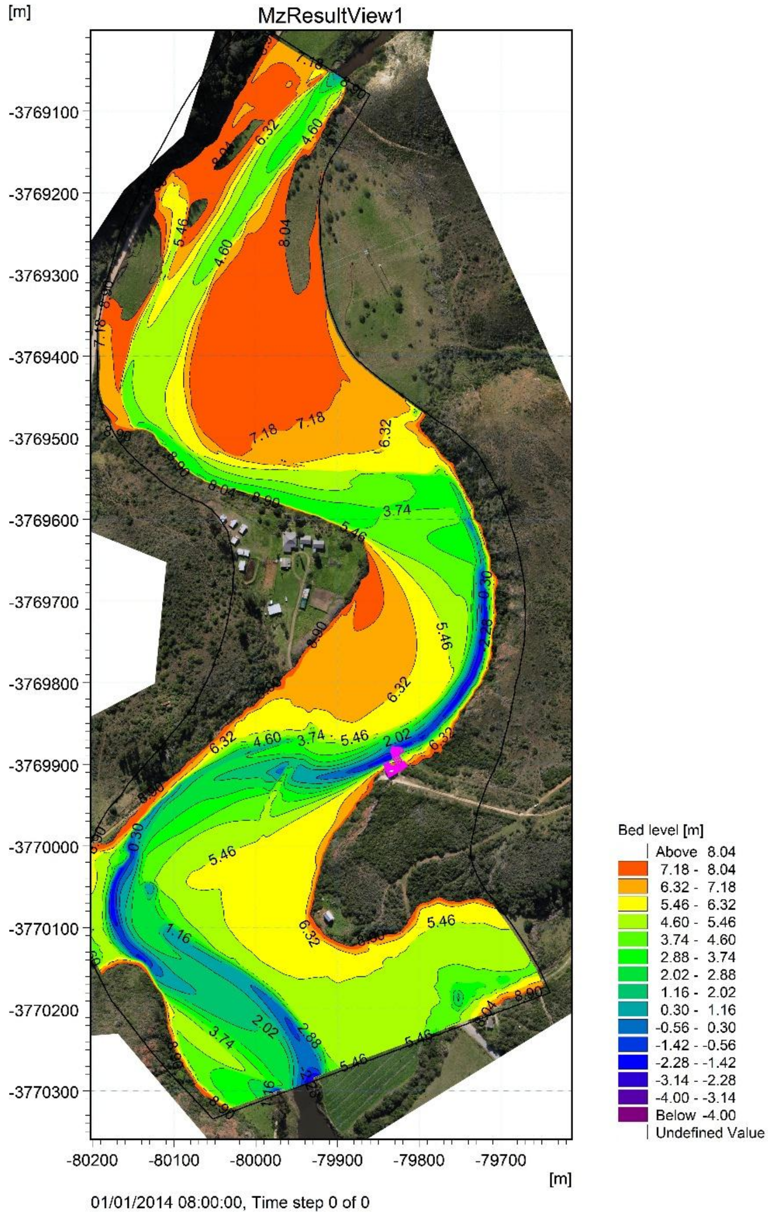


Figure J-14: Simulated bed levels at the end of 100-year flood peak

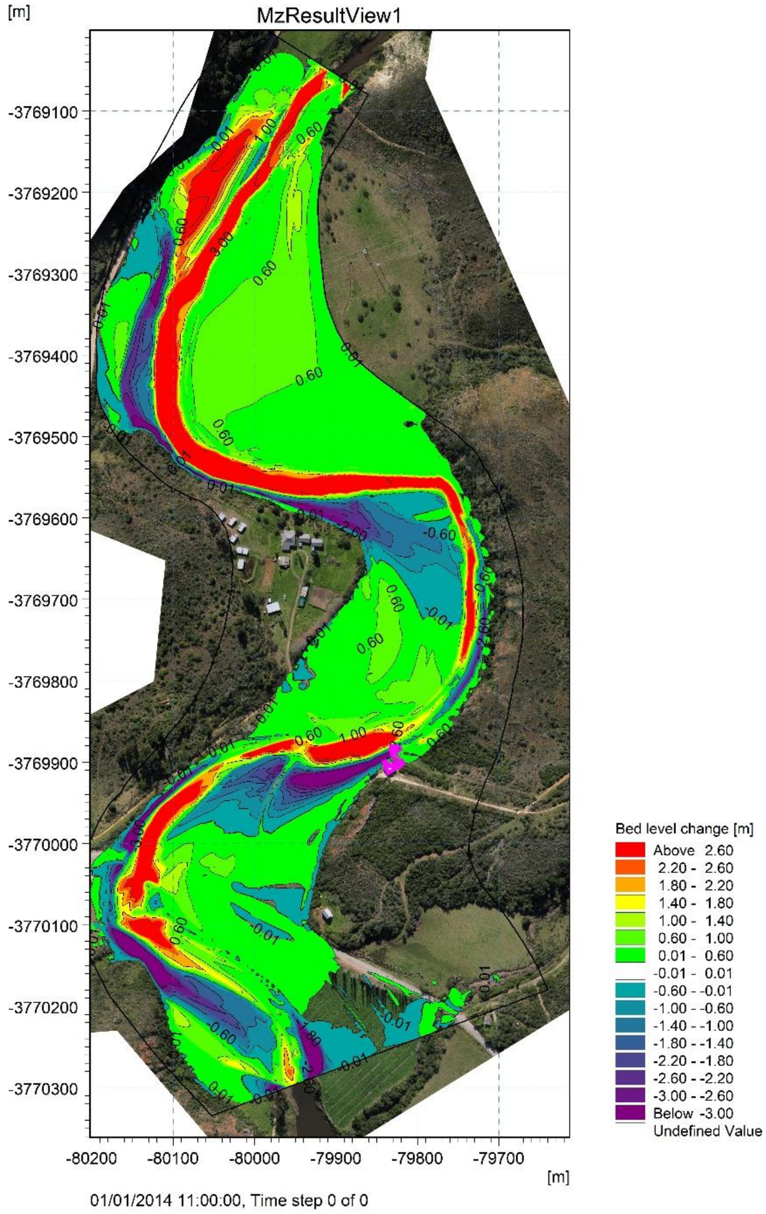


Figure J-15: Simulated bed level change at the end of 100-year flood peak

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Concept and Viability Design Report

UPGRADING OF MOORDKUIL RAW WATER PUMP STATION

Mechanical, Electrical and Structural Design

Client: Department of Water and Sanitation (DWS)

Implementing Agent: Mossel Bay Municipality (MBM)

Submission date: 2025/09/26

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Current revision		**0**				

Approval			
Author signature		Approver signature	
Name	Izak van der Merwe	Name	Stephan Kleynhans
Title	Associate	Title	Design Director

CONTENTS

Appendices

APPENDIX A

Moordkuil River Abstraction Works Detailed Design Report Rev 02 – July 2025

Figures

Tables

1 Introduction

1.1 Project Background

Zutari (Pty) Ltd ("Zutari") was appointed by Neil Lyners and Associates (Pty) Ltd ("Lyners") as the sub-consultant responsible for the mechanical, electrical, and structural design of the Moordkuil Raw Water Pump Station Upgrade ("Moordkuil Pump Station"), located near the town of Klein Brak River. Lyners is the principal Professional Service Provider (PSP) or Consultant for this project, with the Department of Water and Sanitation (DWS) as the Client and Mossel Bay Municipality (MBM) as the Implementing Agent.

The Moordkuil Pump Station abstracts raw water from the Moordkuil River and pumps it to the Klipheuwel Dam, an off-channel storage facility for Mossel Bay Municipality (refer to Figure 1-1). The existing pump station comprises two axial flow pumps, of which one is currently operational (refer to Figure 1-2 and Figure 1-3). The design flow is 800 ℓℓ/s with both pumps operating in parallel (two duty, no standby). Over time, the station has become increasingly maintenance-intensive, primarily due to sediment and grit ingress from the riverbed, which has led to more frequent pump maintenance and failures. Historical maintenance records indicate that the pumps required regular refurbishment, causing significant operational disruptions and increased maintenance costs.

1.2 Earlier Studies

Over time, the use of axial flow pumps has declined due to the unavailability of spares and the complexities associated with long drive shafts. In the early 2010's, DWS engaged Zutari for advice on alternative pump types, noting that the proposed solution should require minimal civil construction and/or modification to existing structures. Because the pump station elevation was such that single stage pumping is not possible, it was proposed that two-stage pumping be considered with immersible pumps located in the river and end-suction pumps in the pump station building. In order to protect the immersible pumps during flood events, it was proposed that the pumps be located in a small concrete sump, positioned at the intake of the existing axial flow pumps.

In 2014, DWS appointed Lyners for the Moordkuil Pump Station upgrade. In turn, Lyners appointed Zutari as sub-consultant to undertake the mechanical, electrical and structural designs. At the start of this appointment, DWS indicated that sediment loads have been increasing over time due to developments (both commercial and agricultural) taking place within the upper catchment areas of the Moordkuil River. It was thus proposed that the small concrete sump be replaced with a larger intake structure to address the sedimentation concerns. ASPTech, specialists in river hydraulics and sedimentation, was appointed to undertake the necessary sedimentation modelling and to propose a layout for the intake structure, which was to be located at the intake of the existing pumps. The project was, however, suspended before the Concept and Viability (C&V) design phase could be completed but a Options and Feasibility Report (January 2015) and an Implementation Report (January 2016) were prepared, which form the basis for the current scope of work.

The project has since resumed with Lyners appointed in September 2024 to undertake the completion of project, which includes the phases from Concept and Viability to Close-Out.

1.3 Project Scope and Deliverables

The current appointment is to undertake all phases from ECSA Stage 2 (Concept and Viability Design) up to and including ECSA Stage 6 (Close-out). Zutari's deliverables for the structural, mechanical and electrical scope are summarised as follows:

- Concept and Viability Report (this report);

- Detailed Design Report;
- Tender Documentation as relevant for the structural, mechanical and electrical scope;
- Construction drawings; and
- Close-out Report as relevant for the structural, mechanical and electrical scope.

1.4 Project Area

The Moordkuil Pump Station is located on the banks of the Moordkuil River near the town of Klein Brak River as indicated in Figure 1-1.

Figure 1-1: Project Locality Plan

Figure 1-2: Axial flow pump motors

Figure 1-3: Axial flow pump riser pipes/columns

1.5 Report Structure

The Concept and Viability (C&V) Report is structured as follows:

- Chapter 1 provides the project background, details of the earlier studies, and lists the project deliverables;
- Chapter 2 details the options analysis undertaken to determine the optimal abstraction location and configuration;
- Chapter 3 considers the hydraulic design of the intake structure and raw water pump station;
- Chapter 4 provides details of the civil and structural design of the intake structure and modifications/additions to the existing pump station structure;
- Chapter 5 addresses the mechanical design of the raw water pump station;
- Chapter 6 details the electrical, control and instrumentation aspects of the raw water pump station; and
- Chapter 7 contains the conclusions and recommendations from the concept and viability design.

2 Options Analysis

2.1 Initial Investigations

During the previous project phases (2014 – 2016), several technical studies were undertaken, including:

- **Hydrology, Hydraulics and Sediment Dynamics:** Based on the increased sedimentation caused by developments in the upstream catchment area, sedimentation modelling had to be undertaken. The detailed modelling confirmed that sediment deposition at the existing pump intakes, especially during small floods, was a key contributor to pump wear and system failures. The studies recommended that the previously envisaged small concrete sump rather be replaced by a new larger intake structure to improve local scouring and thereby minimize the risk of sediment ingress.
- **Mechanical and Electrical Options Analysis:** Multiple pumping and intake configurations were evaluated. The preferred solution involved replacing the existing axial flow pumps with four immersible pumps (each 200 ℓ/s), arranged in series with four end-suction pumps. This configuration provides greater redundancy, flexibility, and ease of maintenance. An intake structure with hoppers was recommended to settle out coarse sediment before water reaches the immersible pumps, thereby reducing wear and extending pump service life.
- **Structural and Geotechnical Assessment:** The proposed intake structure is to be constructed within the existing development footprint. Geotechnical investigations evaluated the suitability of the founding conditions, with recommendations for a combination of piles and/or reinforced concrete spread foundations to address the variable alluvium thickness and hydraulic forces on the intake structure.

Based on the outcomes of these studies and subsequent stakeholder engagements, the following key decisions were made during the 2014 – 2016 project phases:

- The pump station will be upgraded to provide a maximum abstraction rate of 800 ℓ/s, utilizing four new pump lines (200 ℓ/s each), which each pump line having 2 x pump sets in series.
- A new low-level intake structure with hoppers will be constructed within the existing footprint to minimize the environmental impact associated with the new infrastructure. The immersible pumps will be housed inside this structure. The structure will be founded on piles.
- Other upgrades required to accommodate the four end-suction pumps will be limited to the extents of the exisitng building.
- All existing mechanical equipment, pumps and electrical switchgear will be replaced, with provision for a possible transformer upgrade.
- Structural modifications to the exisitng building will include a new gantry, repairs to the existing building and improvements to site access.
- An access road will be constructed up to the proposed low-level intake structure to access the mechanical equipment for routine operation, inspection, and maintenance of the mechanical systems, as well as to support the structural upkeep of the intake facility.
- The design should incorporate measures to protect concrete works from aggressive water quality (notably brackish water during tidal events).

Figure 2-1 and Figure 2-2 show the layout and isometric of the pump station developed during the initial project phases. In 2015, before the project was suspended, DWS procured the 4 x immersible and 4 x end suction pumps based on the outcomes of the initial design phases.

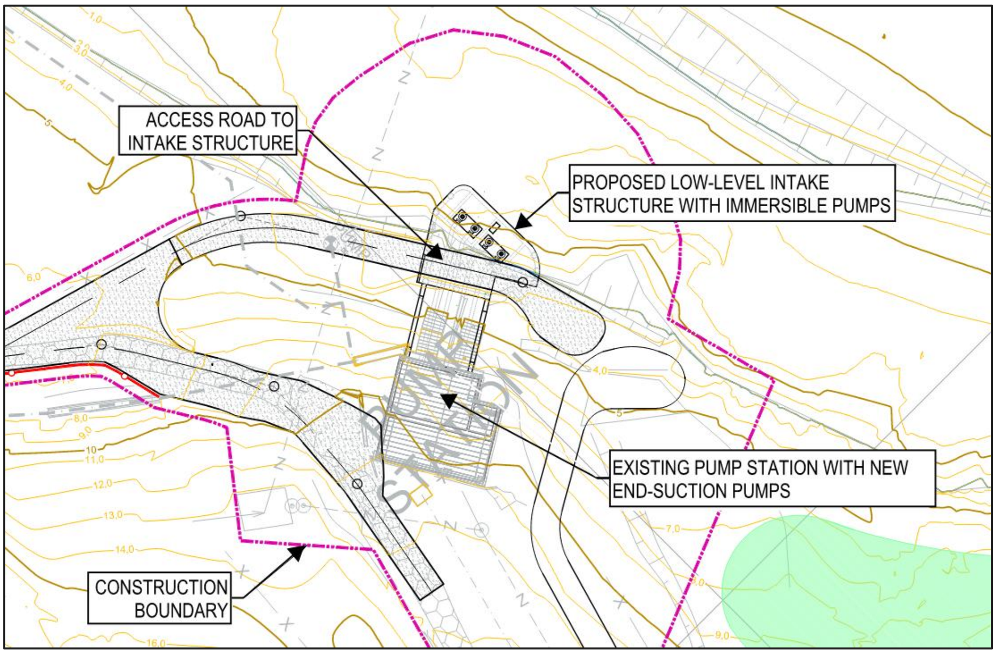

Figure 2-1: Preliminary Pump Station Layout (2014 – 2016)

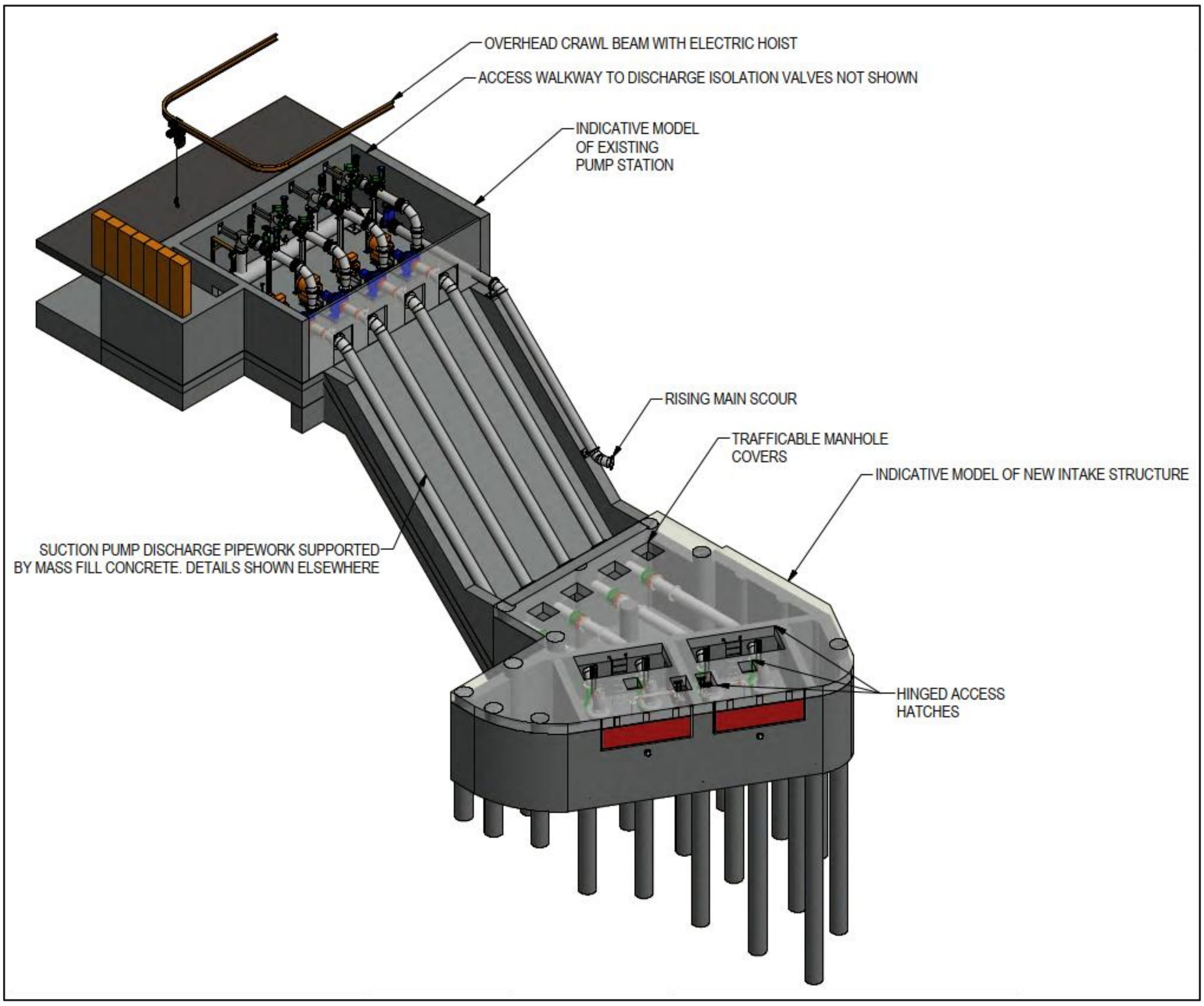

Figure 2-2: Preliminary Pump Station Isometric (2014 – 2016)

2.2 Verification Study

Upon recommencement of the project in late 2024, an underwater survey was undertaken to assess changes in the riverbed topography since 2014. ASP Tech, who undertook the initial sedimentation study, was subsequently appointed by Lyners to conducted a Verification Study to evaluate, among other factors, the appropriateness of the original intake structure in light of the updated bathymetry survey. The resulting report is provided as Appendix A.

The key findings from this verification study in relation to the initial solutions are as follow:

- There is an upstream rock obstruction that resulted in a significant amount of sediment deposition at the location for the existing pump intakes, which location was previously proposed for the new intake structure (refer to Figure 2-3). It is recommended that the rock outcrop be removed as the sediment deposition will influence the effectiveness of the intake structure.
- The width of the hoppers upstream of the pump intakes must increase from 2m to 4m to improve sediment removal and to account for the higher sediment loads in the river.

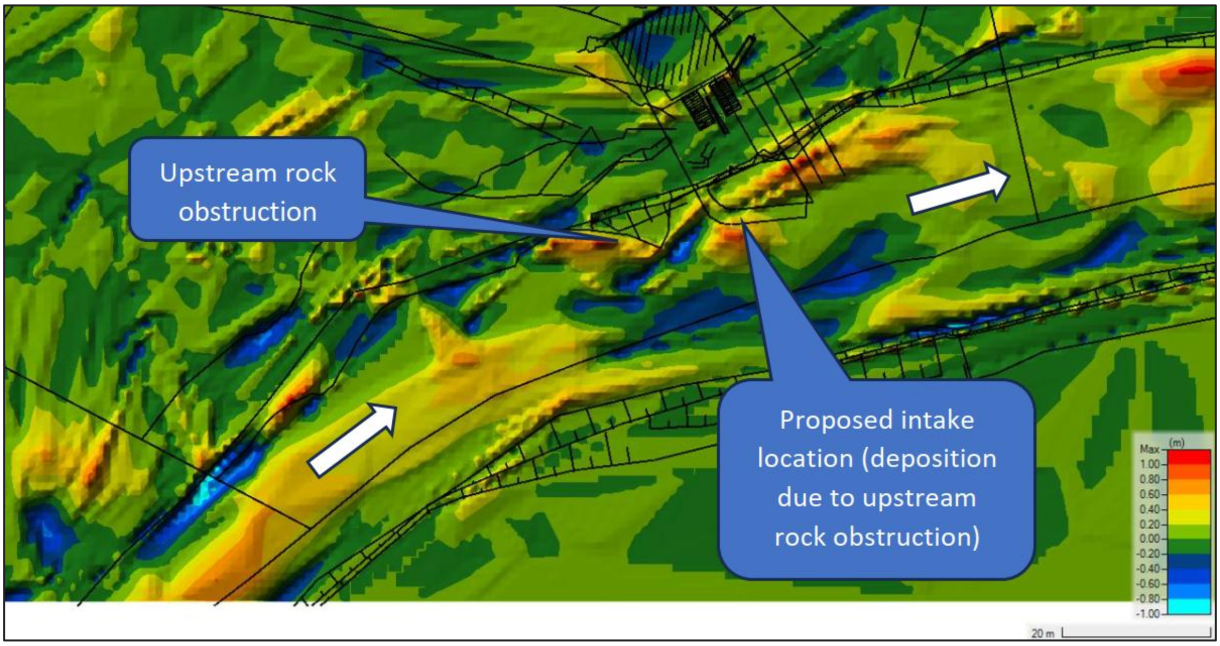

Figure 2-3: Riverbed topography changes from 2014 to 2025 (Red = deposition, blue = scour) (provided by ASP Tech)

The report further included comment on pumping options, noting the following:

- Consideration can be given to a single stage pump solution as the intake structure size can be increased to accommodate larger immersible pumps. KSB KRT 200-402 immersible pumps, or similar, installed inside the intake structure was proposed as a possible solution. Also refer to Section 2.3.1.1.
- End-suction pumps are generally considered more suitable to potable water applications as oppose to pumping raw water that could contain sediment and debris.

With reference to the comment on the end-suction pumps' suitability to pump raw water, it was observed on site that the farmers immediately downstream of the intake structure are using end-suction pumps. In discussion with one of the farmers, he indicated that the pumps required minimal maintenance. Furthermore, the introduction of the intake structure will limit the size of sediment and debris that can reach the pumps. While waste water pumps will be more suited to pump raw water, end-suction pumps are commonly used for river abstractions throughout the country.

2.3 Final Option Selection

Based on the findings of the Verification Study and subsequent engagements with DWS, MBM and other stakeholders, it was agreed that further consideration had to the location of the intake structure as well as the pump type selection. This section provides an overview of the additional considerations, outlines the revised options that were developed, and describes the process for selecting the final option.

2.3.1 Additional Considerations

2.3.1.1 System Configuration

Single stage pumping systems, in place of the initially proposed two-stage (i.e. in-series) pumping systems, present the following operational and strategic benefits:

- **Simplified control and operation** – Single-stage systems eliminate the need for coordination between the two in-series pumps. This significantly reduces the risk of pumps running dry or operating at shut-off head for extended periods of time due to faulty instrumentation.
- **Reduced Maintenance Requirements** – With only four pumps in operation compared to eight pumps in the two-stage setup, maintenance requirements are much less. This is likely to translates to lower labour costs, fewer spare parts, and reduced downtime for servicing.
- **Improved System Reliability** – Fewer pumps with the same reliabiltiy mean fewer potential points of failure. This enhances overall system reliability.

The disadvantages of a single-stage system are as follow:

- The pumps needs to be installed low enough to satisfy the NPSH requirements of the selected pumps.
- The pumps will be installed below the 1:100-year flood level, which could cause flooding of the pump installation.
- If the pumps are installed above the minimum water level, it requires a priming system consisting of vacuum or priming pumps that requires a high level of maintenance.
- If the pumps are installed above the minimum water level, it requires foot valves on the suction pipe. These valves are prone to malfuntion due to floating debris and gravel that get stuck in the valve and prevent proper closure of the valve and thus increase priming problems.

To overcome the disadvantages noted in the last two points above, immersible pumps, such as the KSB KRT 200-402 pumps proposed by ASPTech, in lieu of the initially proposed two-stage system is an option. Although this solution offer certain advantages, using an immersible pump of this size introduces certain technical challenges that require further consideration. The proposed KSB KRT 200-402 pump measures over 2 m in height, as shown on Figure 2-4, and weighs 1,367 kg. Due to its large dimensions and weight, special lifting provisions will have to be made at the intake structure. Furthermore, close-coupled motor configuration, and specific sealing requirements for maintaining a watertight installation, this size of pump necessitates specialised maintenance procedures that are rarely available within South Africa. This is aggravated by the fact that the pump will be located inside the intake structure with restricted access. Furthermore, each immersible pump cost approximately R1.1 million compared to approximately R335 000 for an end suction pumpset that can achieve the same duty point. Therefore, the end suction pumpsets can be replace at least 3 times at the cost of the immersible pumps.

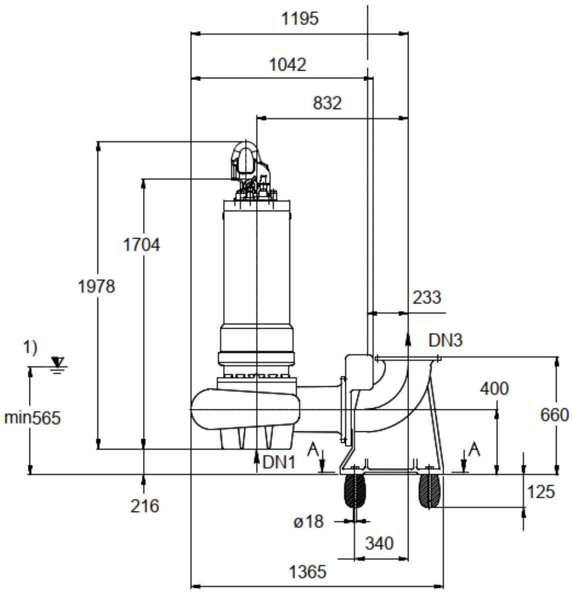

Figure 2-4: KSB KRT 200-402 Dimensions

DWS and MBM noted a preference for end-suction pumps due to their simplicity to maintain and the fact that DWS have experience in maintaining these pumps themselves.

However, as noted during the initial phases (2014 – 2016) of the project, the floor level of the current pump station is too high for the installation of end-suction pumps without the addition of immersible pumps located at the intake structure. **As such, an alternative system configuration, that maintains the benefits of a single-stage pumping system, as well as the benefits of using end-suction pumps were investigated. This alternative entails the construction of a drywell structure next to the current pump station but at a suitable level to enable end-suction pumps to be used in a single-stage pumping system.** This alternative is discussed further in the following sections.

It should, however, be noted that DWS procured the immersible and end-suction pumps after the 2014 – 2016 study with the intention to free-issue the pump to the installation contractor during the construction phase. The project was, however, suspended before construction commenced. The procured pumps are still available for installation.

2.3.1.2 System Layout

Due to the requirement for removal of the upstream rock outcrop, in terms of river sediment transport, and considerations about the founding of the intake structure in the initial solution (such as the potential need for piles to achieve suitable founding), an alternative layout for the intake structure upgrade was developed. This layout places the intake structure directly on the rock outcrop. As shown on Figure 2-5, if the intake structure is to remain in the current position (Concept Layout 1), the drywell proposed in the previous section, will be placed north of the existing pump station. If the intake structure is moved to the rock outcrop (Concept Layout 2), it is proposed to place the drywell to the east of the current pump station building.

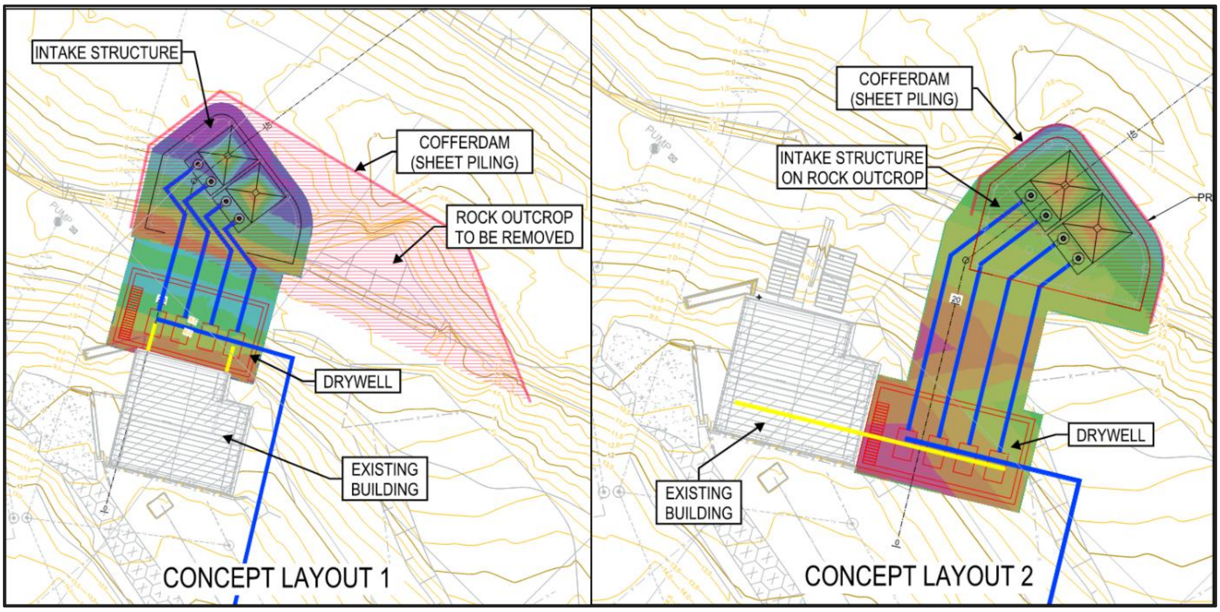

Figure 2-5: Concept Layout Options

The main advantages of the revised Concept Layout 2 option are as follow:

- Due to the rock outcrop, it is likely that this option will provide better and more economical founding for the abstraction works (to be confirmed with a supplementary geotechnical investigation);
- There is a reduced risk that the intake structure would affect the current dynamics of the river, such as causing scouring of the opposite bank. The structure would essentally replace the existing rock and not cause a localised narrowing of the river as is the case for the initial solution (Concept Layout 1);
- The existing pump station can remain operational during the construction of the intake structure and drywell; and
- For Concept Layout 2, the pumps installed in the drywell can be serviced by a single crawl beam and hoist, whereas a more complicated lifting equipment arrangement will be required for Concept Layout 1.

2.3.1.3 Water Treatment Considerations

Klipheuwel Dam is the raw water source of preference for Mossel Bay Municipality due to its lower treatment costs compared to Wolwedans Dam, which is the other main water source. Table 2-1 presents a cost comparison between using water from Klipheuwel Dam and Wolwedans Dam for Mossel Bay Municipality's bulk water supply, based on data from May 2022 to May 2025. The analysis indicates a **saving of R3.93/kL** when sourcing water from Klipheuwel Dam.

Flow meter records, for the same period, show an average monthly consumption of 347 017 kL from Klipheuwel Dam. If Moordkuil Pump Station is taken out of operation during the upgrade, and water is sourced from Wolwedans Dam instead, the bulk water supply will cost nearly **R1.4 million more per month**, totalling to **R24.5 million** over an 18-month construction duration. The benefit of Concept Layout 2, which will allow the existing pump station to remain operational during the construction phase, is significant in terms of the overall project costs.

Table 2-1: MBM Bulk Water Supply Cost Comparison

	Klipheuwel Dam	Wolwedans Dam
Approximate Treatment Cost (R/kL)	R8.26	R11.5 - R12.19 (R11.85 average)
Raw Water Tariffs (R/kL)	R3.014	R3.358
Total Cost (R/kL)	R11.274	R15.208

2.3.2 Option Selection

Taking into account the additional considerations outlined in the previous section, three final implementation options were evaluated:

- **Option 1** – Original concept (2016 report) with two stage pumping and without a drywell, as proposed during the initial project phases
- **Option 2** – Inclusion of a drywell with single stage end suction pumps as per Concept Layout 1
- **Option 3** – Inclusion of a drywell with single stage end suction pumps as per Concept Layout 2

Table 2-2 presents a comparative costing of the three options.

Table 2-2: Comparative Costing

		Option 1: Original Concept	Option 2: Concept Layout 1	Option 3: Concept Layout 2
Intake structure		R7,018,300	R7,018,300	R5,575,900
Drywell		R0	R2,417,620	R2,362,120
Mechanical Works		R8,420,760	R8,420,760	R8,995,760
Electrical Works		R4,700,000	R4,700,000	R4,400,000
Access Road [1]		R2,134,598	R2,134,598	R2,134,598
Meter Chamber and Air Valves [1]		R2,955,500	R2,955,500	R2,955,500
Rising Main Pipe [1]		R432,550	R432,550	R475,175
Temporary Pumping [2]		R2,900,000	R2,900,000	R0
P&Gs	30%	R8,568,512	R9,293,798	R8,069,716
Sub Total		**R37,130,220**	**R40,273,126**	**R34,968,768**
Add Forward Escalation on Civils	8%	R1,003,276	R1,196,685	R1,080,263
Add Forward Escalation on M&E	16%	R2,099,322	R2,099,322	R2,143,322
Sub Total		**R40,232,817**	**R43,569,133**	**R38,192,353**
Contingencies	20%	R8,046,563	R8,713,827	R7,638,471
Total (Excluding VAT)		**R45,176,783**	**R48,986,952**	**R42,607,239**

Note 1 – The cost estimates for these items were provided by Lyners in accordance with their civil design.
Note 2 – This value account for the temporary pumping system that will be required for options 1 and 2 to mitigate the additional water treatment cost (see section 2.3.1.3)

Option 3 is considered the most economical, followed by Option 1. While Option 3 includes additional costs related to the drywell, it benefits from significantly reduced costs for the intake structure – primarily due to the smaller cofferdam required (refer to Figure 2-5). The difference in mechanical and electrical (M&E) costs as shown in Table 2-2 is attributed to temporary pumping requirements during construction to ensure continues water supply to Mossel Bay Municipality.

Table 2-3 summarises the key advantages and disadvantages of each option. Option 3 is preferred from an Operational, Technical, Construction and Stakeholder perspective. Although Option 3 has a larger construction footprint, it has a lower long-term impact on the river dynamic, which makes this option beneficial from an environmental perspective.

Table 2-3: Option Comparison

Considerations	Option 1: Original Concept	Option 2: Concept Layout 1	Option 3: Concept Layout 2
Operational	•Complex two-stage system with higher maintenance and control requirements. •Specialised maintenance on the immersible pump.	•Simplified single-stage system with end-suction pumps. •Improved reliability. •End-suction pumps are easy to maintain	•Simplified single-stage system with end-suction pumps. •Improved reliability. •End-suction pumps are easy to maintain
Technical	•Existing pump station offers limited space for the mechanical and electrical equipment associated with the upgrade. •Complex lifting equipment arrangement inside the pump station to service all booster pumps.	•Includes the addition of a drywell for the mechanical installation •Existing pump station can be used for electrical installation. •Complex lifting equipment arrangement inside the pump station to service all pumps.	•Includes the addition of a drywell for the mechanical installation •Existing pump station can be used for electrical installation. •All pumps can be serviced by a single crawl beam.
Construction	•Requires pile foundations for the intake structure. •Existing pump station will be taken out of operation for the entire construction period. •Rock outcrop to be demolished.	•Requires pile foundations for the intake structure. •Restricted construction for the drywell between the existing pump station and the river •Existing pump station will be taken out of operation for the entire construction period. •Rock outcrop to be demolished.	•Intake structure founded on rock outcrop •More space available for drywell construction, but drywell will be deeper. •Existing pump station can remain operational for most of the construction period.
Environmental	•Smallest construction footprint. •Intake structure narrows the existing river and can potentially negatively impact on river dynamics.	•Slightly larger construction footprint. •Intake structure narrows the existing river and can potentially negatively impact on river dynamics.	•Largest construction footprint. •Intake structure location on rock outcrop minimises the narrowing and potential impact on river dynamics.
Stakeholder	•Relocation required of the farmer's pumps downstream of the intake structure to prevent the intakes from silting up.	•Relocation required of the farmer's pumps downstream of the intake structure to prevent the intakes from silting up.	•Farmer's pumps can remain in place.

Based on the advantages noted in Table 2-3, along with its lower overall cost, Option 3 was selected for implementation. To maximise the value of the existing pumps already procured, a phased implementation strategy will be adopted:

- **Phase 1** – Utilisation of existing pumps (immersible pumps in the hopper/intake structure and end-suction pumps in the drywell); and
- **Phase 2** – Replacement of the immersible pumps with a single stage of end-suction pumps installed in the drywell.

It should further be noted that a supplementary geotechnical investigation will be required for the revised location for the intake structure and the new drywell. This investigation will form part of the detailed design phase of the project.

3 Hydraulic Design

3.1 Intake Structure

After the initial Verification Study, ASPTech updated their report to included 2D hydrodynamic modelling of the sediment dynamics expected for the new location of the intake structure on the rock outcrop as per Concept Layout 2 in Figure 2-5.

For ease of reference, the key findings from this modelling, as documented in Appendix A, are summarised below:

- The proposed intake is situated within the scour zone on the outer side of the river bend and will be self-scouring during both minor and major flood events;
- The height of the proposed intake remains low and submerged during floods, so it does not divert flow towards the right bank. Simulations show that the opposite right bank is a sediment deposition zone, so erosion protection is not required here;
- The left bank and floodplain between the proposed intake and the causeway are subject to scouring during 10-year and 100-year floods. Erosion in this area should be monitored, and infrastructure in critical locations requires protection from scour; and
- The proposed intake location on the left bank bedrock upstream of the existing pump station should be used for the detailed design of the new river intake structure.

3.2 Pump Station

The minimum, normal and maximum system curves were calculated for the system. The minimum system curve scenario will occur when the pipeline is still new (i.e. smooth), using a roughness coefficient (k_s) of 0.03 mm, and when the river level is high, conservatively assumed at 8.5 masl (note, the 1:100-year flood level is estimated at 8.2 masl). The normal system curve scenario represents typical operating conditions throughout most of the pipeline's lifespan, with a k_s value of 0.15 mm and a river level of 2 masl. The maximum system curve scenario will occur when the pipeline is aged (i.e. rough), using a k_s value of 0.6 mm, and when the river level is at its lowest minimum recommended operating water level of 1.95 masl.

Figure 3-1 illustrates the system curves for the minimum, maximum and normal operating conditions. Figure 3-1 also shows the recommended duty point for the pump station duty point of 800 ℓ/s @ 34 m (i.e. 200 ℓ/s per pump @ 34 m head). It is recommended that the duty point is selected on the maximum system curve to ensure that the pump station can deliver the design flow for its entire design life.

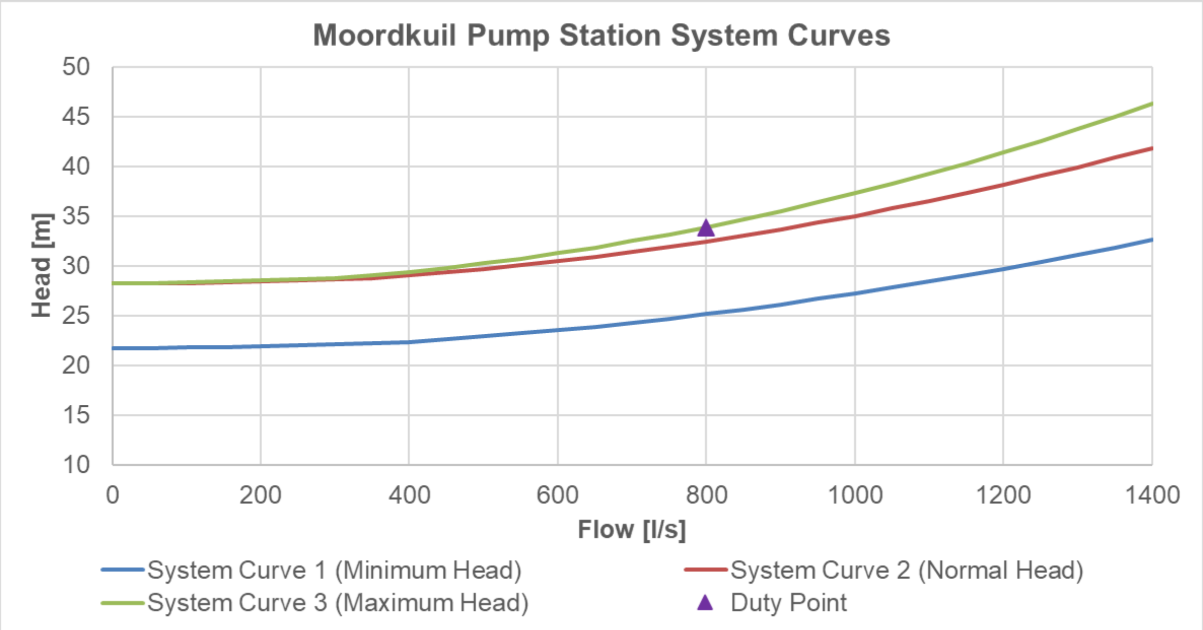

Figure 3-1: Moordkuil Raw Water Pump Station system curves

4 Civil Design

Figure 4-1 gives the floor plan for the proposed Moordkuil pump station upgrades. The design considerations for each of the main civil components are discussed in further detail in this section.

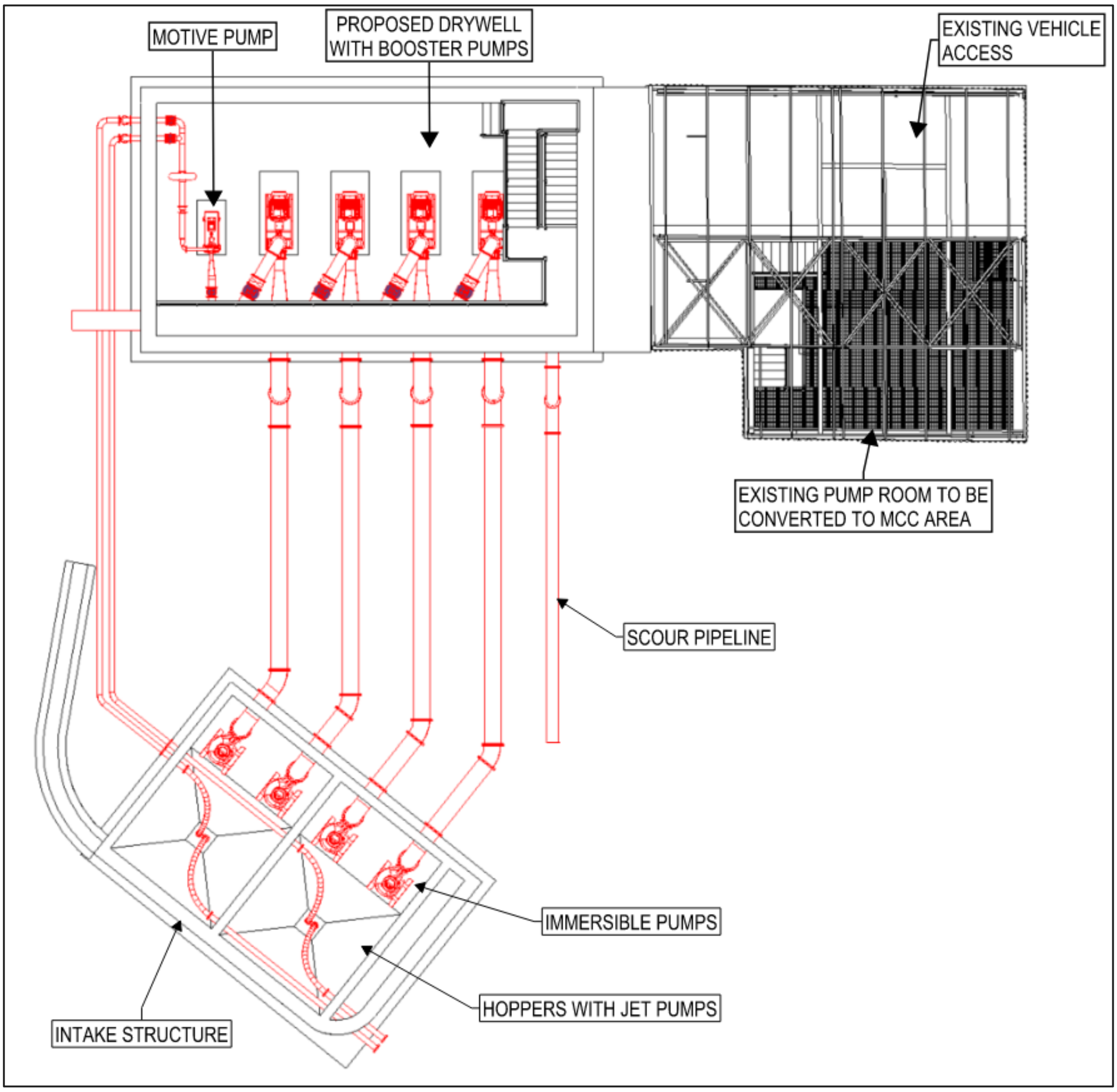

Figure 4-1: Pump station floor plan

4.1 Intake Structure

This section summarizes the design for the intake structure, which was based on the recommendations by ASP Tech during their initial investigations as well as the subsequent hydraulic verification.

Figure 4-2 shows the positioning of the intake structure on the existing rock outcrop within the natural narrowing of the river.

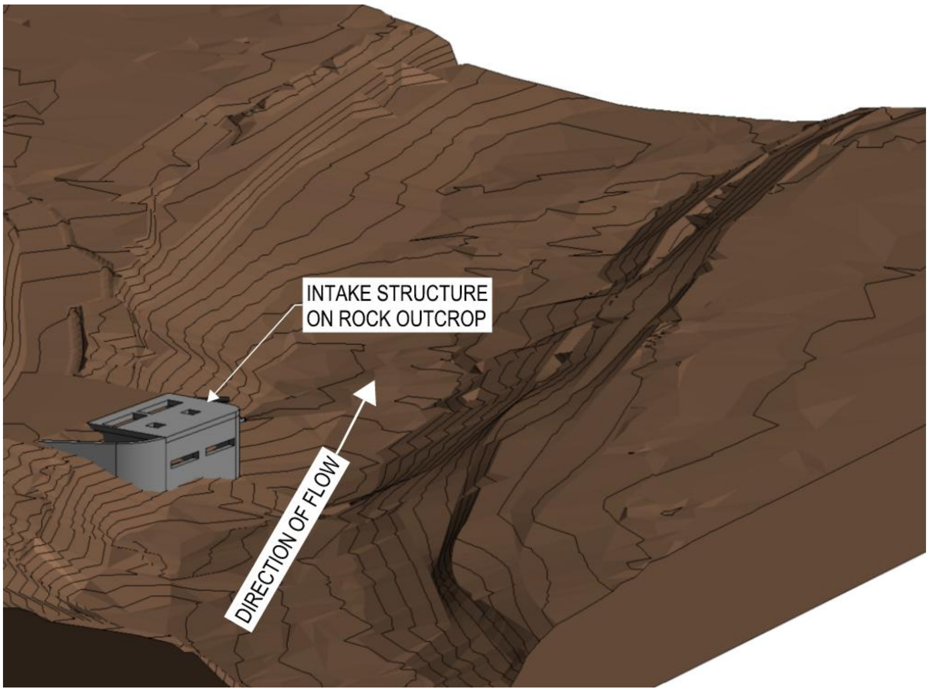

Figure 4-2: Intake structure positioning

Figure 4-3 and Figure 4-4 illustrate the plan layout and sectional view of the intake structure.

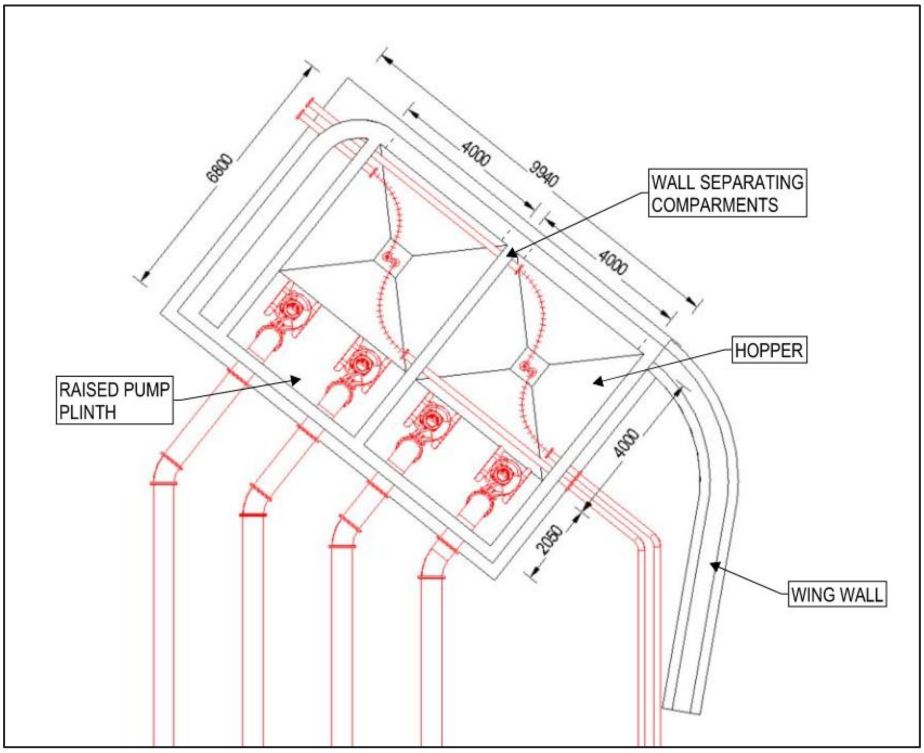

Figure 4-3: Intake structure plan layout

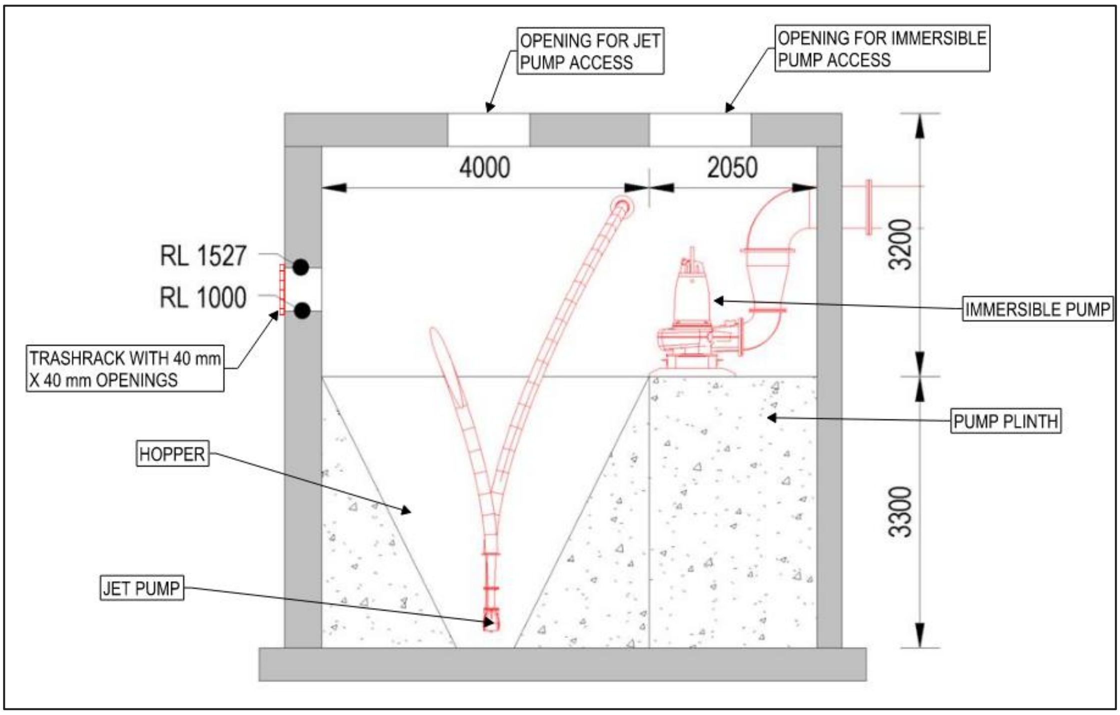

Figure 4-4: Intake structure sectional view

The design comprises two independent compartments to ensure operational redundancy. Each intake opening measures 527 mm in height and 3.4 m in width, and will be fitted with a trashrack featuring 40 mm x 40 mm openings.

To facilitate maintenance, each compartment will be equipped with a stoplog system allowing for isolation when required.

As recommended in the Verification Study, the sediment collection hoppers located upstream of the immersible pumps will be enlarged to dimensions of 4 m (width) × 4 m (length) × 3.3 m (depth). Jet pumps will be used to clean sediment from these hoppers.

The existing immersible pumps are mounted on dedicated pump plinths. When these pumps are eventually replaced, foot valves will be installed in place of the current immersible pumps.

Access to the immersible and jet pumps will be provided through roof openings, which will be sealed using removable precast concrete slabs secured with lockable stainless-steel bars.

4.2 Drywell

The proposed drywell will be located to the east of the existing structure as per Concept Layout 2 (refer to Figure 2-5). Figure 4-5, Figure 4-6 and Figure 4-7 show the isometric, plan and sectional views for the proposed drywell.

The width of the drywell was selected at 8.2 m to match the width of the upper level of the existing building. The length of the structure will be 13.5 m to accommodate the four end-suction pumps as well as the motive pump required to drive the two jet pumps.

The floor level for the drywell will be at RL 4.5 m based on the net positive suction head (NPSH) requirements of the proposed end-suction pumps (see Section 5.3). The drywell will feature a reinforced concrete lower structure supporting a steel upper structure. It is proposed that the concrete lower section will extend from the foundation up to the upper floor level of the existing pump station (RL 10.5 m). The steel upper structure will be designed to integrate seamlessly with the existing building's steel framework. The concrete lower portion will be constructed to be watertight, ensuring protection against water ingress during river flood events; the 1:100-year flood level is anticipated to reach approximately RL 8.2 m.

It is proposed to place the drywell at an offset of at least 2 m from the edge of the existing structure due to the level difference between the two structures. This is to ensure sufficient space is available for temporary lateral support and to ensure that the existing structure does not get undermined.

The drywell will be accessed from the existing pump station through a stairway. It is proposed that the stairway has two landing, of which one leads onto an intermediate walkway to access the handwheels of the discharge valves. It is proposed to construct the stairway and walkway from steel with GRP grating. The will also allow for simple and relatively quick construction.

The drywell will incorporate a drainage channel and sump pump to mitigate the risk of flooding due to a leaking coupling. A free drainage outlet was considered; however, this will allow water to push back into the drywell when the river is in flood.

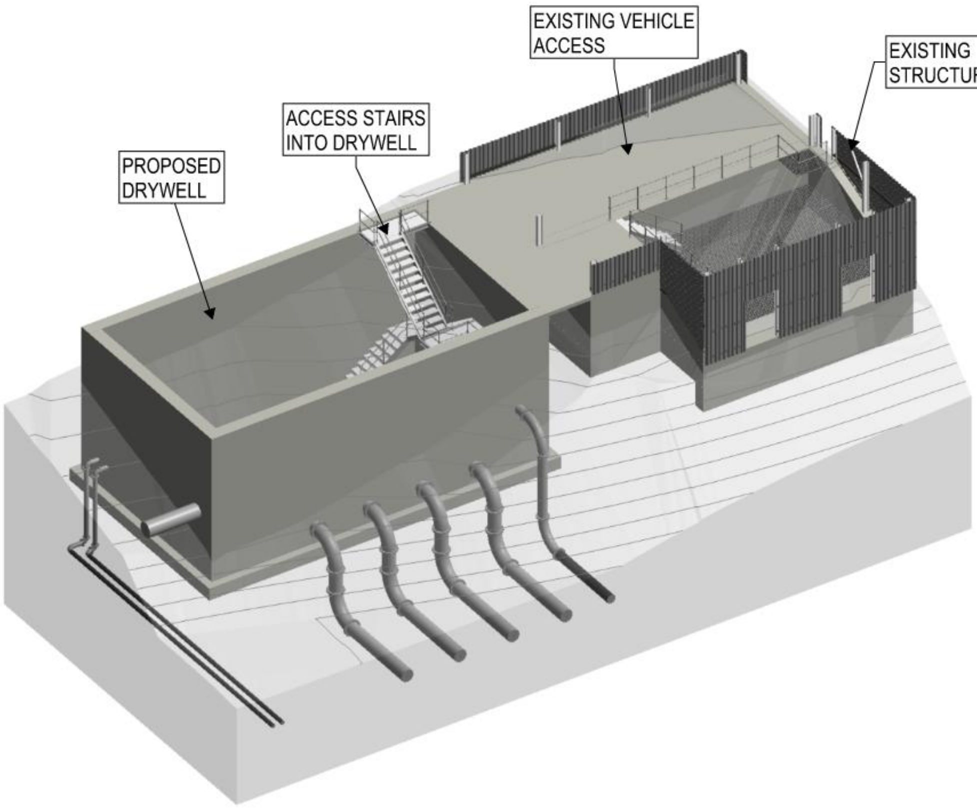

Figure 4-5: Drywell isometric view

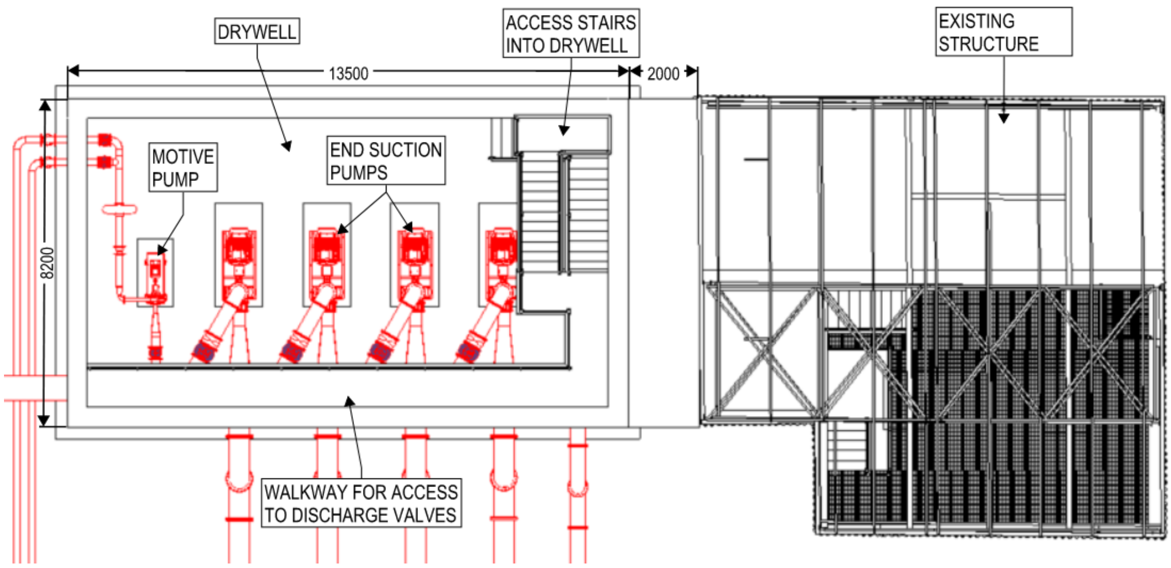

Figure 4-6: Drywell plan layout

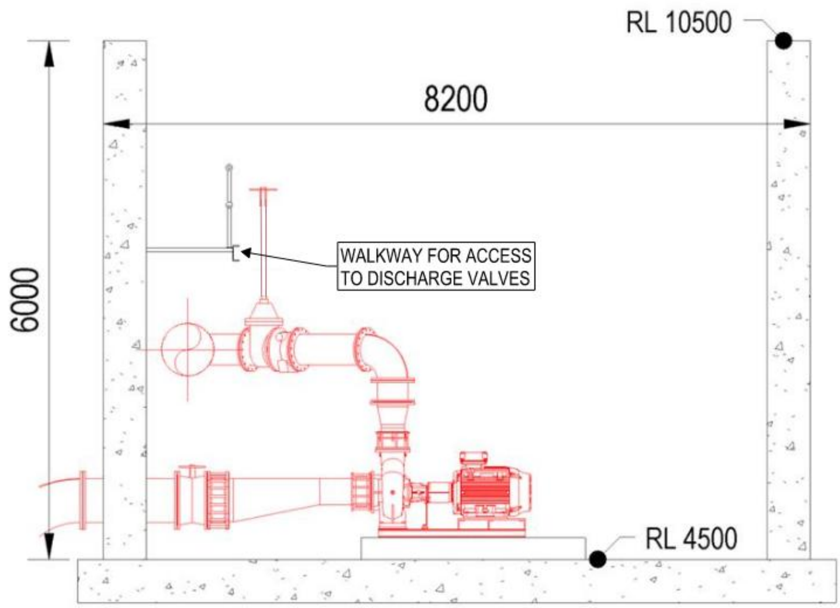

Figure 4-7: Drywell sectional view

4.3 Modification to existing structure

The modification of the existing structure will include the following:

- Removal of the eastern cladding for tying in with the drywell steel top structure;
- Replacement of the existing U-shaped crane beam with a single crane beam into the drywell; and
- The construction of a false floor above the existing pump floor for the electrical equipment.

Figure 4-8 shows the cladding that will be removed from the existing building's eastern face and where the steel top structure for the drywell will tie-in. Figure 4-8 also shows that the roof of the existing structure falls towards the drywell. As such, the roofs for the existing steel top structure and for the drywell will have to be tied together with a suitably sized central drainage channel.

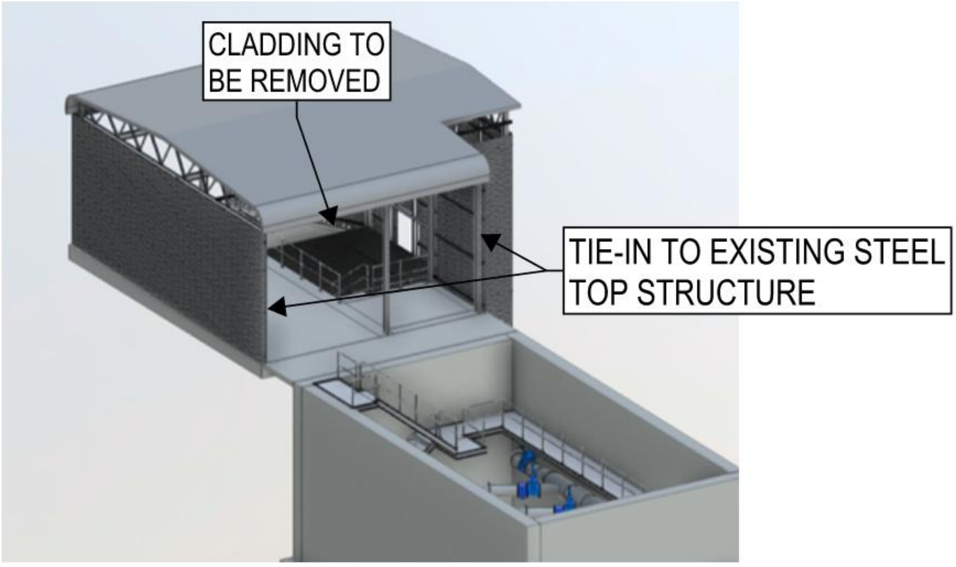

Figure 4-8: Existing structure cladding to be removed

Figure 4-9 shows the proposed crane beam that will be installed to service the pumps inside the drywell. The installation will allow for the existing vehicle bay to be utilised to load the pumps.

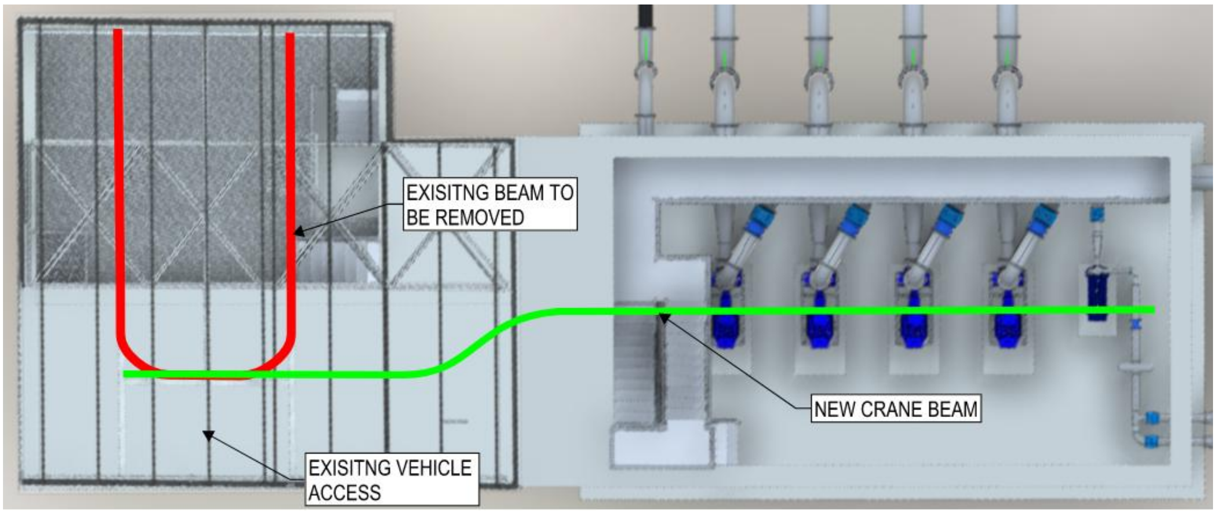

Figure 4-9: Crane beam modifications

Figure 4-10 and Figure 4-11 indicatively show the proposed false floor to be installed above the existing pump floor to house the electrical equipment. As the current pump floor is at RL 8 m – close to the 1:100-year flood level – the new false floor will be at least 1 m above the existing floor level to reduce the flooding risk for the electrical equipment. This elevation also keeps the false floor above the reinforced concrete motor plinths, mitigating the need for these plinths to be demolished.

It is proposed that the false floor is constructed from steel supports with solid GRP panels. The crawl space below the false floor will be used for cable routing to and from the MCC units.

Additionally, a cut-off wall is planned on the river side of the structure for extra flood protection.

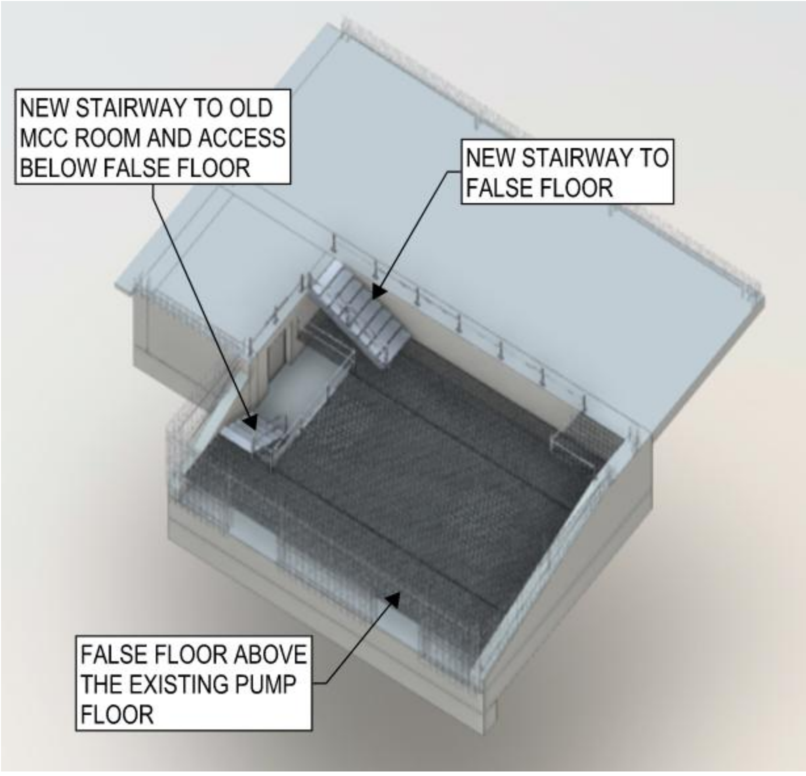

Figure 4-10: Proposed false floor for electrical equipment – isometric view

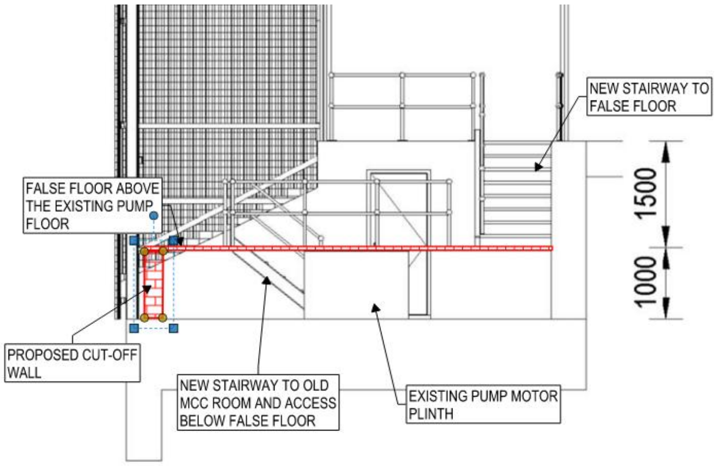

Figure 4-11: Proposed false floor for electrical equipment – section view

Figure 4-12 illustrates examples this type of raised floor arrangement which is commonly implemented within a data centre environment.

Figure 4-12: False floor example typically used in data centres

5 Mechanical Design

5.1 Design Philosophy

As noted in Section 2.3.2, the mechanical design for the Moordkuil Pump Station upgrade will involve a phased approach so that the existing pumps can be utilised as part of the first phase of the project.

In Phase 1, the existing pumps will be configured to operate in series, with each end suction and submersible pair producing 200 ℓ/s. The pumps will operate in a 4 duty, 0 standby, configuration to deliver a total flow of 800 ℓ/s under normal operating conditions.

In Phase 2, the arrangement will be simplified by removing the immersible pumps and installing foot valves onto the inlet pipework in the intake structure. The existing end-suction pumps will be replaced with larger end-suction pumps, which will draw directly from the intake structure via its individual suction lines. Each end suction pump will then independently deliver 200 ℓ/s in a 4 duty, 0 standby, configuration supplying a total flow of 800 ℓ/s.

To facilitate this transition, the pump plinths have been designed to accommodate the larger end suction pumps that will be installed in Phase 2. This staged approach ensures the efficient use of the existing pumps with minimal modifications required when the Phase 2 pumps are installed.

The mechanical installation discussed in the follow section are primarily for the Phase 1 installation. The detailed considerations for the phase 2 system, such as the operation of the foot valves and the suction pipework priming, will be included in the detailed design report.

5.2 Pump Selection

The pump selection for the Moordkuil Pump Station was done in conjunction with the options analysis and phasing discussed in Section 2. Table 5-1 below summarises the details of the existing pumps (Phase 1) and the outcome of the pumps selected for Phase 2.

Table 5-1: Pump Analysis

Description	Submersible Pumpset	End Suction Pumpset	End Suction Pumpset	Motive Pumpset
Phase	1	1	2	1 and 2
Orientation	Vertical	Horizontal	Horizontal	Horizontal
Configuration	4 duty, 0 standby	4 duty, 0 standby	4 duty, 0 standby	1 duty, 0 standby
Duty Flow (ℓ/s)	200		200	30
Duty Head (m)	37		33.9	37 (2)
Make and Model (1)	Flygt NS 3202 LT 3~ 614 (existing)	Lowara NSCF 250-315/750X/W45VDC4 (existing)	KSB ETA 250-40 (future)	KSB Etanorm 100-080-315 (future)
Efficiency at Duty Point	80%	82%	83%	78%
Absorbed Power at Duty (kW) per pump	24.5	64	80.5	14
Total Absorbed Power at Duty (kW)	98	256	322	14
Motor size (kW)	30 3	75 3	110	22
Total Installed Capacity (kW)	120	300	440	22

Note 1 – The listed pumps' make and model for the future phase are examples only; other manufacturers or models that meet the performance specifications are also available in the market.
Note 2 – The motive pump will be supplied from the discharge side of the end-suction pumps and will therefore deliver an effective head of ± 70m.
Note 3 – These are the motor sizes for the existing pumps.

For the conceptual design considerations, the new pump motors were sized with an 25% spare capacity margin above the required absorbed hydraulic power to prevent overloading under varying operating conditions and to ensure reliable operation. A more generous spare capacity is generally recommended for motors started Direct-on-Line (DOL). It should be noted that, for Phase 1, the spare capacity available in the existing pumps is less than the 25% standard specified for the new pumps. The immersible pump motors have a spare capacity of 22%, while the end-suction pump motors provide 17% spare capacity at the duty point. More advance start-up procedures might be required to ensure reliable operation of these existing pumps. This will be investigated further in the detailed design phase.

It is also noted that the existing Lowara end-suction pumps are generally proposed for clean water applications. The additional measures, such as mechanical seal flushing water, that may be required in order to utilise these pumps in a raw water application will be investigated further during the detailed design phase.

5.2.1 Phase 1: Existing Pumps

Phase 1 will utilise the already procured Flygt submersible pumps in series with the Lowara end-suction pumps. The combination of the existing pumpsets will deliver a flow of 800 ℓ/s at a pressure head of 37 m. The combined performance curves for each pump pair, Lowara & Flygt NS 3202 pumps, are given in Figure 5-1. The combined performance curves were generated by summing the individual pump delivered head by each pump for a specific flow rate.

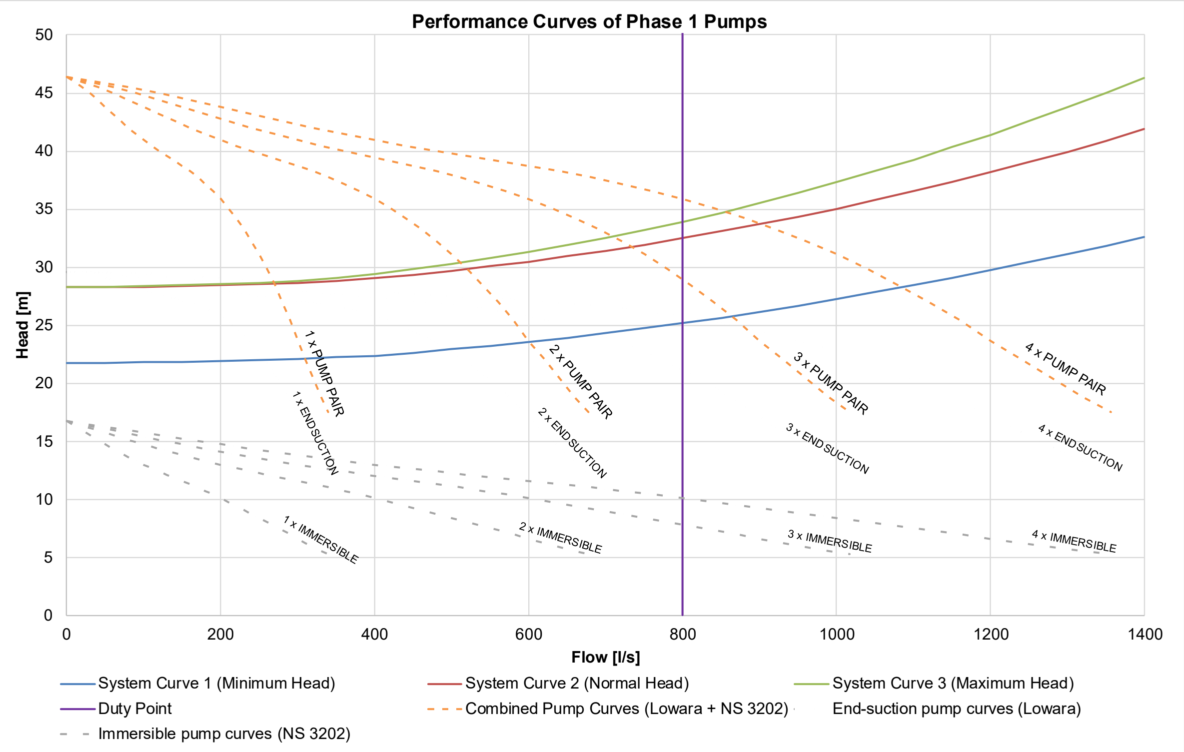

Figure 5-1: Performance Curve for Phase 1 Pumps

Table 5-2 below provides the operating scenario flow rates for all four pump configurations.

Table 5-2: Flow Rates for Operating Scenarios of Phase 1 Pumps

Operating Scenario	Minimum Flow (ℓ/s)	Normal Flow (ℓ/s)	Maximum Flow (ℓ/s)
1 x pump	270	271	310
2 x pumps	520	530	600
3 x pumps	720	750	870
4 x pumps	880	980	1080

5.2.2 Phase 2: Proposed Future Pumps

The performance curves for the phase 2, KSB ETA end suction pumps are given in Figure 5-2.

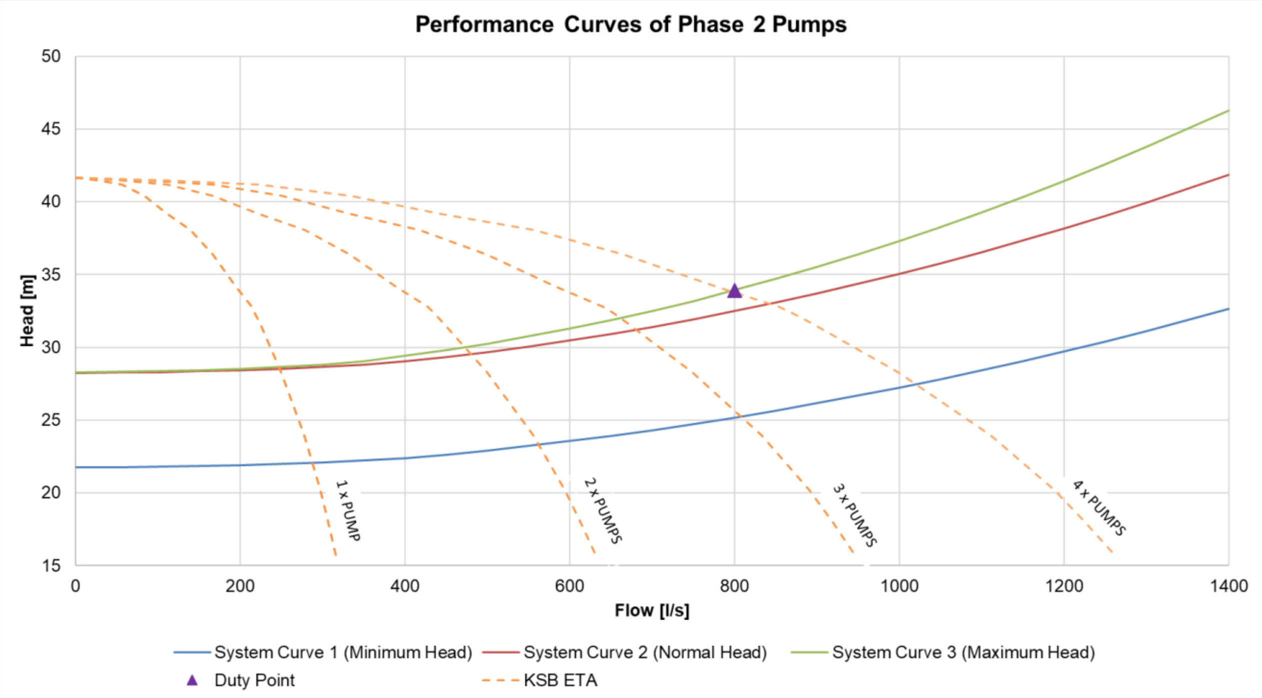

Figure 5-2: Performance Curve for Phase 2 Pumps

Table 5-3 below provides the operating scenario flow rates for all four pump configurations.

Table 5-3: Flow Rates for Operating Scenarios of Phase 2 Pumps

Operating Scenario	Minimum Flow (ℓ/s)	Normal Flow (ℓ/s)	Maximum Flow (ℓ/s)
1 x pump	250	250	297
2 x pumps	500	505	594
3 x pumps	669	749	832
4 x pumps	800	854	1010

5.3 NPSH Calculations

The water flowing through a pump drops in pressure at the eye of the impeller due to the acceleration. If it drops low enough to the vapour pressure, boiling will occur leading to the formation of vapour bubbles. As the vapour bubbles move from the impeller eye it reaches a region with a pressure higher than the vapour pressure. This results in the bubbles collapsing (imploding) as they change back to a liquid phase, causing shockwaves (cavitation) which result in damage to the impeller. This cavitation also results in a decrease in the pressure head produced from the pump.

To prevent damage due to cavitation, it is important to ensure that the $NPSH_A$ (Net Positive Suction Head Available) is more than the $NPSH_{req}$ (Net Positive Suction Head required) as specified by the manufacturers. $NPSH_{req}$ is the pressure head needed to ensure that the eye of the impeller is always above vapour pressure. $NPSH_A$ is the actual positive suction head available for the particular installation.

The NPSH is of particular concern for the phase 2 installation, where the pumps will be installed above the normal water level. As such, the NPSH calculations was used to inform the floor level required for the drywell i.e., the floor level was selected to ensure the $NPSH_A$ will always be more than the $NPSH_{reg}$. The following equation was used in the $NPSH_A$ calculations (from *Pumping Station Design* by *Garr M. Jones, 2006*):

$$NPSH_A = H_{bar} + h_s - H_{vap} - h_{fs} - \Sigma h_m - h_{vol}$$

Where:

H_{bar} = barometric pressure (10.3m water head in this case);

h_s = static pressure;

H_{vap} = vapour pressure (0.13m in this case);

h_{fs} = pressure loss due to friction;

Σh_m = sum of minor pressure losses; and

h_{vol} = partial pressure of dissolved gases (deemed to be negligible in this case).

It is recommended that a **safety margin of at least 0.6 m** is maintained for the $NPSH_A$ compared to the $NPSH_{req}$ for all operating conditions.

Table 5-4 below shows a summary of the results of the NPSH calculations for the expected operating flow range for the phase 2 pumps, as obtained from Figure 5-2, and for a **drywell floor level of RL4.5 m**. For the reference pumps used in this design, the safety margin for the entire operating range will be greater than 0.6 m.

Table 5-4: NPSH Required and Available Summary

	Flow (ℓ/s)	$NPSH_{req}$ (m)	$NPSH_A$ (m)	Safety margin (m)
Minimum Flow	200	4.2	6.4	2.2
Maximum Flow	297	5.0	5.9	0.9

Figure 5-3 below further illustrates that with a RL4.5 m floor level for the drywell, the $NPSH_A$ for the Phase 2 pumps will exceed the $NPSH_{req}$ for the full operating flow range.

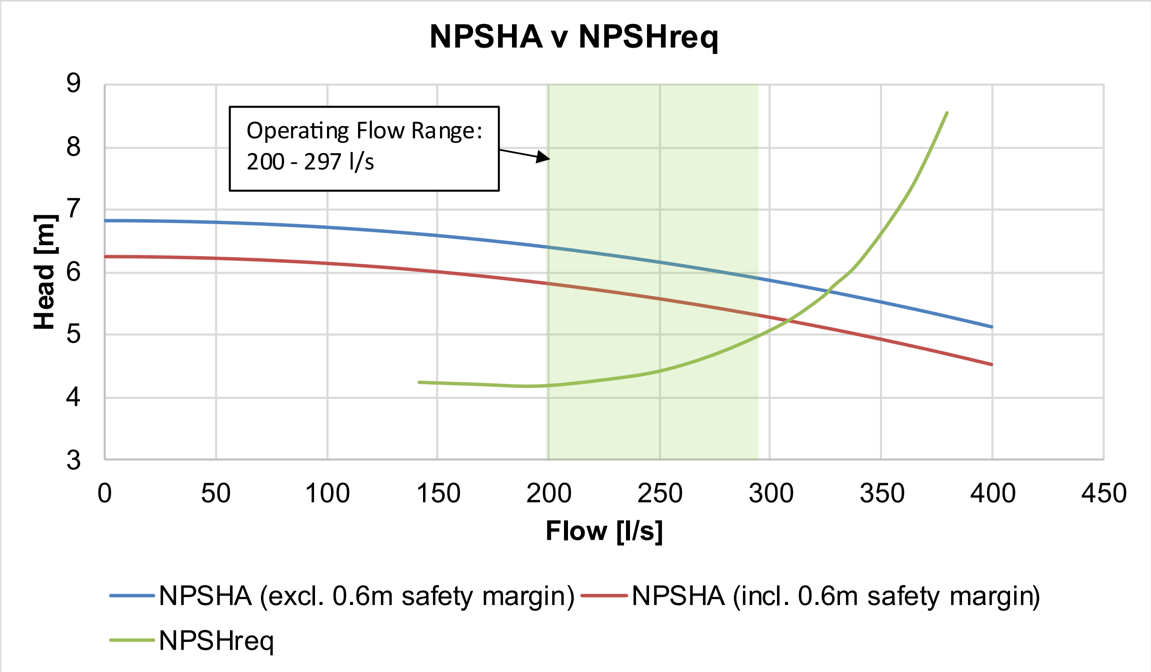

Figure 5-3: NPSHA vs NPSHreq

It should be noted that this calculation is dependent on the equipment used in the installation, and as such, the following should be considered when procuring equipment for the phase 2 installation.

- The calculation assumed a local loss factor of **5** over the foot valve proposed for phase 2. According to literature, based on the type of foot valve selected, this actual local loss factor can range from 1 up to 15.
- The NPHA$_A$ was compared to the reference pump used for the phase 2 design. The final pump procured must have a similar or lower NPSH$_{req}$ curve than the reference pump.
- The pumps should not be operated for a water level below the specificied minimum operating level of **RL 1.95 m.**

5.4 Pipework and Valves

Table 5-5 gives a summary of the preliminary sizing of the pipework for the Moordkuil Pump Station. The sizing is based on minimizing velocities in the suction and discharge pipework to be under 1.6 m/s and under 3.1 m/s, respectively.

Table 5-5: Pump Station Pipework Sizing

Pipe Segment	Minimum Flow (ℓ/s)	Maximum Flow (ℓ/s)	Proposed Size	Minimum Velocity (m/s)	Maximum Velocity (m/s)
Suction Pipework	250	310	DN500	1.3	1.6
Discharge Pipework	250	310	DN350	2.6	3.1
Discharge Manifold	800	1080	DN600	2.8	3.8 [1]
Motive Suction Pipework	30		DN200	1.2	
Motive Discharge Pipework	30		DN150	1.8	
Jet Pump Flexible Hose	30		DN150	1.8	

Note 1 – the discharge manifold was sized to match the rising main. It is noted that under normal conditions, the velocity in the rising main (and thus in the discharge manifold) is at the upper limit of the acceptable range. Under maximum flow condition, this velocity is above the acceptable range. The implication of this will be further assessed in the detailed design phase in conjunction with a water hammer analysis. However, to mitigate the high velocities, it is recommended not to operate the pumps while the river is in flood conditions.

Figure 5-4 below shows the proposed pipework and valve arrangement for the pump station. The following valve types and sizes are proposed for the different pipe segments:

- Suction Isolation Valve: DN500 Butterfly Valve
- Discharge Check Valve: DN350 Slanted Seat Tilting Disc Check Valve
- Discharge Isolation Valve: DN350 Resilient Seal Gate Valve
- Rising Main Scour Valve: DN350 Wedge Gate Valve
- Motive Pump Suction Valve: DN200 Resilient Seal Gate Valve
- Motive Pump Discharge Valve: DN150 Nozzle Check Valve
- Jet Pump Isolation Valves: DN150 Resilient Seal Gate Valve

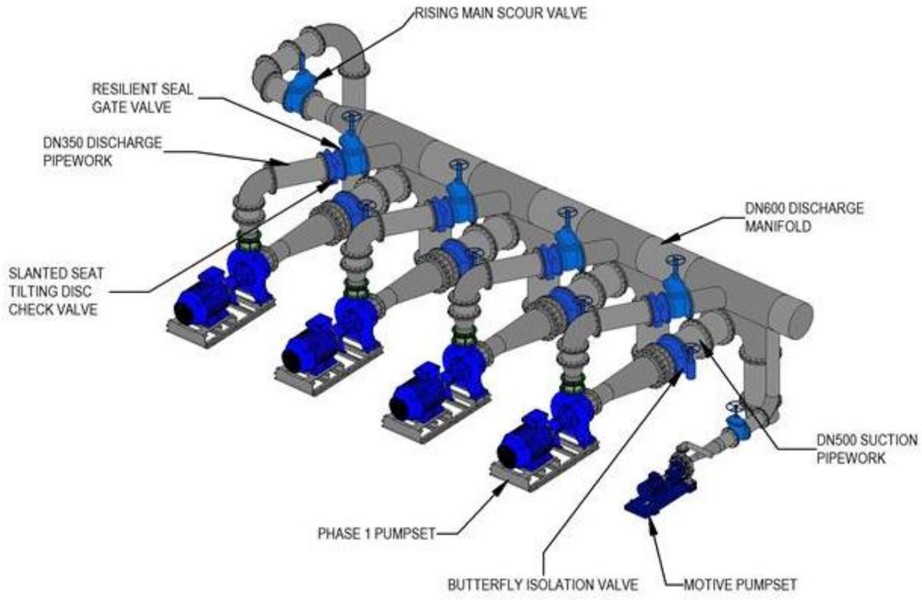

Figure 5-4: Pump Station Pipework and Valve arrangement

While vertically installed nozzle or silent check valves were considered, they are more prone to blockages from floating debris like plastic, branches, and rags in raw water applications. As such, slanted seat tilting disc check valves are preferred for this application. As these valves are not suited for a vertical installation, the check valves will be placed on the horizontal pipework upstream of the discharge isolation valves.

It is further proposed that a scour pipeline back to the river is provided to enable the rising main to be scoured.

5.5 Lifting Equipment

The existing I-beam and attached crawl beam will be modified and extended to span across both the current and the new adjacent dry well (refer to Figure 4-9). A motorised trolley with an electric chain hoist will operate on the extended crawl beam, which will run over the centre line of the pumps and be installed to allow pumps to be lifted over one another with adequate clearance for the hoist and hook. The minimum safe working load of the lifting equipment shall be 2000 kg. The Phase 1 combined pumpset weight (i.e. pump, motor and baseplate) is approximately 1200 kg. The Phase 2 pumpset (i.e. pump, motor and baseplate) will weight approximately 1500kg. The lifting equipment will be rated for 2000kg (2-ton) to meet the recommended safety margin of 30%.

The option exists to repurpose the existing hoist in the pump station, shown in Figure 5-5, which is also rated for 2000 kg. The feasibility of using the existing hoist will be considered in the detailed design phase.

Figure 5-5: Existing pump station hoist

5.6 Motive and Jet Pump System

Permanently installed jet pumps, one located in each of the two hoppers, are proposed to intermittently remove settled sediment. A section of the intake structure is illustrated in Figure 5-6 in which the hoppers, jet pumps and submersible pumps can be seen. The timeous removal of coarser settled sediment protects the submersible pumps, the low-lift pipeline, the end suction pumps and the rising main from transporting abrasive particles that can damage the mechanical equipment and settle in the pipelines. The removal of settled non-cohesive sediment through the jet pumps to the river should occur in short bursts with minimal to no impact on the ecology of the river downstream of the works.

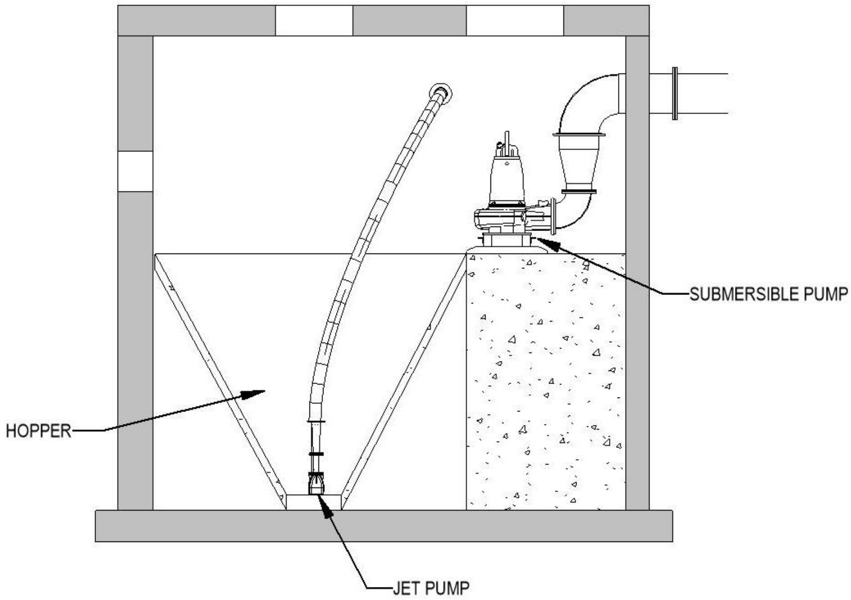

Figure 5-6: Intake structure section

Sediment-free motive water is required for each jet pump at a rate of 30 ℓ/s and a total head of 65 m to ensure optimal sediment removal in the hoppers, as recommended by ASPTech in the Verification Report.

It is recommended that the motive water be supplied by tapping into the DN600 discharge line from the Moordkuil pump station. Since the discharge line already provides at least 28 m of head (minimum delivered head when one pump is running), it is recommended that the dedicated motive pump only needs to supply the remaining 37 m of head to meet the total 65 m required by the jet pumps.

As per Table 5-1, a KSB Etanorm 100-080-315 pump was selected for the reference design for the motive pump. Figure 5-7 shows the pump curve for this pump against the proposed duty point.

A sediment filter will be installed downstream of the motive pump to ensure that the water supplied to the jet pumps remains sediment-free.

For operation and control, it is envisaged that only one jet pump will be used at a time. Automatically actuated isolation valves will be installed to alternate between the two jet pumps as required.

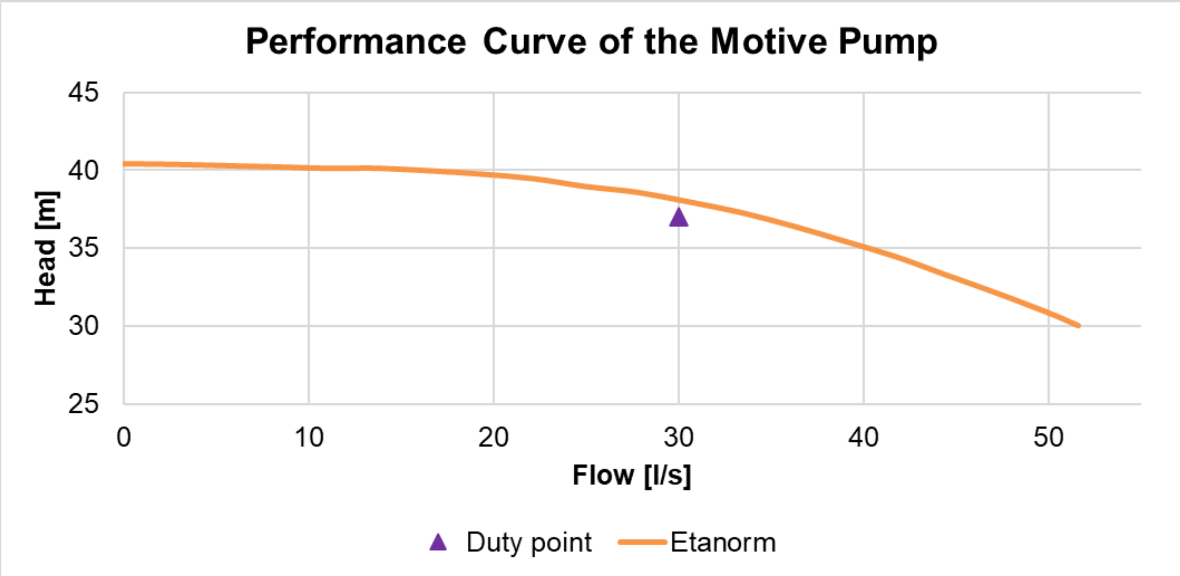

Figure 5-7: Pump Performance Curve for the Motive Pump

6 Electrical, Control & Instrumentation Design

6.1 Bulk Power Supply

The electrical supply authority is Mossel Bay Municipality. The supply to site includes a 22kV overhead line and a 500kVA 22kV/400V outdoor transformer. The transformer is located approximately 60m from the existing building. Initially, this transformer fed the Moordkuil Pump Station and two water pumps belonging to local farmers located on either side of the river. Upon further investigation it has been found that the transformer now only feeds the Moordkuil Pump Station and one farmer, located on the opposite side of the river.

As stated in Section 2.3.2, the option chosen for the project will have two phases; the first comprising of four 30 kW submersible pump motor and four 75kW booster pump motors while phase two will have four standalone 110 kW end suction pumps. The existing and new equipment for this project fed from the transformer are listed below in Table 6-1.

Table 6-1: List of proposed electrical equipment

Description	Rated Power (kW)	
Project Phase	Phase 1	Phase 2
Farmer Water Pump	30	30
4 x Submersible Pumps @ 30kW each	120	-
4 x Booster Pump motors @ 75kW each	300	-
4 x Future End-Suction Pumps @ 110kW each	-	440
1 x Motive Pump @ 30 kW	30	30
Miscellaneous (Small power & lighting, gantry crane, instrument power etc.)	10	10
Total Installed power	460	480

When accounting for efficiency and power factor losses, the estimated load to be supplied by the transformer is about 589 kVA for Phase 1 and 558 kVA for Phase 2. In addition, the client has requested that we use direct online (DOL) starters for all pumps, which has increased the maximum apparent power demand. The existing 500 kVA transformer will not have sufficient capacity for the required upgrades and will need to be replaced with an 800 kVA transformer.

The cables running from the transformer to the incomer will also need to be upgraded to accommodate the increase in power demand. It is estimated that 2x120mm² PVC Cu cables per phase are required. These cables will be routed in a trench to the existing building.

6.2 LV Distribution

As mentioned in Section 4.3, the existing Moordkuil Pump Station building will be retrofitted to accommodate the additional LV MCC equipment required for the new pump systems A false floor constructed from steel supports with solid GRP panels will be installed at a level above the 1:100-year floodline. This was chosen as the existing electrical room is too small to accommodate the new MCC and to allow cables to run underneath for ease of installation and maintenance. The area will be carefully controlled as it will now be designated as an electrical room; appropriate access control and safety measures will be in place to ensure compliance with regulations. The LV MCC will be placed on the GRP panels and the flooring beneath the MCC shall be reinforced to accommodate the weight of the MCC panels.

The approximate dimensions of the MCC are 4800mm wide x 600mm deep x 2050mm high from the level of the floor, however, this is subject to change pending final vendor information. The MCC will include an allowance for a programmable logic controller (PLC) panel to be placed at the end of the MCC. The 30kW drives, which will supply the submersible pumps, will be housed in the main MCC. Once Phase 2 commences, the submersible pumps will be removed and the 30kW drives will become equipped spares. In addition, 110kW DOL starters will be installed from Phase 1. The 75kW booster pumps will initially be connected to these starters which will eventually be replaced by the new 110kW end-suction pumps in Phase 2.

It has been agreed with DWS that the drives will be direct online (DOL) starters and not variable seed drives (VSD). This was decided for ease of maintenance and longevity of the pump station. A motor starting study should be carried out to ensure all equipment are suitable sized and that appropriate standards are met, such as NRS-048-4. This study shall be performed as part of the detail design phase of the project.

DWS has previously indicated that they do not prefer using any other drive besides DOL drives, but the health of the equipment and the stability of the bulk supply network must be considered. If the motor starting study deems the in-rush current and voltage drop not acceptable, the project will consider replacing the DOL drives with soft-starter drives.

The cables from the MCC to the pumps in the drywell will be routed underneath the GRP panels and through the existing building to the drywell. In the drywell, the cable will run underneath the elevated walkway and tee off to each pump. From the same MCC, the cables going towards the intake structure will be routed to the intake structure. Reasonable measures will be put in place to ensure exposure to water is minimised, especially given that the intake structure is located below the 1:100-year floodline. The final layout and design will be done during the detail design phase of the project.

6.3 Existing farmer’s pump system

As mentioned in Section 6.1, only one of the farmers’ pumps is connected to the existing 500kVA transformer. The scope of this project will be to reconnect the supply of this pump to the new 800kVA transformer. Alterations to the farmer’s pump system and electrical equipment shall be kept to a minimum. If alterations are made to any electrical equipment or cabling, the project becomes responsible for compliance.

6.4 Control System

The Moordkuil Pump Station will have a straightforward control and operating scheme and shall include, but is not limited to, the following:

- Pump system protection;
- Pump system control;
- Automated valve control;
- Measurement, indication and control of pressure, flow, level etc.;
- Electrical power quality measurement and power failure sequences; and
- Emergency stop sequences.

As a minimum, the control system will comprise of the following equipment:

- Programmable logic controller (PLC);

- Uninterruptible power system (UPS);
- Human machine interface (HMI);
- Input-output (IO) interface cards;
- Control network cabling (ethernet TCP/IP, Modbus etc.); and
- Instrumentation.

6.4.1 Business intelligence and reporting

To promote digitisation and remote monitoring, it is proposed that the Moordkuil Pump Station include equipment to allow remote, real-time monitoring to the Mossel Bay Municipality. Currently no remote monitoring system is in place for the existing system and routine visits are required to ensure the pump station is running as planned. DWS has advised only monitoring is required, no remote control will be implemented for this project. Remote monitoring will ensure that the operators receive real-time information of the status of the plant and to allow the operators to act accordingly for any breakdowns or alarms that might arise.

Due to the complexity of the hardware and software required to implement remote monitoring, the appointed contractor must ensure that the development team, either in-house or sub-contracted, have specialist experience and certification in SCADA and historian development as well as experience and certification in data warehouse and business intelligence development.

The software and hardware requirements for remote monitoring include, but is not limited to:

- SCADA servers and licensing;
- VPN routers;
- At least one PC based operator workstation for remote monitoring located at the Mossel Bay Municipality Technical Services office;
- Managed switches; and
- GPS clock.

This equipment will be housed in either the PLC panel or a dedicated area within the Mossel Bay Municipality Technical Services offices.

7 Conclusions and Recommendations

The objective of this report is to finalize the concept and viability design undertaken for the structural, mechanical and electrical components for the Moordkuil Pump Station upgrade.

A Verification Study conducted on the riverbed topography at the onset of this investigation identified the presence of a significant rock outcrop upstream of the proposed intake structure. As a result, it was determined that the outcrop would need to be removed, and the intake structure enlarged beyond the dimensions proposed in the 2014 – 2016 feasibility study to ensure effective operation.

An alternative solution involves relocating the intake structure directly onto the rock outcrop, which presents several technical and economic advantages:

- **Improved Foundation Conditions:** Relocating the intake would result in more favourable geotechnical conditions, potentially yielding a cost saving of approximately R1.7 million.
- **Operational Continuity:** By situating the intake upstream at the rock outcrop, it may be possible to maintain operation of the existing pump station throughout the construction period.
- **Minimal Disruption to Adjacent Infrastructure:** The nearby farmers' pump station, located just downstream of the existing station, would remain unaffected.

Should the existing pump station remain operational during construction, an estimated cost saving of approximately R24.5 million could be realized over the 18-month construction period by avoiding the need to purchase water from the Wolwedans Dam and the associated saving in chemicals at the water treatment works.

A new dry well pump station could be constructed at an estimated cost of approximately R2.4 million. This facility would enable continuous operation of the existing pump station with minimal interruption of the water abstraction during the construction phase. An additional benefit of constructing a permanent dry well is that it would allow the end-suction pumps to be installed at a lower elevation. This could possibly eliminate the requirement for immersible pumps within the intake structure in the future and enable the use of foot valves in combination with a priming system. A new dry well will also provide additional space for the installation of the proposed electrical equipment as the existing MCC room is very small.

Based on the abovementioned, **Option 3** (wetwell on rock outcrop with drywell next to pump station, Concept Layout 2) is recommended for implementation for the Moordkuil Pump Station upgrade, as it offers the lowest capital and operational costs, best operational reliability, and acceptable environmental impact. To maximise the value of existing assets, a **phased approach** should be adopted:

► **Phase 1**: Utilisation of existing immersible and end-suction pumps that was bought based on the previous (2014 – 2016) investigation.

► **Phase 2**: Replacement of the immersible pumps with foot valves and installing larger single stage end suction pumps in die drywell. *The detailed considerations for this system, such as the operation of the foot valves and the suction pipework priming, will be included in the detailed design report.*

The hydraulic design ensures the intake structure are self-scouring and resilient to sediment deposition. The civil design provides for robust, flood-resistant structures, with careful integration of new and existing facilities to maintain operational continuity during construction.

The mechanical design supports both current and future pump configurations, with appropriate safety margins for motor sizing and lifting equipment.

The electrical design requires upgrading of the transformer and cabling to accommodate increased power demand, with a focus on direct online (DOL) drives for reliability and ease of maintenance.

The control system will be kept as simple as possible, with automated protection, measurement, and reporting. Remote monitoring will be implemented for real-time status updates, but remote control will not be enabled, as per client requirements.

Based on this concept and viability design report, it is recommended to proceed with the detailed design of the selected option, i.e. Option 3 at an estimated cost of R42.6 million.

APPENDIX A

Moordkuil River Abstraction Works Detailed Design

Report Rev 02 – July 2025

MEMO IN PROGRESS

MOORDKUILS RIVER ABSTRACTION WORKS DETAILED DESIGN

Review of the proposed hydraulic design of the intake works with 2D hydrodynamic of the scour/deposition and 3D CFD modelling of hydraulic forces

REPORT rev02

July 2025

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Table of Contents

1. Introduction

The Department of Water Affairs and Sanitation (DWS) recently appointed the Mossel Bay Municipality as their Implementation Agent to implement the proposed upgrading of the Moordkuil Pump Station which is part of the Mossel Bay Regional Water Supply Scheme (RWSS). The Moordkuil Pump Station abstracts water from the Moordkuil River and discharges into the Klipheuwel Dam which is an off-channel storage dam – see Figure 1-1. The required pumping capacity of the Moordkuil pumpstation is 800 litre/s to be provided by four pumps. The purpose of the proposed upgrade (mainly the river works) include:

- Prevention of sedimentation interrupting the operation of the pumpstation under all river flow conditions.
- Provision of a permanently installed sediment removal system to remove suspended sediment that settles in the pump forebay.

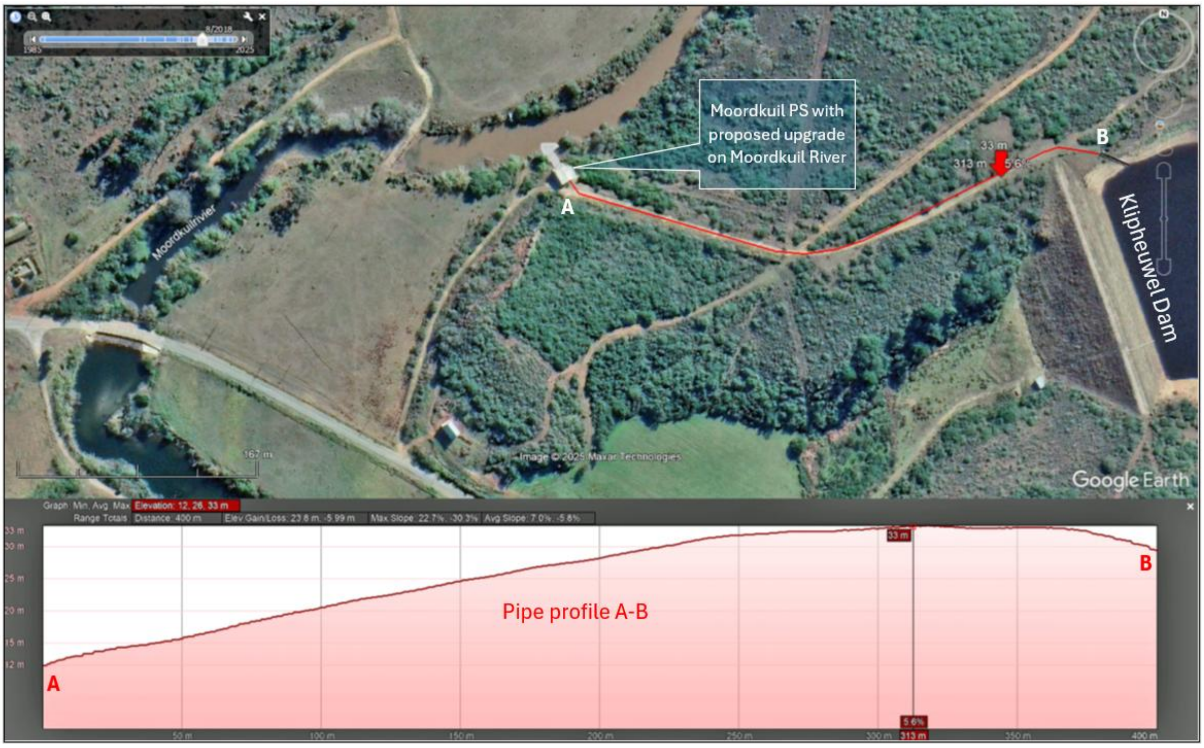

Figure 1-1: Moordkuil Pumpstation with proposed upgrade on the Moordkuil River. The location and profile of the rising main (A-B) are also shown

LYNERS (Pty) Ltd appointed ASP Technology (Pty) Ltd during May 2025 to perform the following tasks:

a) Compare the 2025 and 2014 underwater surveys and make recommendations on the intake location and upstream obstructions in the river (bedrock and man-made wall).
b) Review the proposed hydraulic design of the intake works: pump selection, minimum submergence required, pump bay space required to limit sediment deposition, dividing wall between pumps, jetpump selection with motive pump/or tap-off from high lift pump-pipe system; trashrack openings to protect the jet pumps; hopper design and size to deposit fine sand; floating debris control; cleaning of trashracks, sheetpile soil anchors, etc.
c) 3D CFD modelling with FLOW3D-HYDRO of the hydraulic forces on the sheetpile and piling intake works: 2 scenarios: i) intake with sediment not scoured from between the piling; ii) with sediment scoured from between the piling:

- Setup model: convert bathymetric survey and structure to appropriate format, boundary conditions, allocate reference points for forces
- Simulations: 2D model of large domain followed by hybrid 3D model, repeat for two scenarios
- Post-process data and produce figures/tables of water levels along the sheet piling and piling, pressure plot of sheetpile wall, fluid force values at reference points over time for two scenarios

This report presents the results of the above required tasks. Information available to ASP is first presented followed by relevant results and recommendations.

2. Comparison of the 2025 and 2014 underwater surveys and recommendations on the intake location and upstream obstructions in the river

2.1. Comparison of the 2025 and 2014 underwater surveys

The 2025 and 2014 local surveys at the proposed abstraction works were compared to identify any scour and deposition that occurred in the river within the elapsed period of 11 years, which might affect the original proposed intake location of the upgraded abstraction works. The 2014 survey was subtracted from the 2025 survey, see Figure 2.1-1, the positive values on the legend indicate deposition and the negative values on the legend indicated scour. Scour was observed on the outside bend of the river and deposition on the inside bend of the river as expected, however the upstream rock obstruction at the left bank caused deposition further downstream where the proposed intake works is located. This deposition will influence the effectiveness of the abstraction works negatively, due to the area not being able to scour properly during floods and keep the intake area clean and clear from deposited material. The rock should be removed.

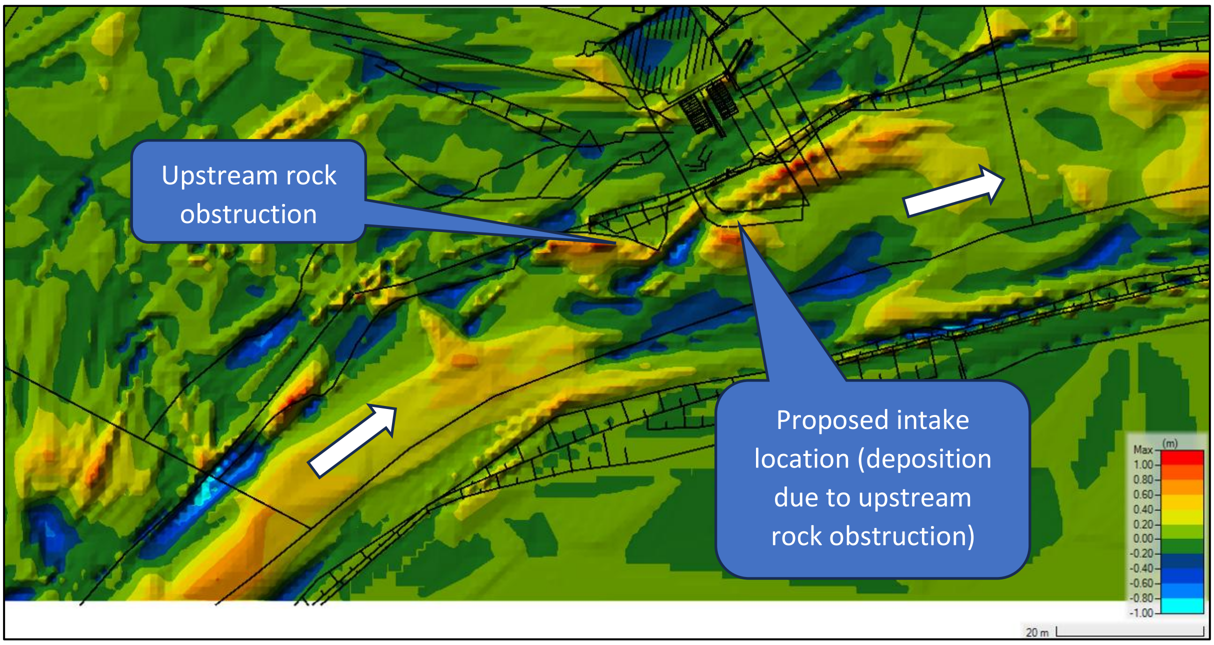

Figure 2.1-1: Difference between 2025 and 2014 surveys of the proposed Moordkuil Pumpstation with proposed upgrade on the Moordkuil River. (Red legend = deposition, blue legend -= scour)

2.2. Recommendations on the intake location and upstream obstruction

The intake location of the proposed abstraction works is located at the lowest point in the river, with the downstream low water bridge (that was closed off to prevent saltwater from pushing back up in the river due to ocean tides) that creates the control in the river at an approximate elevation of 1.950 masl. The intake is located on the outside bend of the river with an angle of approximately 19 degrees to the flow direction (left bank used as reference line). Figure 2.2-1 shows an isometric view looking at the abstraction works from upstream. The invert level of the intake is located at 0.600 masl and the soffit of the intake is located at 1.100 masl, ensuring that the intake is submerged below the minimum operating level.

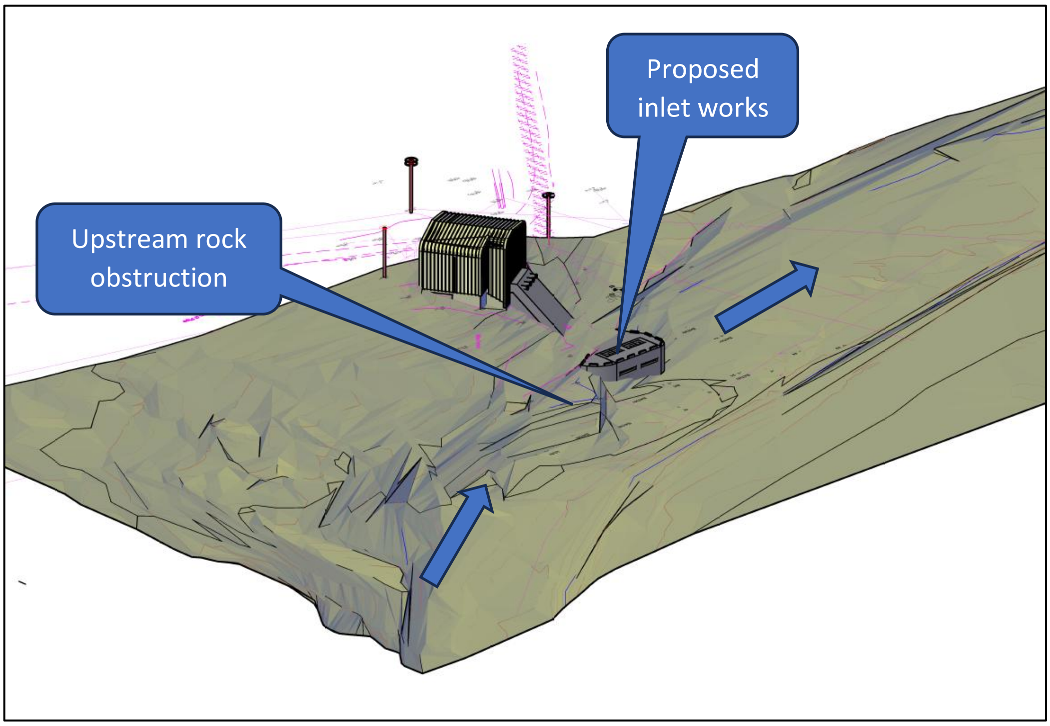

Figure 2.2-1: Isometric view looking at the proposed abstraction works from upstream (2025 survey)

Figure 2.2-2 shows a descriptive cross section summary of the current proposed intake work elevations and key dimensions. The summary of current proposed intake details is shown below:

- Proposed abstraction required = 0.8 m^3/s
- Minimum operating water level (downstream bridge) = 1.950 masl
- Invert level = 0.600 masl
- Soffit level = 1.100 masl
- Level of proposed abstraction deck = 2.500 masl
- Natural ground level @ middle of intake = -1.0 masl
- Intake opening invert above ground level = 1.6 m
- Top of hopper level = 0.400 masl
- Invert level of hopper inside = -1.250 masl
- Intake opening height = 0.500 m
- Width of each opening = 3.0 m
- No of intakes = 2

- Total effective opening inlet area = 3 m^3
- Flow velocity = 0.267 m/s (target = 0.300 m/s with trashracks/screens unblocked)

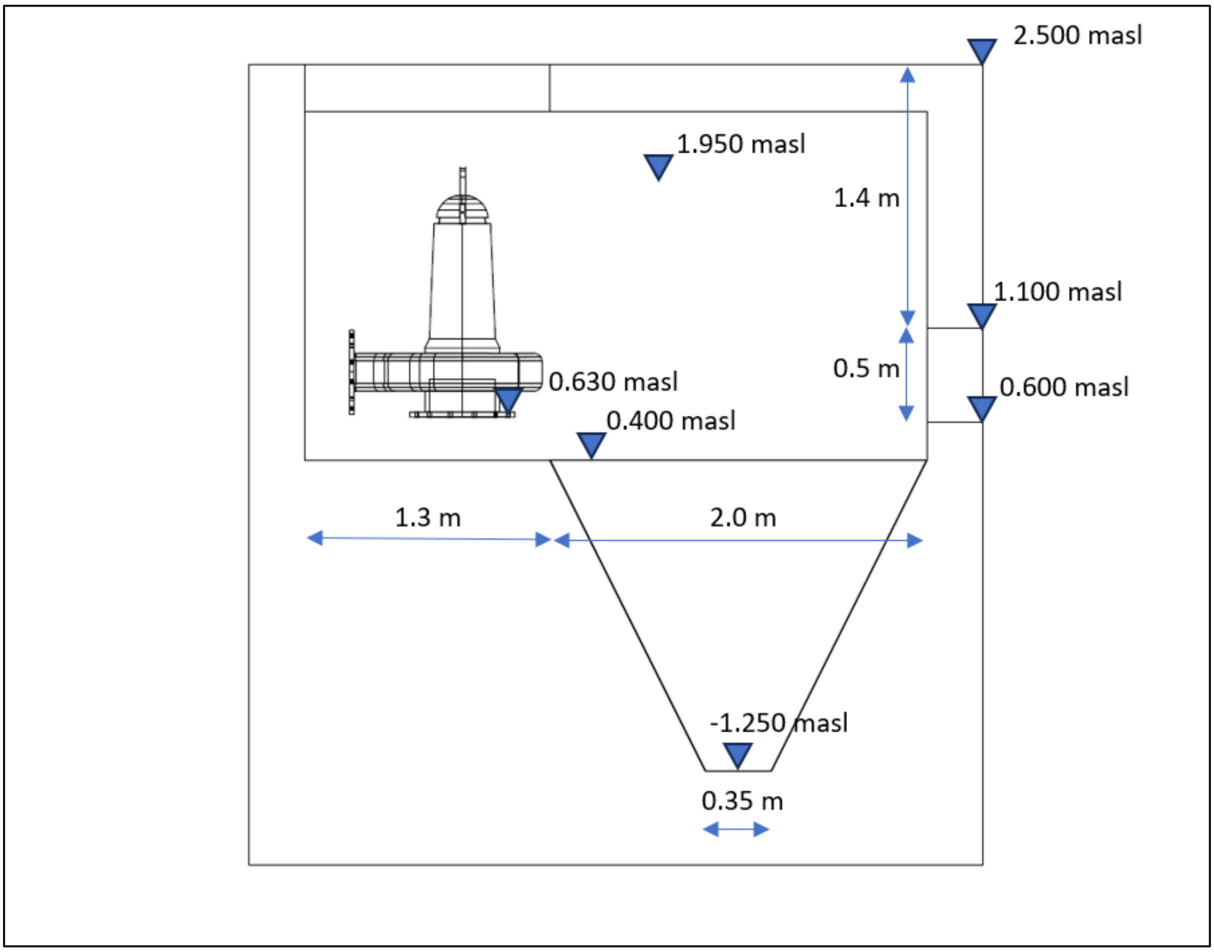

Figure 2.2-2: Example of section view indicating current proposed intake work elevations and key dimensions

The position of the intake is located at a good position with a few minor adjustments that need to be made. The following are recommended at the intake works:

- Proposed abstraction required = 0.8 m^3/s
- Minimum operating water level (downstream bridge) = 1.950 masl
- Enlarge each hopper to 4 x 4 m in plan (approximately 4 m deep)
- Make width of each intake opening 4.0 m from previous 3.0 m
- No of intake openings = 2
- Total width intake openings = 8.0 m
- Intake opening height = 0.527 m
- Assumed open intake area of each opening = 2.0 m^2
- Total open intake area = 4.0 m^2
- Install trashrack with 40 x 40 mm flat grid bars spaced at 50 mm centre to centre
- Closed area factor to be included in velocity calculations = 0.633
- Velocity calculations for opening height of 0.527 m and 4.0 m width each:
    - Q = A x V x Closed Factor

- 0.8 = (8.0 x 0.527) x V = 0.633
- V = 0.300 m/s (blocked scenario = 0.600 m/s)

Figure 2.2-3 shows a descriptive cross section summary of the elevations required for the proposed changes to the inlet works. The summary of proposed abstraction inlet detail requirements is shown below:

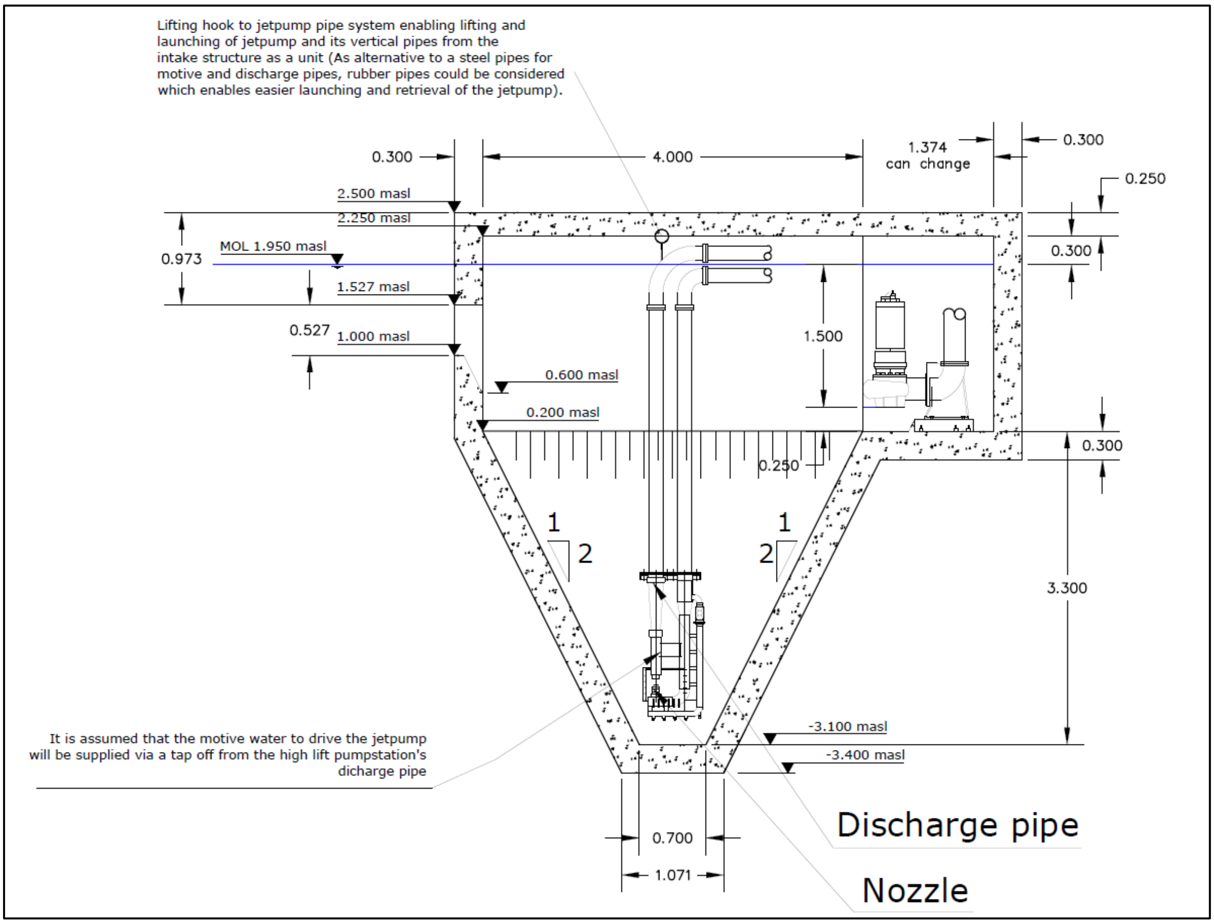

Figure 2.2-3: Example of section view indicating possible changes required on widths and elevations

- Minimum operating water level = 1.950 masl (deck of low water bridge downstream)
- Width of each intake opening = 4.0 m
- Height of each intake opening = 0.527 m
- Screens to be added 40 x 40 mm flat grid bars spaced at 50 mm centre to centre
- Submerge soffit of inlets at least 0.3 m under the MOL (1.650 masl) (0.550 m higher than previous design)
- New intake soffit level = 1.527 masl
- New intake invert level = 1.00 masl
- Top of hopper level = 0.200 masl (0.200 m lower than previous design)
- Invert level of hopper on inside = -3.100 masl (1.850 m lower than previous design)
- Natural ground level @ middle of intake = -1.0 masl
- Intake opening invert above ground level = 2.000 m

The inlet position of the proposed abstraction works should stay the same. The change to the hopper size should be made to the downstream end of the existing proposed design and also moved further inland, which will affect the pipe layout. The upstream rock obstruction should also be removed as indicated in Figure 2.2-4 (up to a level of at least -0.5 masl). The area identified to be excavated will yield a total volume of approximately 300 m^3 material that needs to be removed.

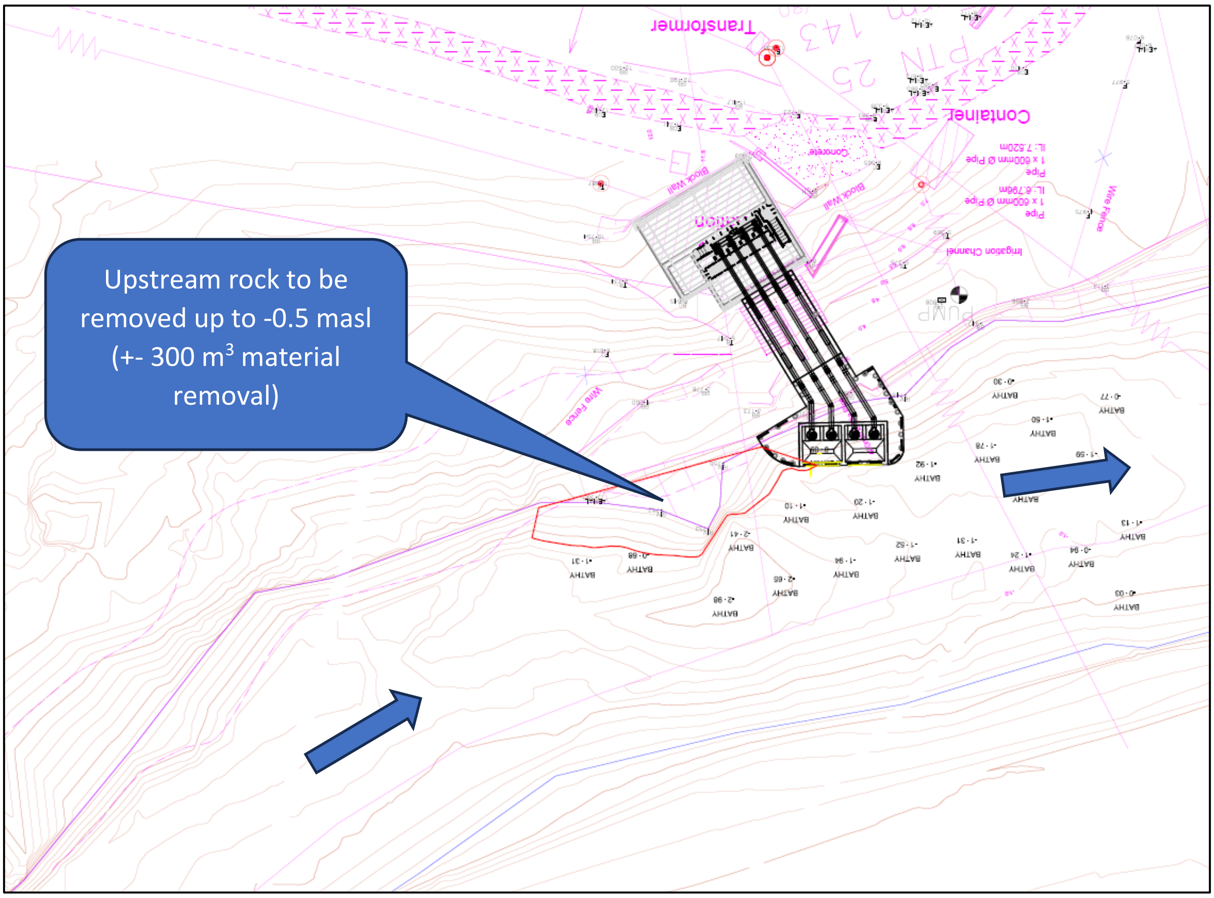

Figure 2.2-2: Plan view of proposed abstraction works on 2025 survey that indicate the upstream rock obstruction that needs to be removed

The minimum operational level of the pumps for the proposed recommended changed intake works is located at 1.950 masl. When the water level in the pool from which the abstraction is done drops to a level below 1.950 masl, the pumps should stop working.

3. Review of the proposed hydraulic design of the intake works

The LYNERS proposed intake works (low level option) in the river (adjacent to the existing pumphouse on the left riverbank) is illustrated in Figures 3-1(a and b) below. The footprint in which the river intake works should be located is shown in Figure 3-2.

Figure 3-1a: Illustration of the low-level intake works (LYNERS, 2016)

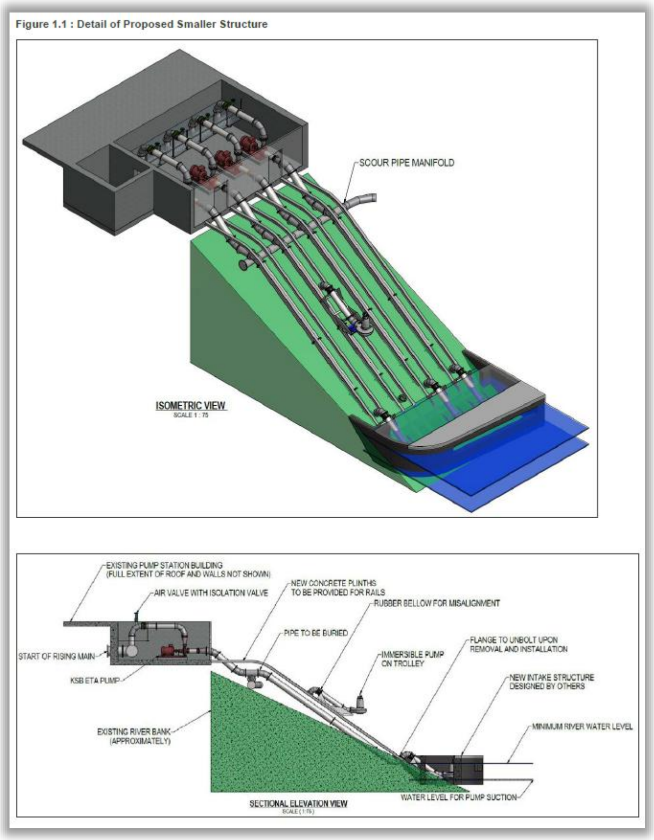

Figure 3-1b: Illustration of the low-level intake works (LYNERS, 2016)

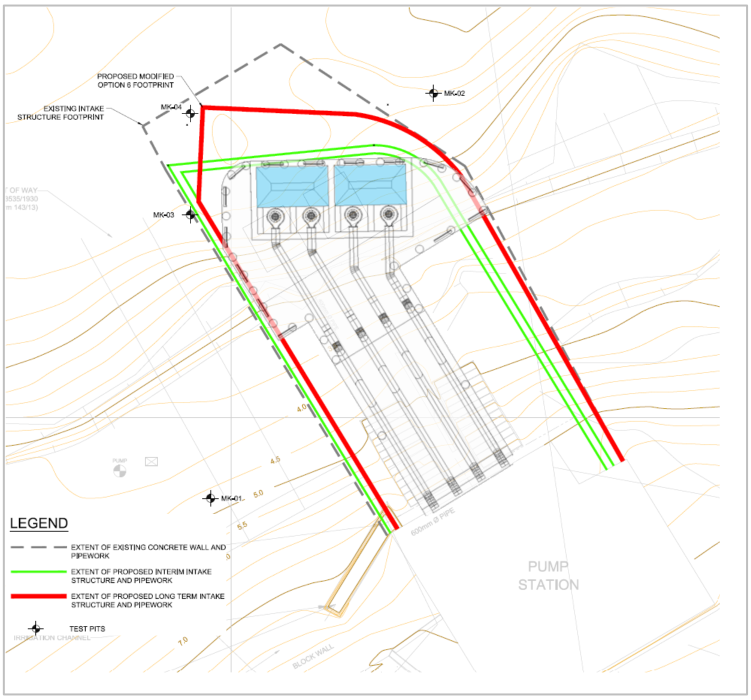

Figure 3-2: Illustration of the low-level intake works (LYNERS, 2016) to be accommodated within the red and green boundaries

Based on the principle of the LYNERS concept design for the river intake works as shown in Figure 3-2, the following modifications are recommended as shown in Figure 3-3:

- The size of the hoppers should be increased to a longer flow path to the pumps enabling the courser fraction (sizes larger than 0.4 mm) to settle in the hopper.
- To provide a larger hopper volume to ensure a less frequent sediment removal from the hopper.

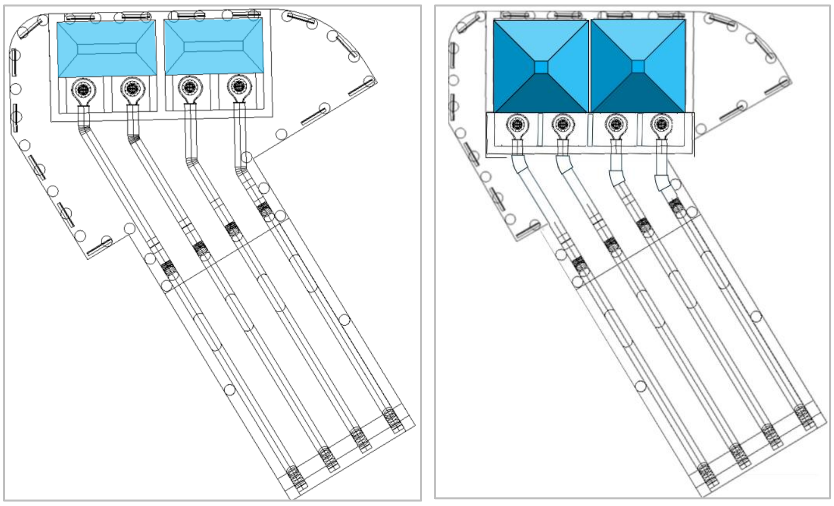

Figure 3-3: Pump and hopper layout by LYNERS (left) and that of ASP (right) and existing intake footprint and possible modification of proposed intake footprint: LYNERS dwg no 13012-c-000-a rev B (right)

It is proposed that the hoppers should be enlarged to 4m x 4m each with hopper slopes of 1H:2V. No dividing wall between hoppers should be present for flexibility of pump operation. The 4m x 4m pump bay should be large enough to also accommodate a motive pump (refer Section 7) to drive the proposed jet-pumps in the hoppers for sediment removal.

4. Selection of raw water pumps

Submersible sewer-type pumps are recommended for the raw water pumps because of their robustness and capability to pump debris and sediment with the appropriated selected impeller material (e.g., grey cast iron). LYNERS indicated that pumps for the proposed works have already been purchased. It is assumed that the submersible pumps were intended to be located in the proposed improved river intake works and that these pumps would deliver to pumps (in the existing pumphouse) which would act as booster pumps to deliver water to the Klipheuwel Dam via an existing 600 diameter pipe about 420 m long (refer Figure 1-1 for the location and profile of the rising main and Table 4-1 and Figure 4-1 for the system curves of the rising main). These purchased pumps are:

<u>Submersible pumps to be accommodated in the proposed improved river intake works</u>:

- Four FLYGT NP 3202 LT by XYLEM (refer **Appendix A** for technical detail)

<u>Booster pumps to be accommodated in the existing pumphouse</u>:

- Four LOWARA NSC 250-315 pumps by XYLEM (refer **Appendix B** for technical detail.

The above submersible type pumps are suitable for pumping raw river water containing some sediment and debris. However, the booster type pumps are less suitable to pumping raw water containing some sediment and debris. In addition, for a pumping system with a relative low pumping head (such as for this project with a static head of about 30m) it is considered that the pumping capacity of the raw water pumps in the river works should be selected such that they can pump directly (without the assistance of a booster pump) to the Klipheuwel Dam. This will significantly simplify the operation and consequently increase the reliability of the pumping system.

Based on the latter reasoning, four KSB KRT 200-402 pumps are recommended with two alternative cases (refer Figure 4-2 for the performance curves of the recommended KSB pumps):

- Case 1: Four duty pumps with no standby with each delivering 200 l/s for a total discharge of 4 x 200 l/s = 800 l/s. Figure 4-3 shows the duty flow rates per pump and head range for this case.
- Case 2: Three duty and one standby (for improved reliability) with each delivering 266 l/s for a total of 3 x 266 l/s = 800 l/s. Figure 4-3 also shows the duty flow rates per pump and head range for this case.

Cooling jackets are recommended for the recommended submersible pumps for additional protection. Variable frequency motors are recommended to operate at their best efficiency both during normal river flow and flood flow conditions as shown in Figure 4-3.

Table 4-1: Calculations of the system curves of the rising main as presented in Figure 4-1

Waterlevel at intake end of pipe 1 =	2	(masl)
Waterlevel at discharge end of last pipe =	33	(masl)
Static head =	31	(m)

Q	Dens, ρ	Dyn Visc., μ	Kin viscos, υ	Dia, D	Length, L	ε	ε/D	Veloc., V	Re= V*D/υ	Darcy Weisb. f	hL in pipe	Turb. Shear stress	sum of k's ; minor losses	hL minor	Loss Sub-total 1	Total head
m^3/s	kg/m^3	kg/(m.s)	m^2/s	m	m	m	-	m/s	-	-	m	N/m^2	Number	m	m	m
0.100	1000	0.00112	0.00000112	0.6000	420	0.0000450	0.000075	0.35	1.89E+05	0.016	0.07	0.26	3	0	0	31.00
0.200	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	0.71	3.79E+05	0.015	0.26	0.92	3	0.08	0.34	31.34
0.300	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	1.06	5.68E+05	0.014	0.56	1.96	3	0.17	0.73	31.73
0.400	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	1.41	7.58E+05	0.013	0.96	3.37	3	0.31	1.27	32.27
0.500	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	1.77	9.47E+05	0.013	1.47	5.15	3	0.48	1.95	32.95
0.600	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	2.12	1.14E+06	0.013	2.08	7.30	3	0.69	2.77	33.77
0.700	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	2.48	1.33E+06	0.013	2.80	9.80	3	0.94	3.74	34.74
0.800	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	2.83	1.52E+06	0.013	3.62	12.66	3	1.22	4.84	35.84
0.900	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	3.18	1.71E+06	0.013	4.54	15.89	3	1.55	6.09	37.09
1.000	1000	0.00112	0.00000112	0.600	420	0.0000450	0.000075	3.54	1.89E+06	0.012	5.56	19.47	3	1.91	7.47	38.47

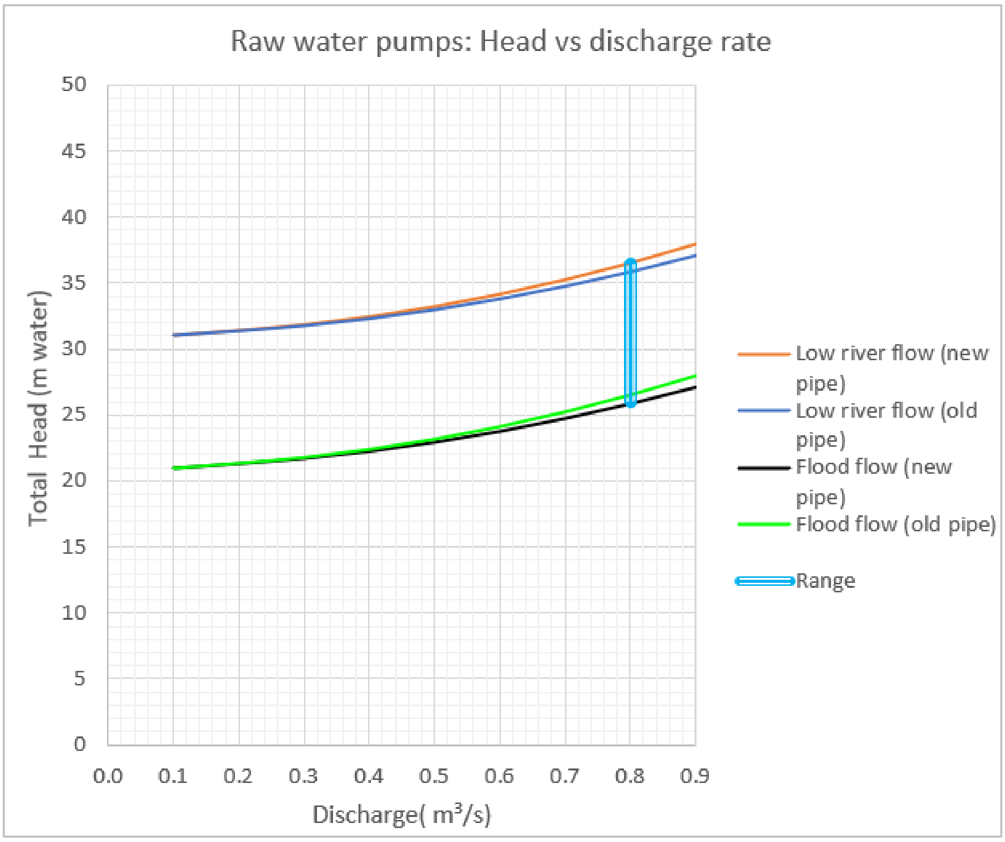

Figure 4-1: System curves of the rising main during low river flow and floods with the required operating range at 800 l/s

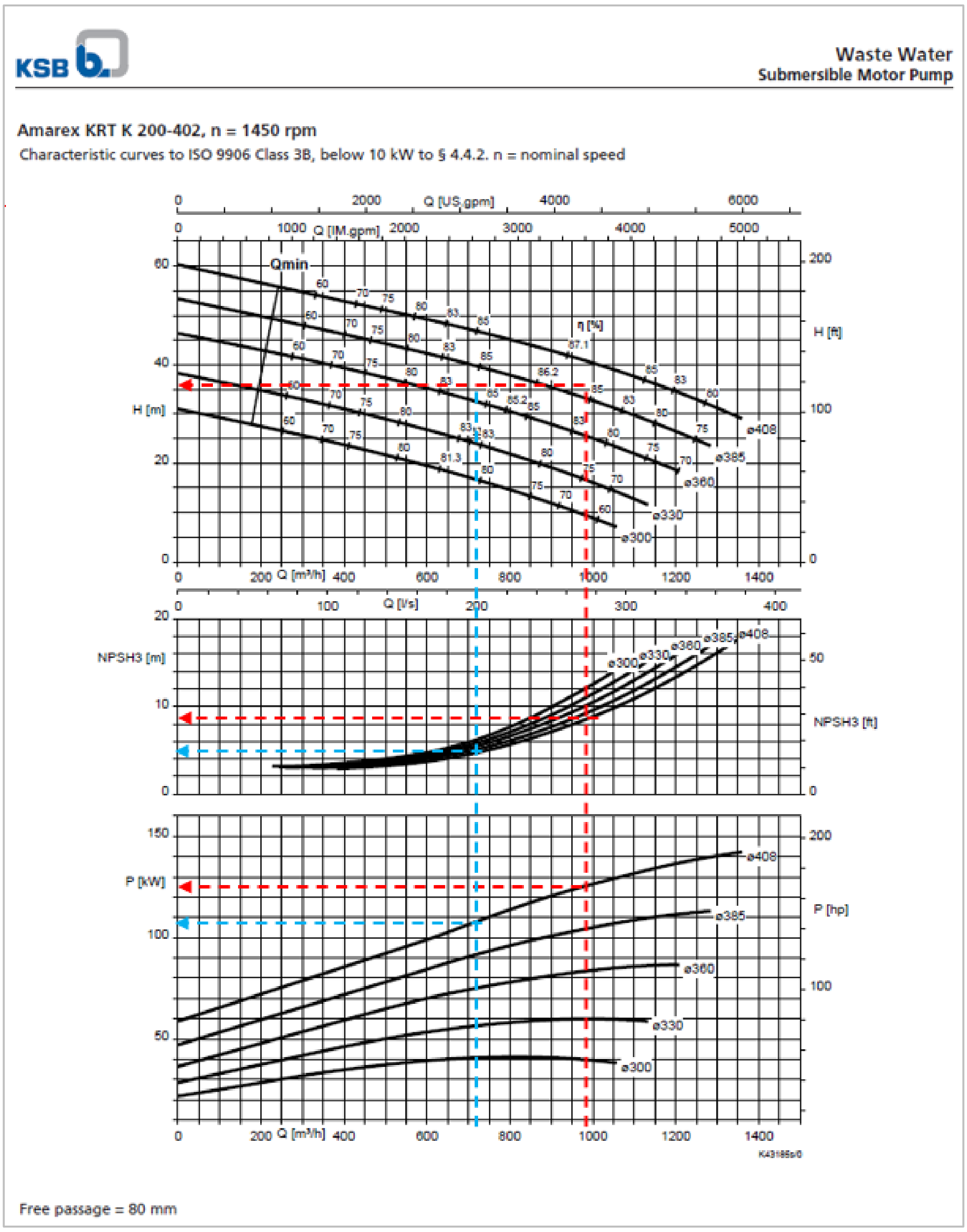

Figure 4-2: Performance curves of the recommended KSB raw water pumps indicating duty conditions for the case of 3 duty and 1 standby at 266 l/s per pump and for the case of 4 duty and no standby at 200 l/s per pump for a total delivery of 800 l/s for both cases.

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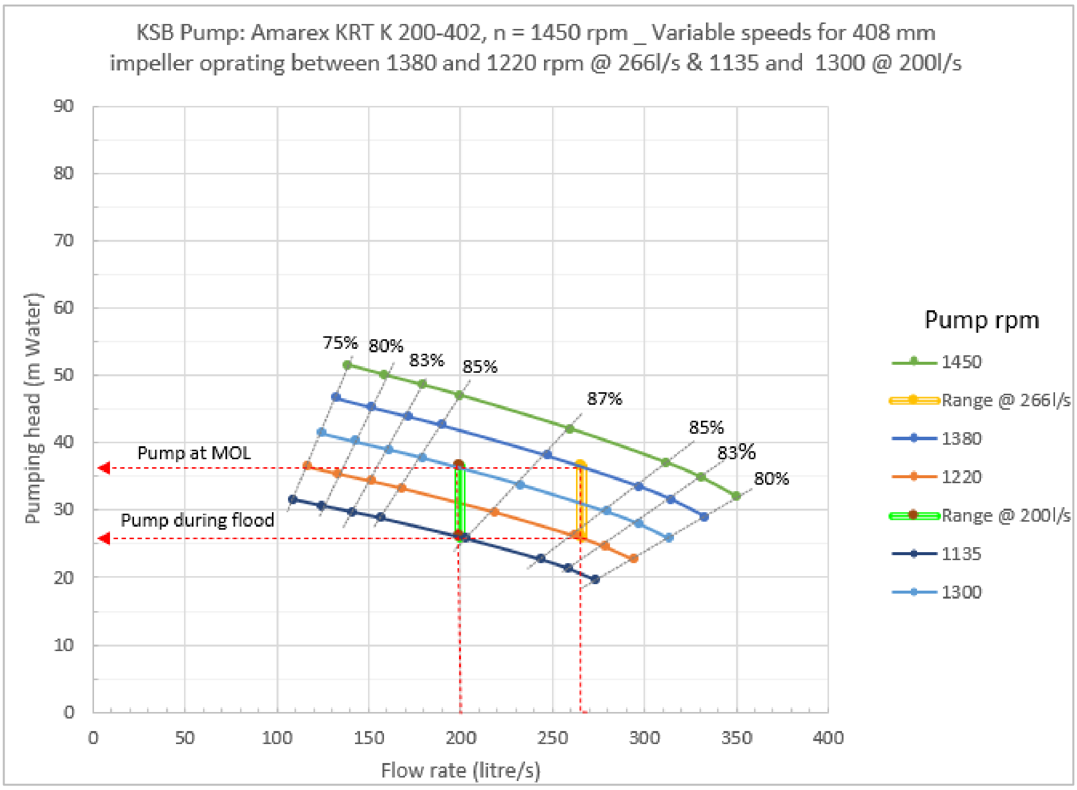


Figure 4-3: Operating ranges for the two possible duty ranges of the proposed KSB pumps to provide for variable river water levels.

5. Required submergence for raw water pumps

To prevent air entrainment (due to the tendency of vortex formation) at the pump intakes, sufficient submergence is required. The relevant excerpts from The American National Standard for Pump Intake Design, (ANSI/HI 9.8, 1998), which are shown in Figure 5-1, were used as a guideline together with the recommended KSB pump's inlet diameter of 200mm (refer **Appendix C**). It is assumed that the pump's body diameter of 0.735m (refer **Appendix C**) above the pump's intake, will dampen vortex formation and that the body diameter will have at least the same effect as the required bellmouth diameter of 0.45 m according to Figure 5-1. With these assumptions a minimum submergence of 1.5 m is required according to Figure 5-1.

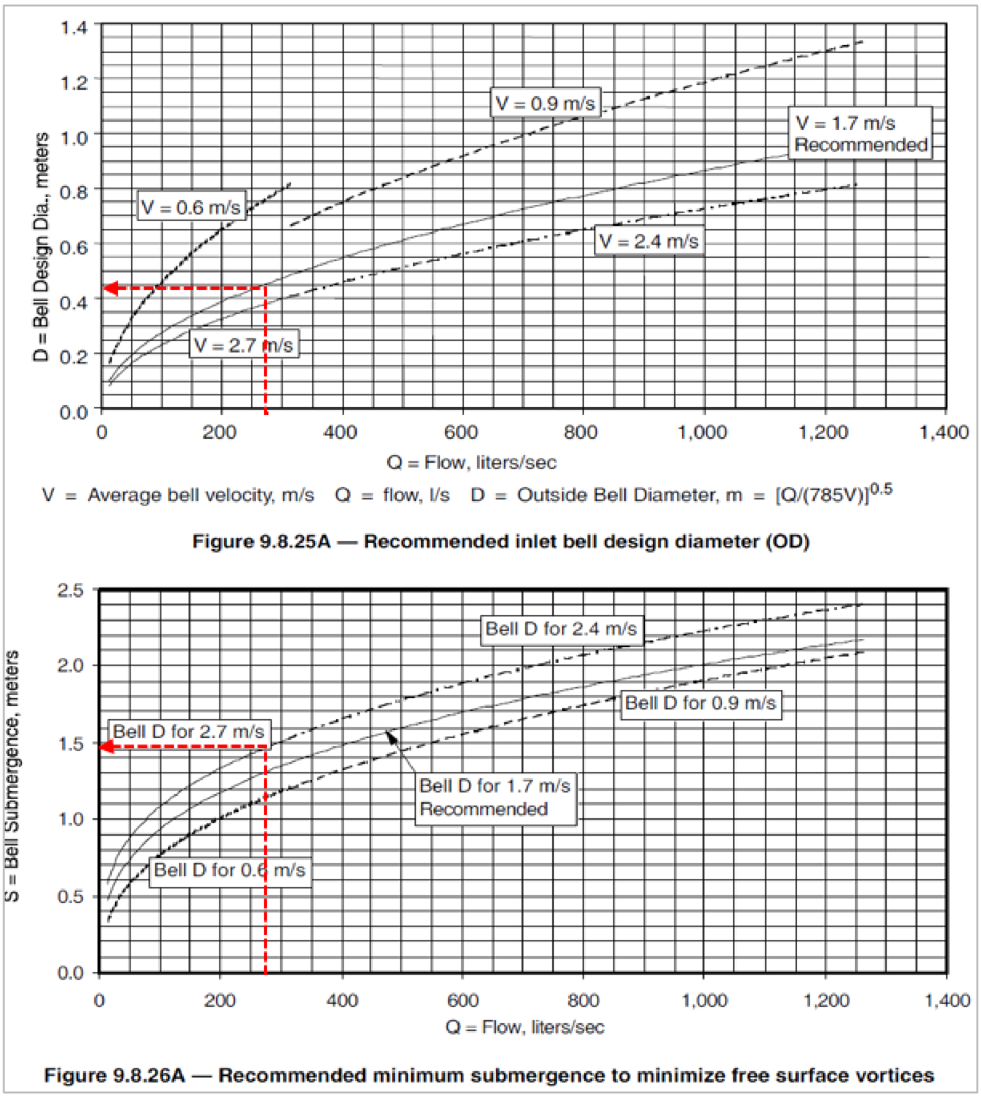


Figure 5-1: Required bellmouth diameter and submergence (ANSI/HI-9.8, 1998)

6. Required and available Nett Positive Suction Head (NPSH) for raw water pumps

The required Net Positive Suction Heads (NPSHreq) for the recommended KSB pumps are 4.5m for Case 1 and 8.5m for Case 2 according to Figure 4-2 and the available NPSH (NPSHA) values are calculated according to ANSI/HI-9.8 (1998) as shown in Excerpt 6-1. Table 6-1 shows the calculation of the Available Net Positive Suction Heads (NPSHA) for the two cases and indicates that the NPSHreq values are satisfied with the NPSHA values.

Excerpt 6-1: NPSH available after ANSI/HI-9.8 (1998)

8.3.4 Net Positive Suction Head Available

Net positive suction head available (NPSHA) is the head available above vapor pressure head to move a liquid into the impeller unit of the pump. It is necessary to ensure that the NPSHA exceeds the NPSHR to prevent cavitation. The following equation is used to compute NPHSA:

$$NPSHA = H_{pa} + H_s - h_f - H_{vp} \qquad (8\text{-}11)$$

where:

H_{pa} = the atmospheric pressure head on the surface of the liquid in the sump – m (ft).

H_s = static suction head of liquid. This is height of the surface of the liquid above the centerline of the pump impeller – m (ft)

h_f = total friction losses in the suction line – m (ft)

H_{vp} = the vapor pressure head of the liquid at the operating temperature – m (ft)

Table 6.1: Calculation of available Net Positive suction head for the two cases

Duty per pump = 200 l/s

Elevation of intake	2	masl	
Atmospheric head (Hpa)	10.3	m H2O	
Vapour pressure of water at 20°C (Hvp)	0.24	m H2O	
Friction head loss incl fall through screens	*0.15*	m	CHECK (assumed 0.2m, to confirm)
Submergence (S) of PUMP intake end at MOL - from ANSI	**1.5**	m	Based on ANSI/HI 9.8
Height of impeller suction face above intake face	0.20	m	See dimensions of selected pump
NPSHavailable= Hpa + Hs - hf - Hvp	11.2	m	
NPSHrequired from pump curves	4.5	m	See selected pump's curves
NPSHavailable / NPSHrequired	2.5	>1.3 OK	

Duty flow rate per pump = 266 l/s

Elevation of intake	2	masl	
Atmospheric head (Hpa)	10.3	m H2O	
Vapour pressure of water at 20°C (Hvp)	0.24	m H2O	
Friction head loss incl fall through screens	*0.15*	m	CHECK (assumed 0.15m, to confirm)
Submergence (S) of PUMP intake end at MOL - from ANSI	**1.5**	m	Based on ANSI/HI 9.8
Height of impeller suction face above intake face	0.20	m	See dimensions of selected pump
NPSHavailable= Hpa - Hs - hf - Hvp	11.2	m	
NPSHrequired from pump curves	8.5	m	See selected pump's curves
NPSHavailable / NPSHrequired	1.32	>1.3 OK	

7. Jet-pump selection for removal of settled sediment in hoppers

It is expected that sediment sizes of 0.4 mm and larger will settle in the hopper in the pump forebay. A jet-pump, which is permanently installed at the invert of each of the two hoppers, is proposed to remove the settled sediment intermittently. The sediment level in the hopper can be automatically monitored by means of a sensor such as the ultrasonic-type bed level sensor used in wastewater treatment works for the continuous sludge/water interface monitoring in sludge thickeners. The advantage of a jet-pump is that it has no moving parts and operates at its best when it is buried in sediment. An example of the type of jet-pump recommended is shown in Figure 7-1 and its operation in Figure 7-2 – refer also to **Appendices E and F**.

COMPONENT PARAMETER	TECHNICAL DETAIL
Production Rate	30 t/h (approx 18 m^3 bulk volume sand per hour
Supply head of motive pump	700 kPa
Supply flow rate to main jet	10 litres/second
Supply flow rate to fluidizer nozzle	2.5 litres/second
Induced suction flow rate	9 litres/second
Head at diffuser outlet	11.7 m
Mixing Chamber diameter	44 mm
Jet pump grid size at suction end	30 mm x 30 mm
Main jet diameter	19 mm
Motive pipe length	50 m
Motive pipe diameter	100 mm
Motive pipe velocity	1.5 m/s
Motive pipe friction head loss	2.5 m per 100 m pipe length
Jet pump depth below water surface	3 m
Discharge pipe length	50 m
Discharge line diameter	100 mm
Discharge pipe flow velocity	2.5 m/s

Figure 7-1: Example of a jet-pump by GENFLO used in a WRC field evaluation research project (WRC, 2002).

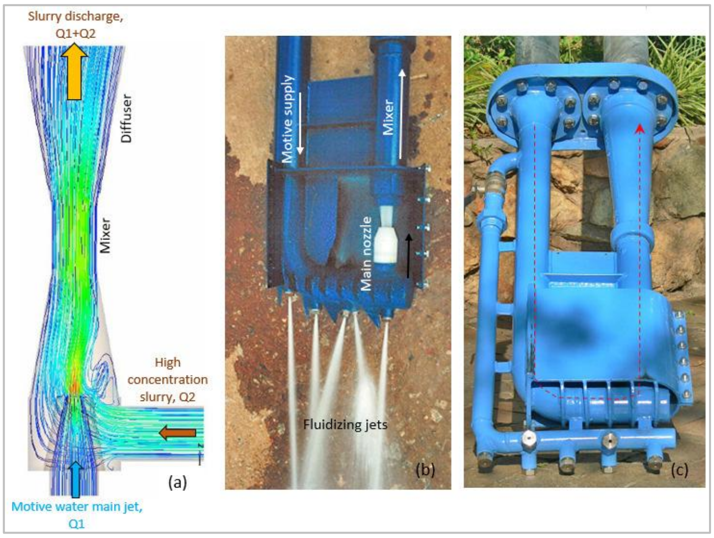


Figure 7-2: Basic operation of a jet-pump. Motive water is supplied in the form of a high velocity jet through a large main nozzle across a gap between the main nozzle and a receiving mixer where the sediment (entrained at the gap) and motive water is mixed and flows via a diffuser to a discharge pipe. The motive water also supplies fluidization water via smaller nozzles to liberate the settled sediment near the suction end of the jet-pump.

The main feature of the jet-pump recommended for the Moordkuil raw water intake is as follows:

- Sediment production rate: 38 t/hr (approx. 23 m^3/hr)
- Mixer inside diameter: 62.5 mm
- Main nozzle diameter: 31 mm
- Fluidization nozzle diameter: 5 mm
- Required motive head: 65 m
- Motive water supply rate (including 10% fluidization water): 27 + 3 = 30 l/s
- Slurry discharge rate: 37 l/s
- Discharge pipe inside diameter: 125 mm
- Motive water supply pipe inside diameter: 100 mm
- Head loss through 20 m long, 100 mm diameter motive pipe: approx. 3 m
- Head loss trough 30m long, 125 mm diameter slurry pipe: approx. 4.5 m (refer **Appendix H**)

The performance curves of a jet-pump with the features are presented in **Appendix D**. A supplier of jet-pumps in South Africa is GENFLO Dredging (https://www.genflo.co.za/). GENFLO Dredging has the software to refine the specifications of a jet-pump based on the required performance and the configuration of the civil works of a project and it is recommended that GENFLO Dredging be approached during the detail design phase for further refinement of the design and a description of its operation.

It is proposed that the motive water be supplied by a submersible pump (located in the river works with the main pumps) at a head of 65 m at the jet-pump inlet (point X in **Appendix G**). The pump selected for the jet-pump's motive pump is the submersible KSB KRT 080-315 pump – refer **Appendix J** for the technical detail of this pump. This pump should also be provided with a cooling jacket and variable frequency motor. The motive water should pass through a screen filter to ensure that the 5 mm diameter fluidization nozzles do not clog. The filter could be of the screen-type which has a relatively small head loss, similar to the type as shown in **Appendix I** and used in farm irrigation systems. The filter system can be accommodated in the existing pump house.

The motive and slurry pipes of the jet-pump could be of either rigid or flexible (hose) type. The latter is for practical reasons considered more appropriate as demonstrated in **Appendix G**. The hose-type alternative has the advantage that the jet-pump could be launched while active (to dig itself into the settled sediment towards the hopper invert). Also, it enables the jet-pumps to be inspected/serviced by lifting the jet-pump to the top of the pump bay without disconnecting the motive supply and slurry discharge pipes. Inspection of a jet-pump from time to time is necessary to establish possible clogging of the fluidization nozzles, and inspection/replacement of the main nozzle and mixer which are subjected to wearing.

8. 2D Hydrodynamic modelling with the proposed new low abstraction works intake located on the left bank protruding bedrock upstream of the existing pumpstation

Before a final decision could be made on the proposed new intake location on the protruding bedrock at the left bank upstream of the existing pumpstation, 2D hydrodynamic modelling was required with movable bed conditions to evaluate whether the proposed intake location will be self-scouring during floods, and to evaluate possible other impacts of the new low intake on the flow patterns and sediment dynamics.

8.1. Model setup

A two-dimensional model Mike 21C of the DHI group was used to simulate the flow patterns and sediment deposition and erosion near the proposed pumpstation intake.

The model was set up based on the new (2025) topographical and underwater survey data. The low water causeway and the bridge downstream of the pumpstation were included in the model. The annual recurrence interval floods shown in Figure 8-1 were routed through the river to simulate scour and deposition patterns. The model was set up considering the following:

a) Downstream boundary conditions were taken from the ASP (2014) study where the downstream water levels considered the year 2060 sea level rise. The backwater effect downstream of the 2D model bathymetry was also simulated by 1D model in the 2014 study and for low river flow conditions tidal effects were considered. The corresponding downstream water levels are shown in Figure 8-2.

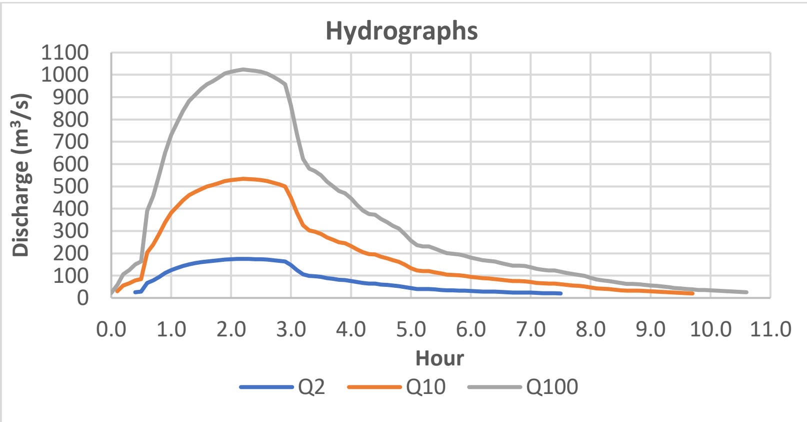


Figure 8-1: Flood hydrographs used in the model

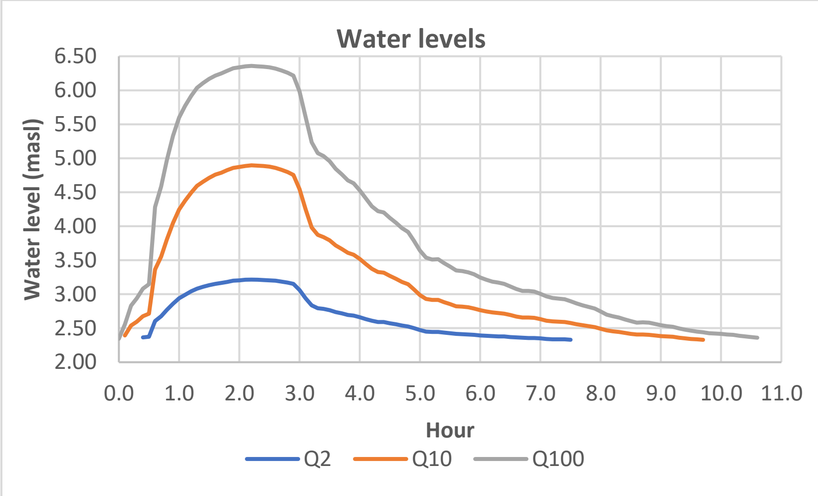


Figure 8-2: Downstream tailwater levels used in the model

b) The riverbed sediment grading was obtained from bed grab sampling and laboratory analysis (2014 study) as shown in Table 8-1.

Table 8-1: Sediment fractions used in the model

fraction no.	mm	% bed	Description
d1	0.002	7	clay
d2	0.03	23	silt
d3	0.11	40	sand
d4	0.45	20	sand
d5	4.00	10	sand

d) Manning roughness n = 0.045 was used in the main channel and n = 0.055 on the floodplains.

The surveyed 2025 bathymetry of the 2D model is shown in Figure 8-3. The elevations in the figure are shown as masl (refer to legend). The existing pumpstation and the proposed intake with a top of structure elevation of 2.5 masl are shown in the bathymetry. The surveyed reach has a bed level in the main channel below mean sea level.

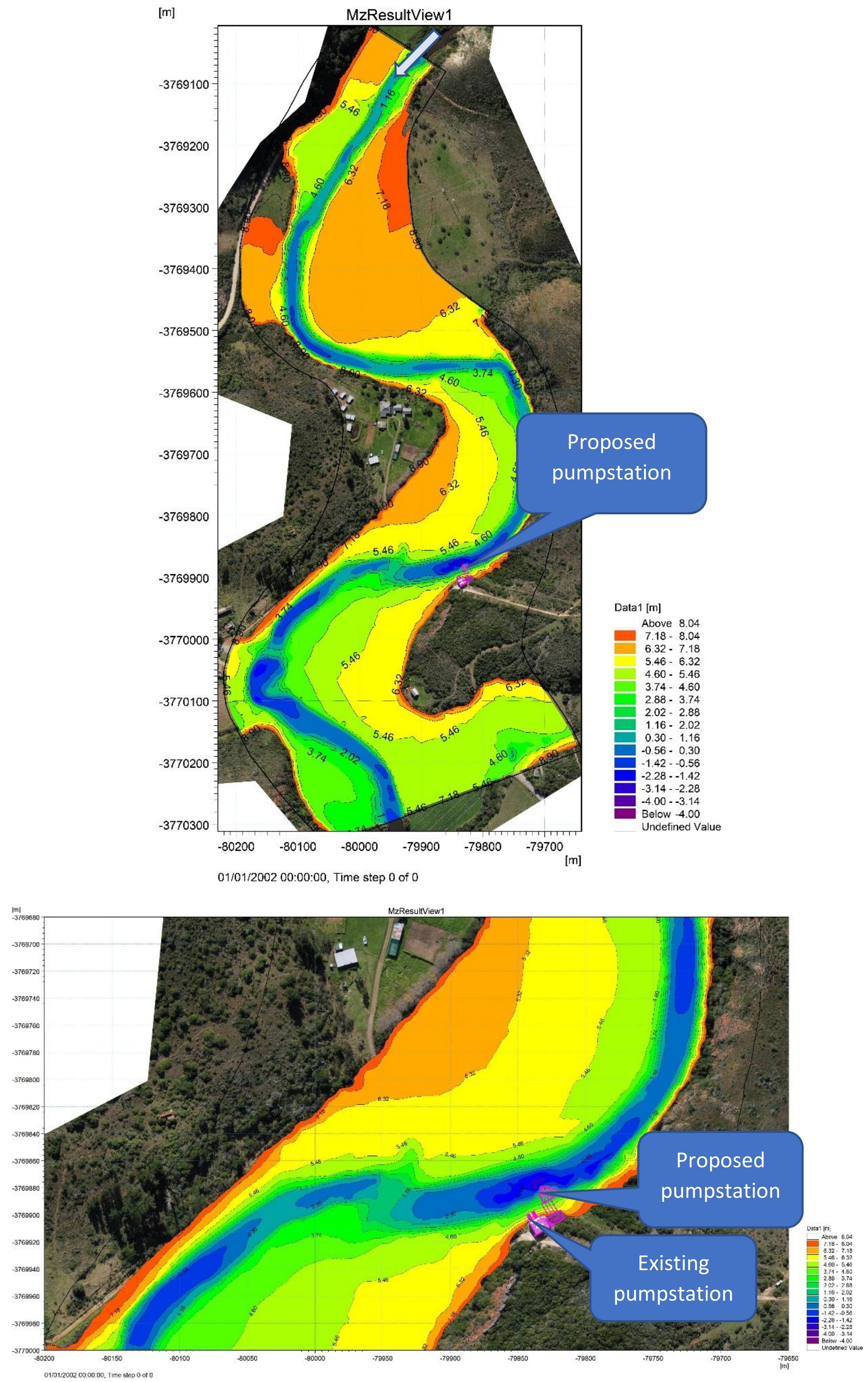


Figure 8-3: 2D model bathymetry for the current scenario based on the topographical survey (masl) (bottom picture: zoomed view near the proposed pumpstation)

The new survey shows that there is bedrock protruding from the left bank upstream of the existing intake. The proposed pump station is located on the above-mentioned protruding rock upstream of the existing pump station.

Simulations were carried out for the current scenario with the proposed new pumpstation intake added. The left bank of the river upstream of the pumpstation was made non-erodible (bedrock) in the model.

8.2. Current bathymetry with proposed new upstream intake on rock scenario

The simulated flow patterns (velocities, flow depths and water levels) are shown in Figures 8-4 to 8-12 for the 2-year, 10-year and 100-year floods respectively. The simulated velocities indicate that the pumpstation location is in a good position, with high flow velocities near the left bank which would help to scour the future pump intake during floods.

During the 2-year flood and 100-year flood the water levels 10 m upstream of the pumpstation in the are 4.2 masl and 7.6 masl, respectively.

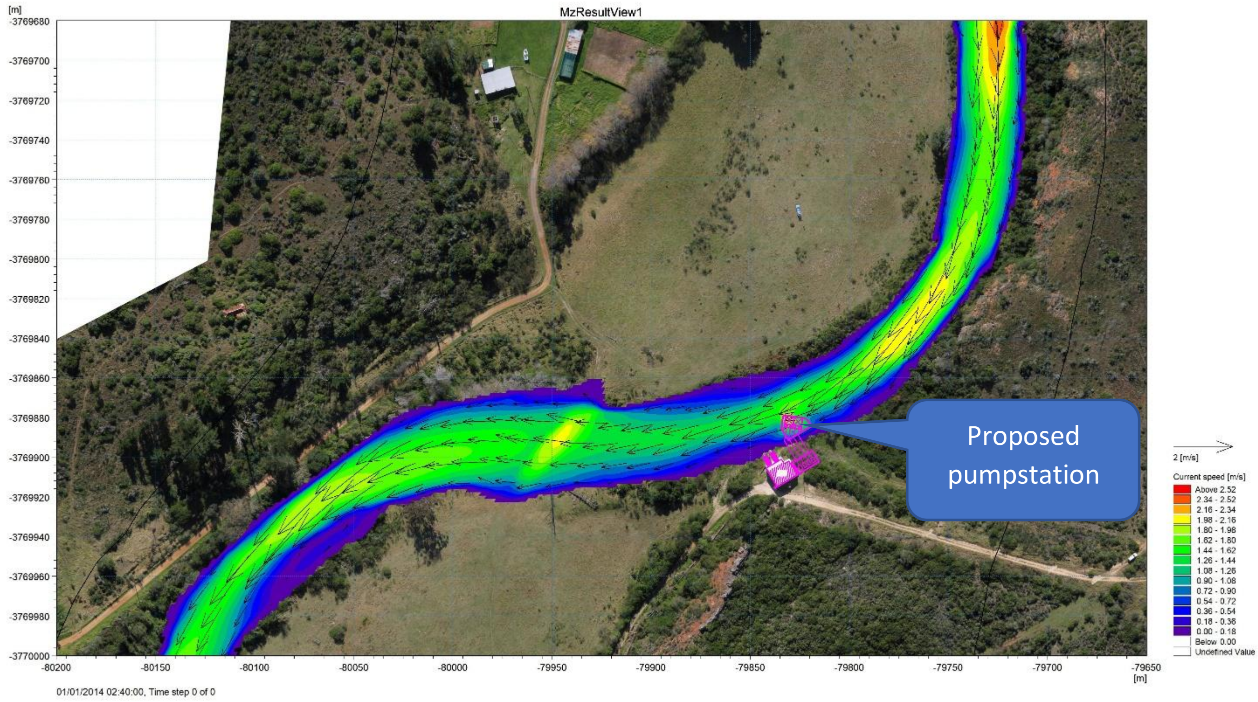


Figure 8-4: Simulated flow velocities during the peak of the 2-year flood

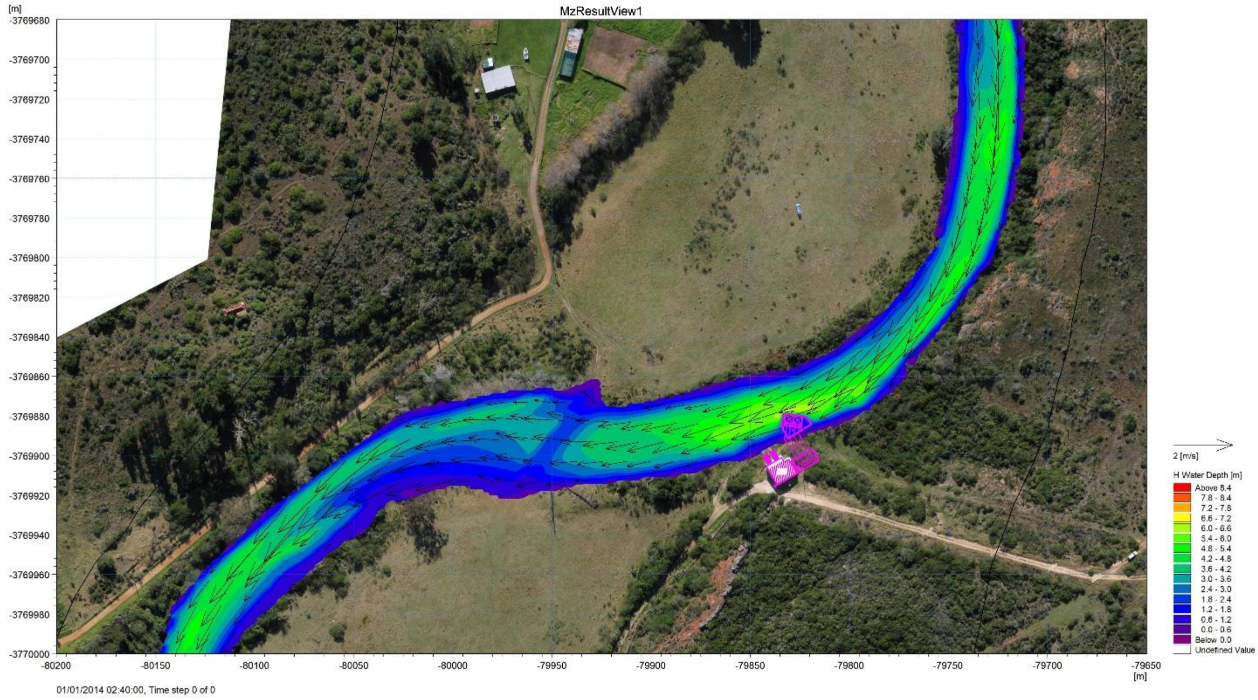


Figure 8-5: Simulated flow depths during the peak of the 2-year flood

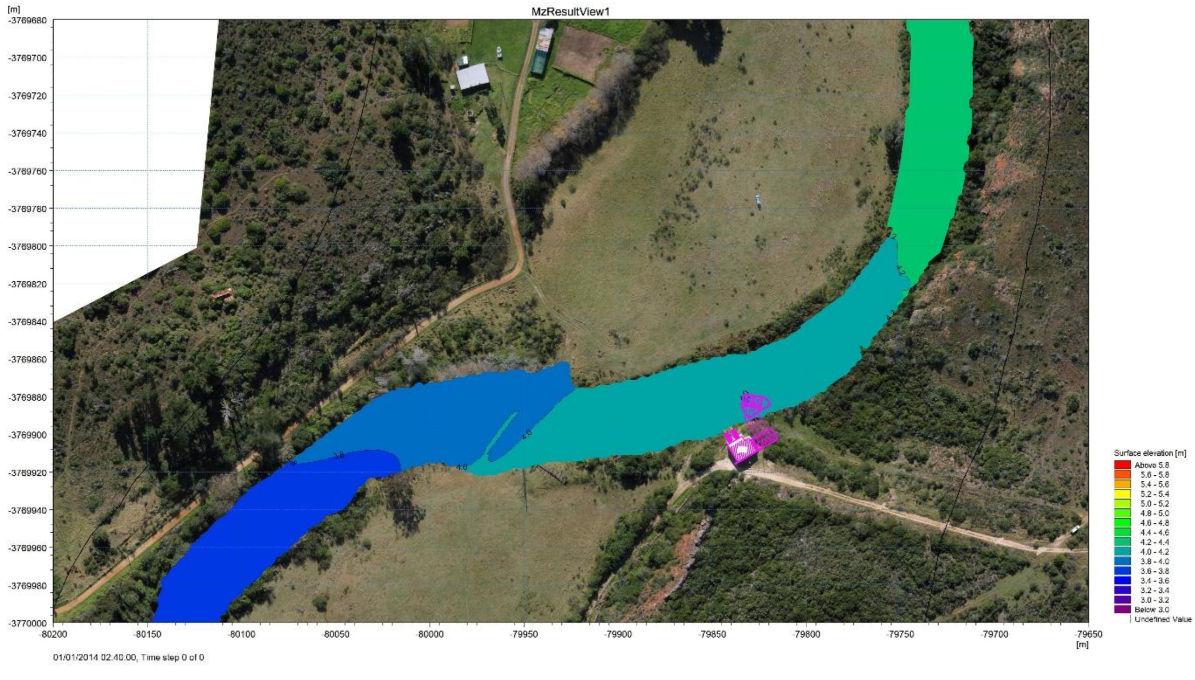


Figure 8-6: Simulated water levels during the peak of the 2-year flood (masl)

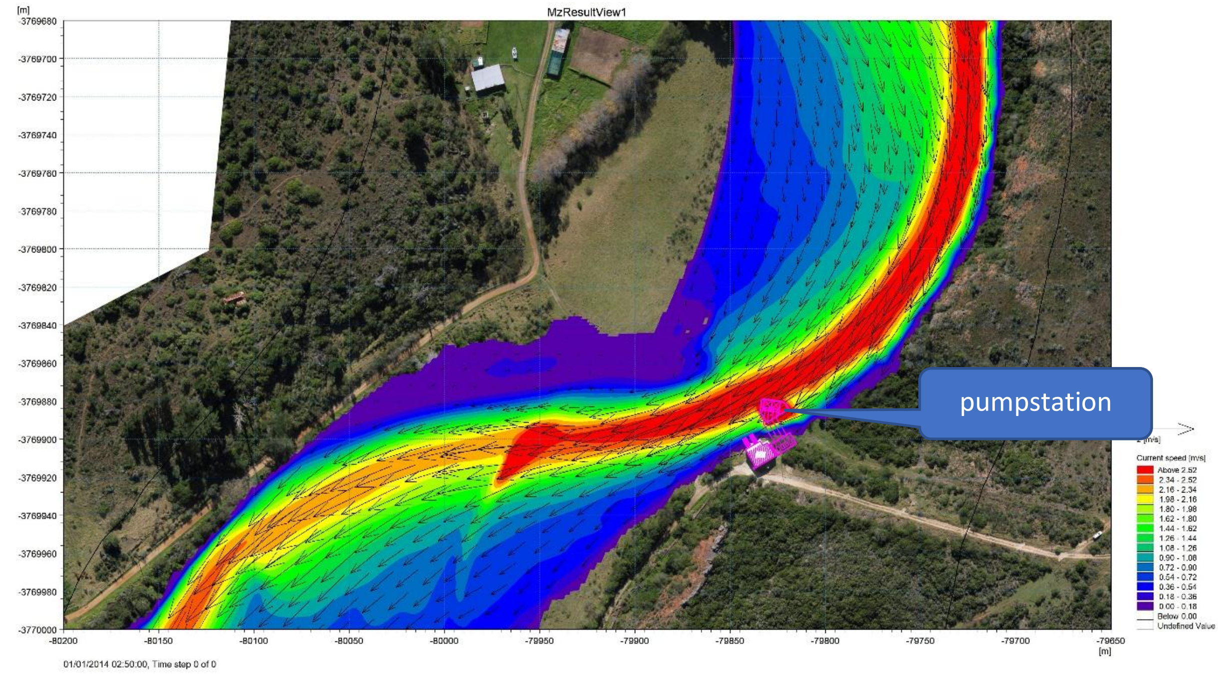


Figure 8-7: Simulated flow velocities during the peak of the 10-year flood

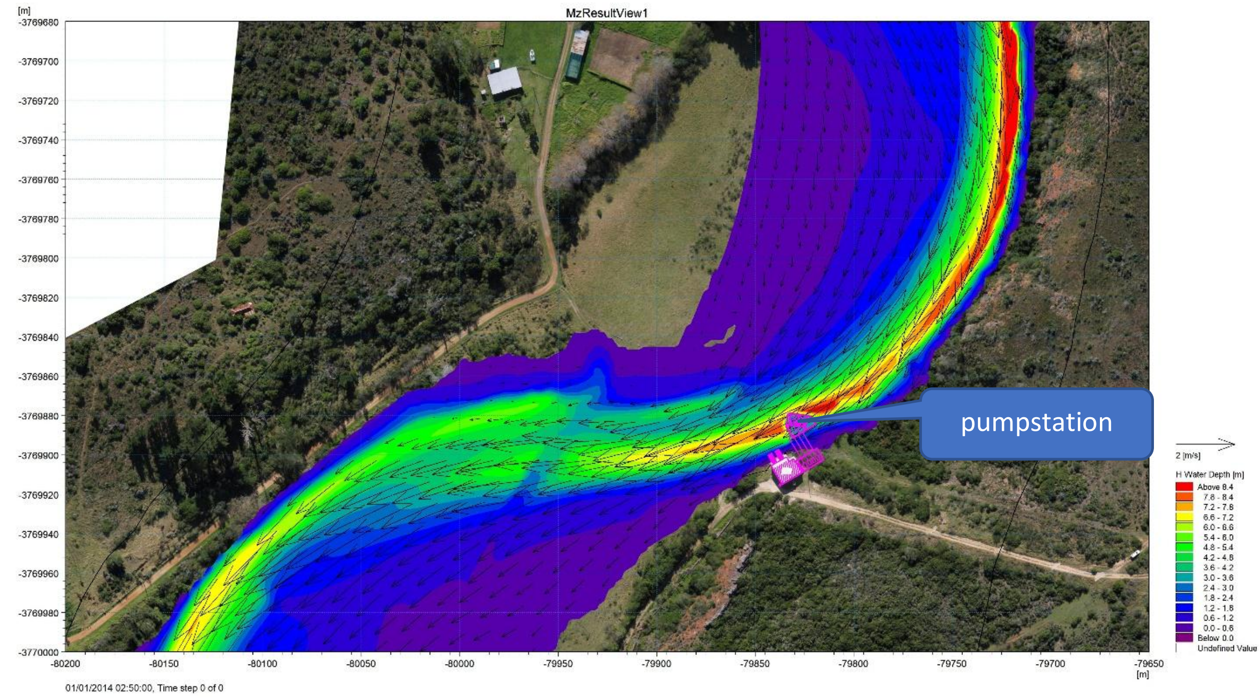


Figure 8-8: Simulated flow depths during the peak of the 10-year flood

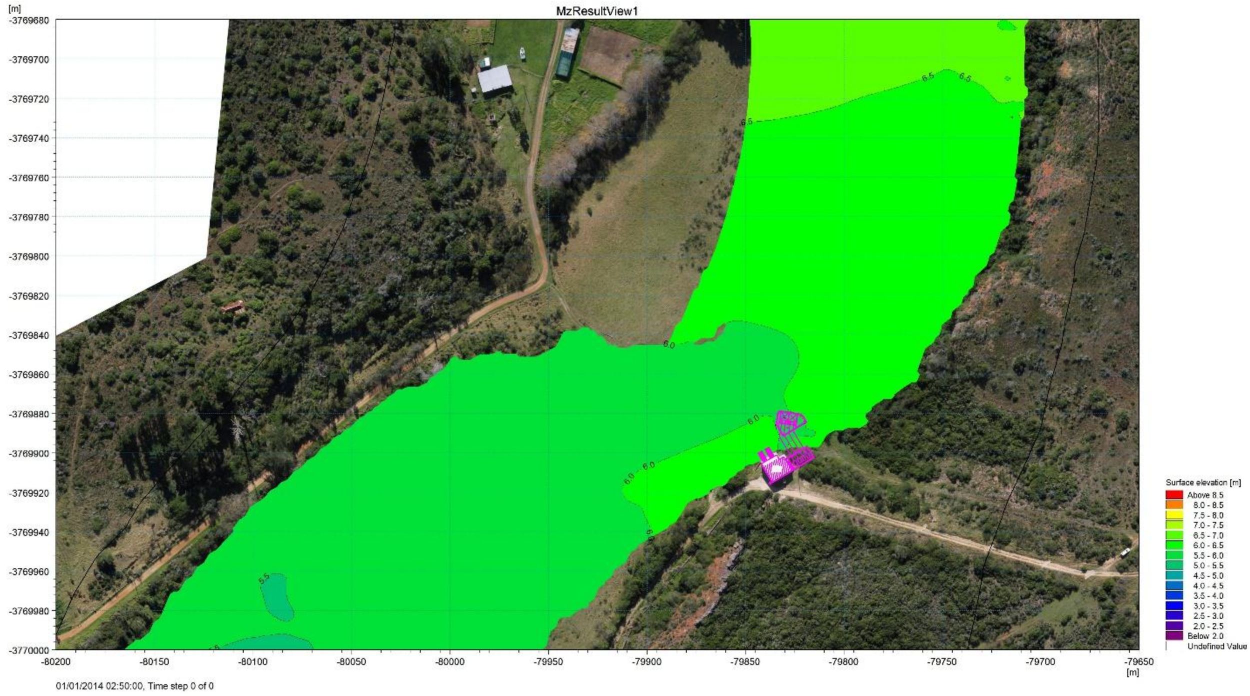


Figure 8-9: Simulated water levels during the peak of the 10 year flood

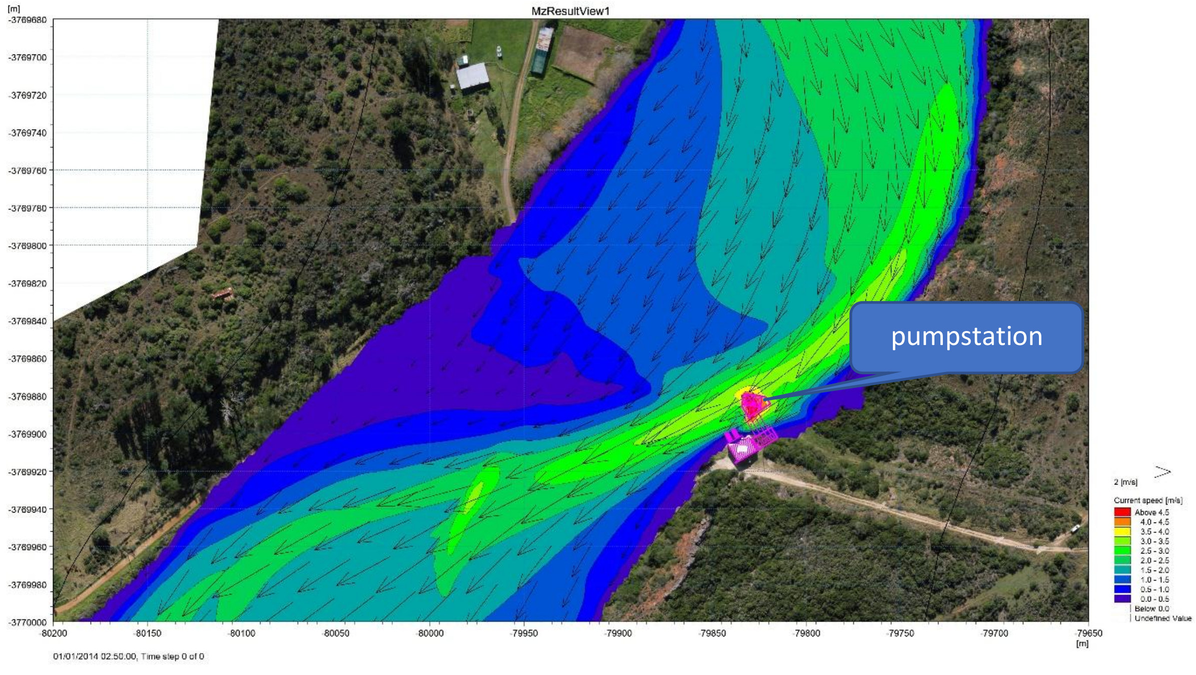


Figure 8-10: Simulated flow velocities during the peak of the 100-year

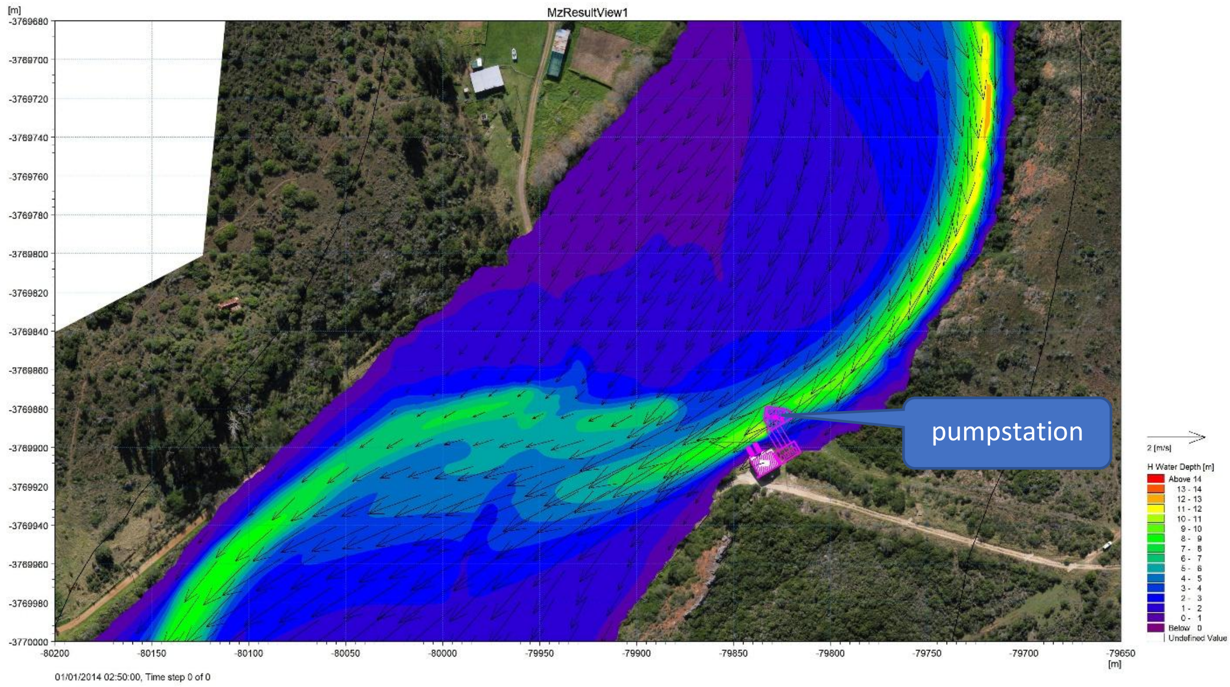


Figure 8-11: Simulated flow depths during the peak of the 100 year flood

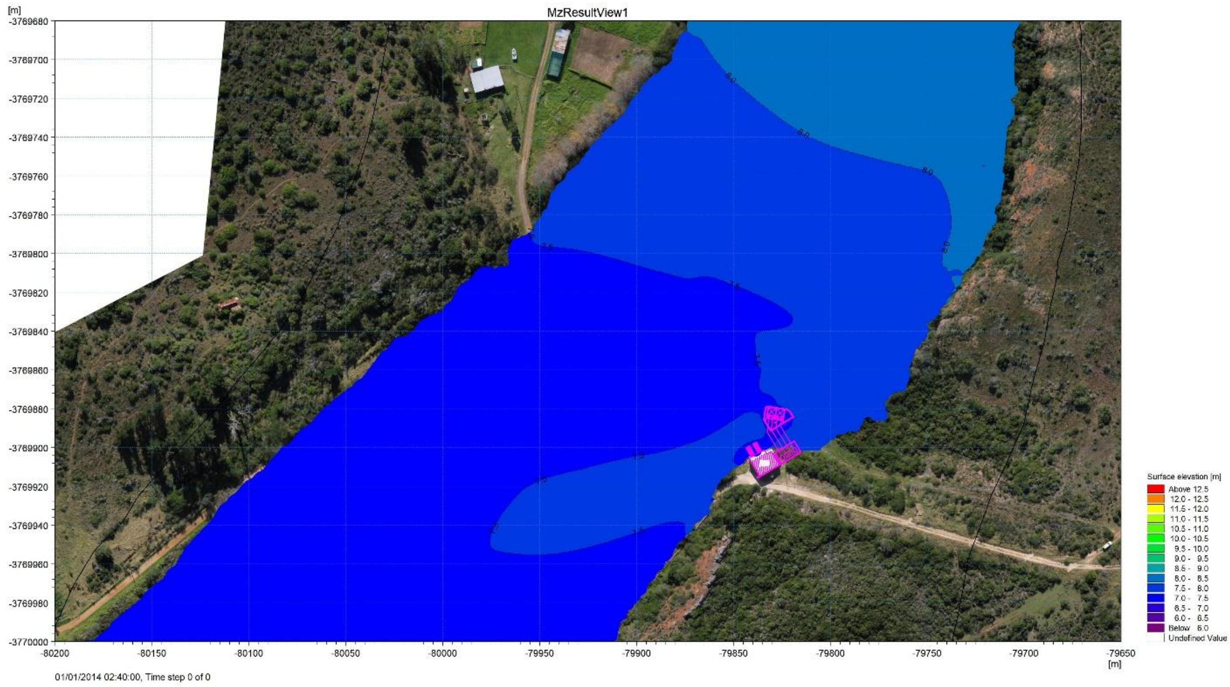


Figure 8-12: Simulated water levels during the peak of the 100 year flood

Figures 8-13 to 8-18 show the simulated bed levels and bed level change following the 2-year, 10 year and 100-year floods, respectively. In these simulations the causeway and the approach roads were specified as non-erodible in the model. The left bank of the river upstream of the pumpstation was also specified as non-erodible (bedrock). During all the floods sediment deposits (positive depths) at the inside of the river bend opposite the pumpstation. At the proposed pump intake during a 2-year flood the bed scoured between 1 m to 1.8 m deep. During larger floods (10-year flood and 100-year flood) the model simulated a large amount of sediment deposit opposite to the abstraction intake closer to the right bank. However, the area near the proposed abstraction intake scoured 1.30 m deep during 10-year flood and 0.90 m deep during the 100 year flood. More simulation results are provided in **Appendix K**.

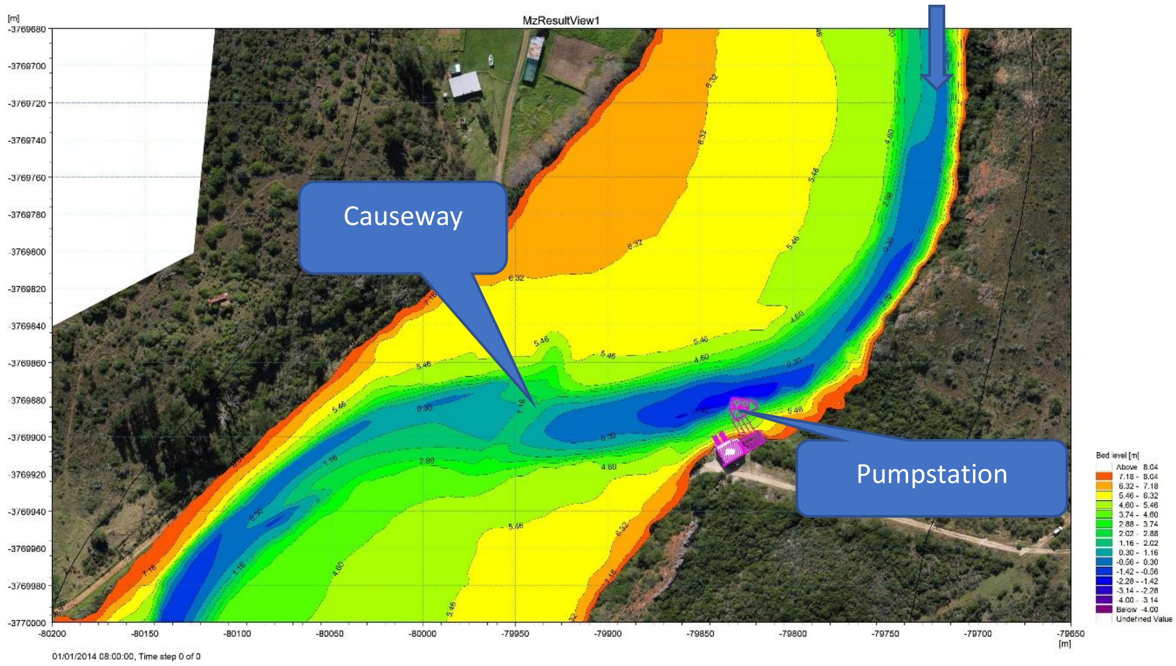


Figure 8-13: Simulated bed level following the 2-year flood

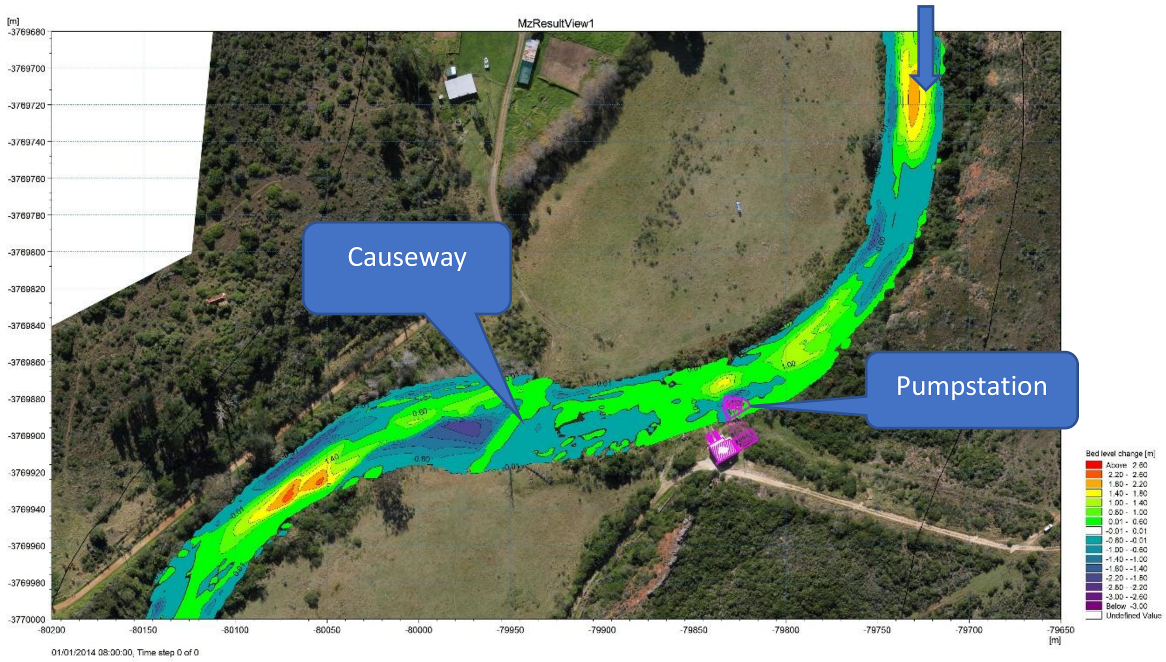


Figure 8-14: Simulated bed level change after the 2-year flood (positive values = deposition; negative values = erosion)

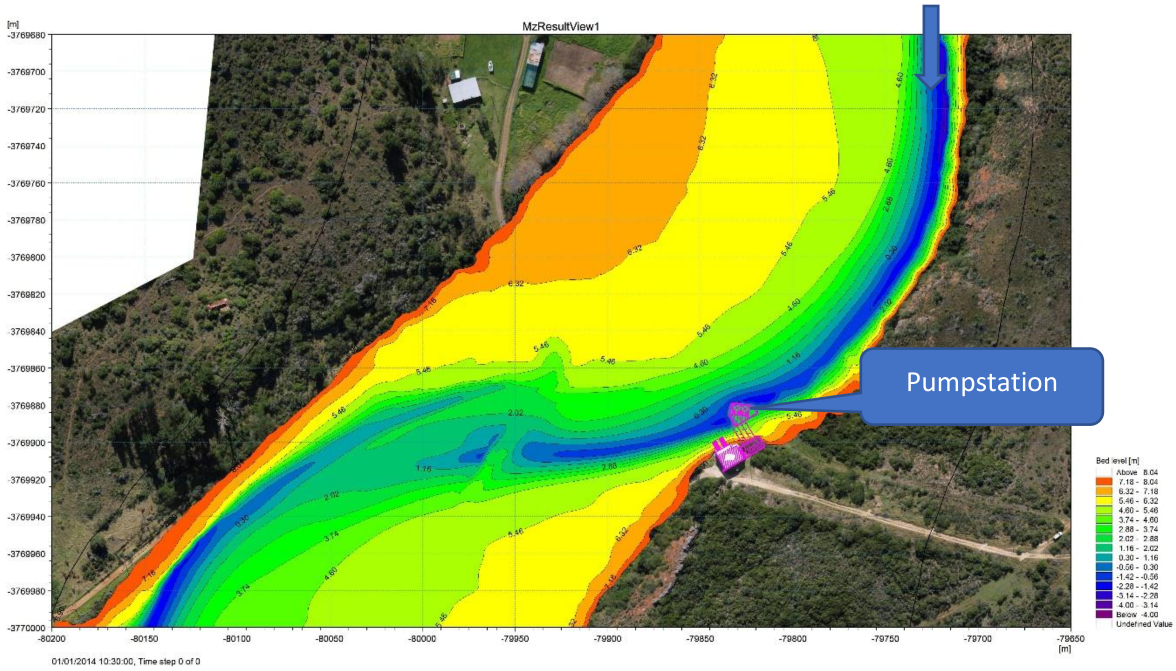


Figure 8-15: Simulated bed level following the 10-year flood

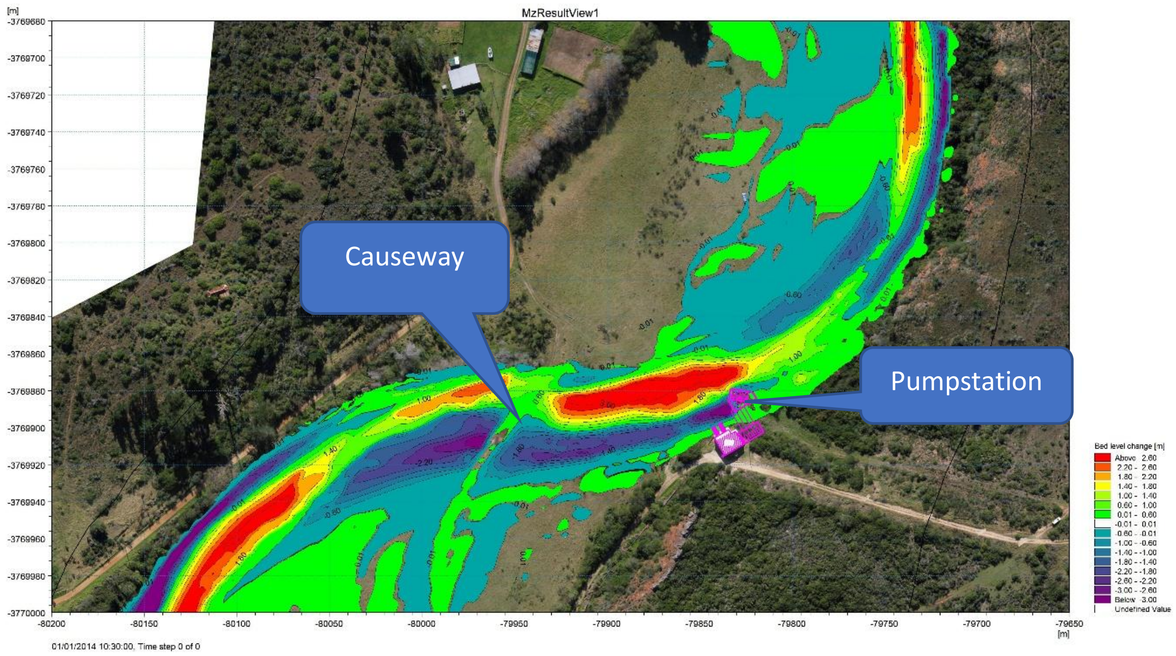


Figure 8-16: Simulated bed level change after the 10-year flood

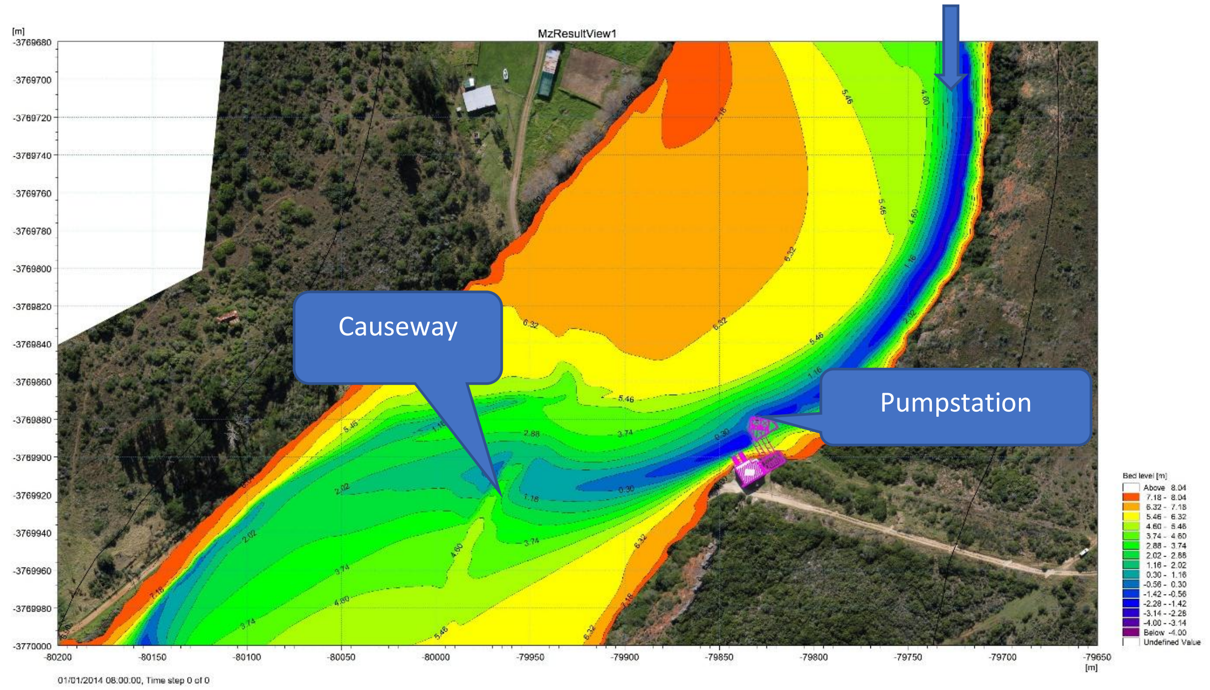


Figure 8-17: Simulated bed level following the 100-year flood

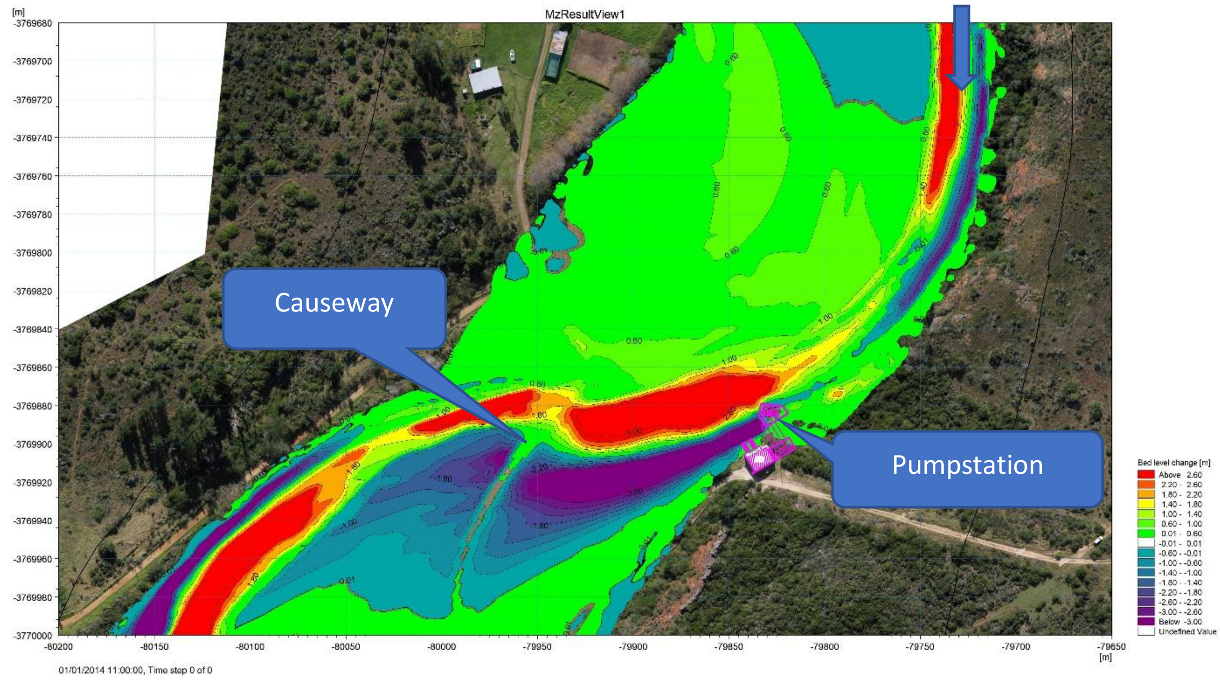


Figure 8-18: Simulated bed level change after the 100-year flood

8.3.Summary of findings

The key findings from the 2D hydrodynamic modelling of the sediment dynamics with the proposed intake located at the left bank on the bedrock upstream of the existing pumpstation are:

- The proposed intake is in scour zone at the outside of the bend and from small to large floods the proposed intake is self-scouring.
- The proposed intake is relatively low and submerged during the floods and therefore does not deflect the flow towards the right bank. The simulations for all the floods indicated that the

inside of the bend (right bank opposite the proposed intake) is a sediment deposition zone. Therefore, no erosion protection is required at the right bank.

- The left bank and floodplain between the proposed intake and the causeway, is scoured during the 10-year and 100-year floods. The possible erosion should be monitored, and critical infrastructure should be protected against scour.
- The proposed intake location on the left bank bedrock upstream of the existing pumpstation should be used for the detailed design of new river abstraction works.

9. 3D CFD modelling with FLOW3D-HYDRO of the hydraulic forces on the sheetpile and piling intake works

(To be included in the follow-up version of this report)

10. Concluding Remarks

The following are main concluding remarks to this investigation:

a) The 2025 and 2014 local surveys at the proposed abstraction works were compared to identify any scour and deposition that occurred in the river within the elapsed period of 11 years, which might affect the original proposed intake location of the upgraded abstraction works. Scour was observed on the outside bend of the river and deposition on the inside bend of the river as expected, however the upstream rock obstruction at the left bank caused deposition further downstream where the proposed intake works is located.

b) The proposed dimensions of the LYNERS proposed hoppers should be increased to at least 4 m x 4 m in plan with side slopes of 1H:2V (implying a hopper depth of about 4m). The reasons for the recommended increase are for the effective settlement of the coarser fraction of suspended sediment (sediment larger than about 0.4 mm) and to increase the volume of the hopper for less frequent sediment removal by jet-pumps. Each hopper should be provided with a permanently installed jet-pump for the intermittent removal of settled sediment from the hoppers. The proposed installation method for the jet-pumps is presented in **Appendix G**.

c) The purchased submersible type pumps are suitable for pumping raw river water containing some sediment and debris. However, the purchased booster type pumps are less suitable to pumping raw water containing some sediment and debris.

d) For a pumping system with a relative low pumping head (such as for this project with a static head of about 30 m) it is considered that the pumping capacity of the raw water pumps in the river works should be selected such that they can pump directly (without the assistance of a booster pump) to the Klipheuwel Dam. This will significantly simplify the operation and consequently increase the reliability of the pumping system.

e) Based on the reasoning under Item (d) above, four KSB KRT 200-402 pumps are recommended with two alternative cases:
   - Case 1: Four duty pumps with no standby with each delivering 200 l/s for a total discharge of 4 x 200 l/s = 800 l/s.
   - Case 2: Three duty and one standby (for improved reliability) with each delivering 266 l/s for a total of 3 x 266 l/s = 800 l/s

f) Cooling jackets are recommended for the proposed submersible pumps for additional protection and variable frequency motors are recommended to operate at their best efficiency both during normal river flow and flood flow conditions.

g) A separate submersible pump (KSB 80-315 – refer **Appendix J**) to be accommodated in the river pump bay with the main pumps is recommended for the jet-pumps' motive pump and the motive water from it should pass through a filter to prevent clogging of the fluidization nozzles of the jet-pumps. The filter system can be accommodated in the existing pump house.

h) The key findings from the 2D hydrodynamic modelling of the sediment dynamics with the proposed intake located at the left bank on the bedrock upstream of the existing pumpstation are:
   - The proposed intake is in scour zone at the outside of the bend and from small to large floods the proposed intake is self-scouring.
   - The proposed intake is relatively low and submerged during the floods and therefore does not deflect the flow towards the right bank. The simulations for all the floods indicated that the inside of the bend (right bank opposite the proposed intake) is a sediment deposition zone. Therefore, no erosion protection is required at the right bank.

- The left bank and floodplain between the proposed intake and the causeway, is scoured during the 10-year and 100-year floods. The possible erosion should be monitored, and critical infrastructure should be protected against scour.
- The proposed intake location on the left bank bedrock upstream of the existing pumpstation should be used for the detailed design of new river abstraction works.

i) Results on forces on the river intake works derived by 3D modelling will be reported on in follow-up versions of this report.

11. References

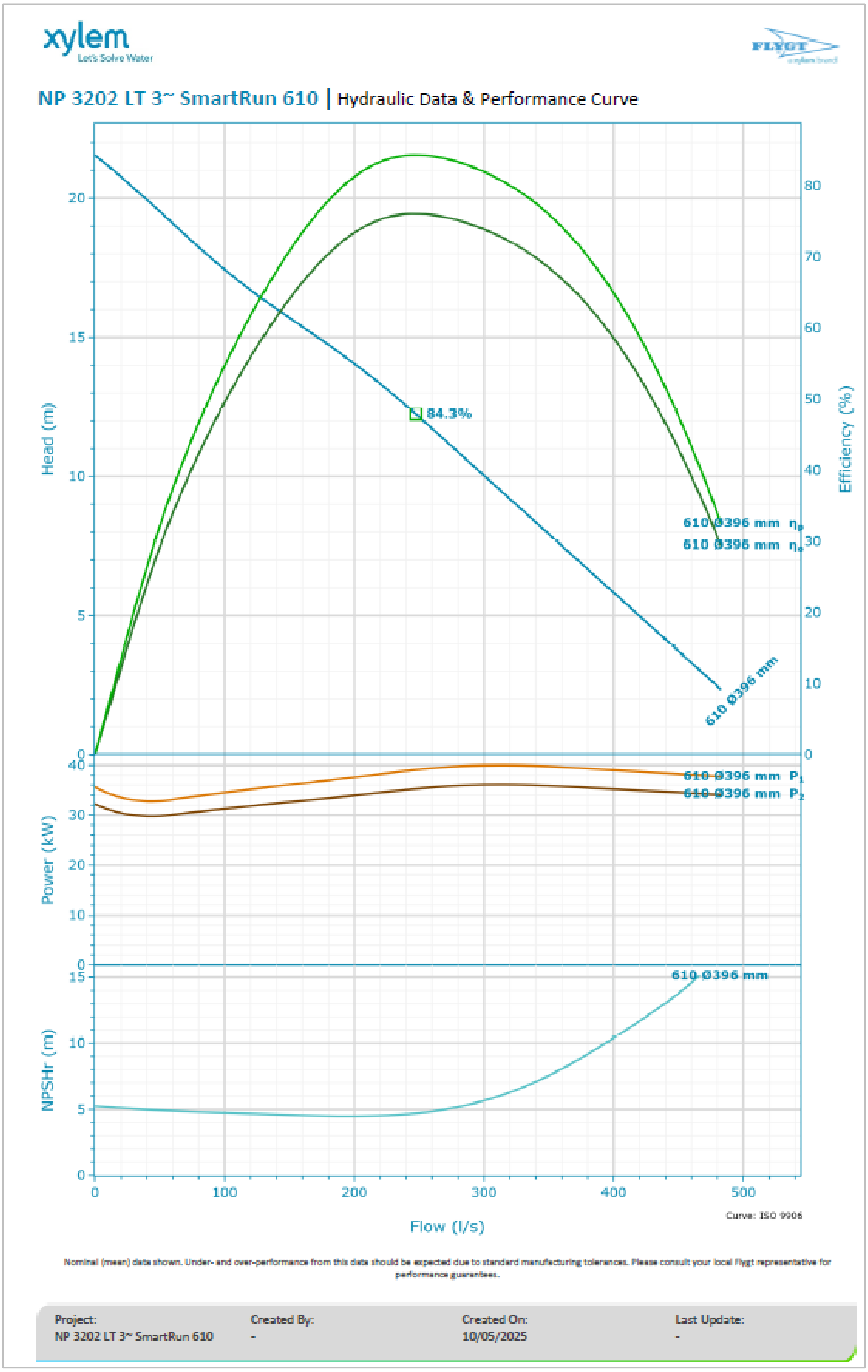
ASP (2014). Moordkuil raw water pumpstation rehabilitation - hydrology, hydraulics and sediment dynamics. Report to Lyners.

LYNERS (2016). Moordkuil Raw Water Intake and Pumping Station: Implementation Report.

Miedema, S., & Ramsdell, RC. (Ed.) (2016). Slurry Transport: Fundamentals, a Historical Overview & The Delft Head Loss & Limit Deposit Velocity Framework (DHLLDV). (1st ed.) SA Miedema / Delft University of Technology.

WRC (2002). AN INVESTIGATION INTO THE REMOVAL OF SEDIMENTS FROM WATER INTAKES ON RIVERS BY MEANS OF JET-TYPE DREDGE PUMPS. Report prepared for the Water Research Commission as a contract research project with Prestedge Retief Dresner Wijnberg. Authors: Bosman D.E., Prestedge G.K., Rooseboom A, Slatter P.T. https://www.wrc.org.za/wp-content/uploads/mdocs/1187-1-021.pdf

Appendix A: Flygt NP 3202 LT pump – Technical detail



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Selection

Series Experior	**Impeller Diameter** 396 mm
Name NP 3202 LT 3~ SmartRun 610	**Inlet Diameter** 300 mm
Frequency 50 Hz	**Outlet Diameter** 300 mm
System Type Single Pump	**Number Of Vanes** 2
Operating Pumps 1	
Standby Pumps No Standby Pump	
Curve Code 610	

Fluid

Fluid Type Water	**Density** 1,000 kg/m³
Fluid Temperature 4 °C	**Dynamic Viscosity** 0.001567 Pa·s
Specific Gravity 1	**Fluid Vapor Pressure** 8.135 mbar

Design Curve

Rated Speed 50 Hz	**BEP Flow** 247.52 l/s
Max Flow 482.92 l/s	**BEP Head** 12.24 m
H@QMin 21.55 m	**Max P2** 36.06 kW
H@QMax 2.33 m	
BEP 84.3 %	

Project:	Created By:	Created On:	Last Update:
NP 3202 LT 3~ SmartRun 610	-	10/05/2025	-

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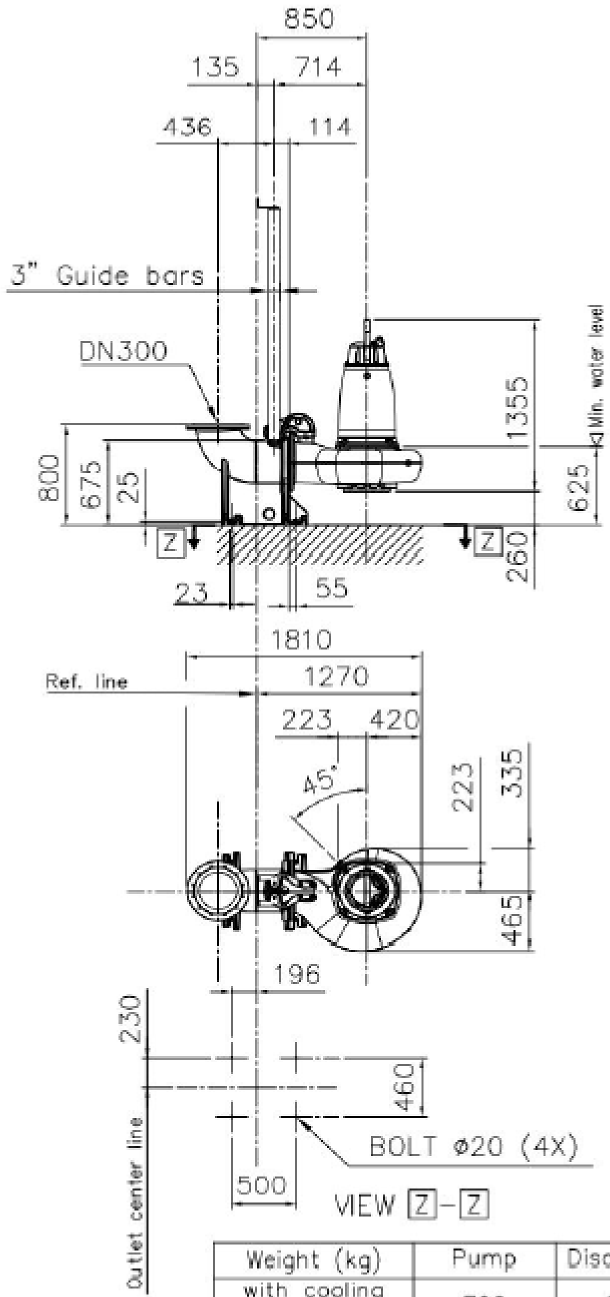
NP 3202 LT 3~ SmartRun 610 | Dimensional Data & Drawing



Weight (kg)	Pump	Discharge
with cooling jacket	790	291
without cooling jacket	740	291

FLYGT	NP 3202 LT	Discharge outlet DN 300	Pump outlet DN 300	Scale 1:40	Date 210312
	090,095,180,185,660,670			Drawing number 6841500	Revision 7

Project:	Created By:	Created On:	Last Update:
NP 3202 LT 3~ SmartRun 610	-	10/05/2025	-

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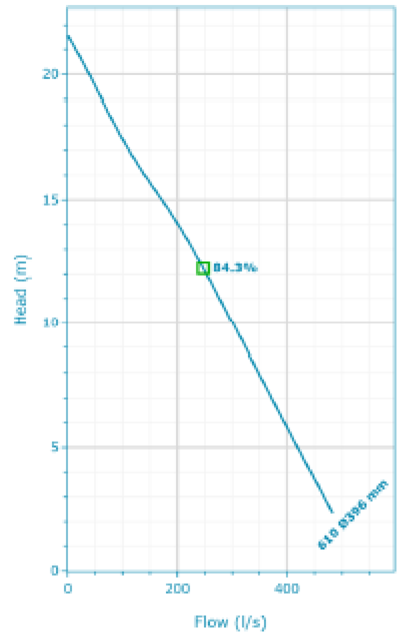
NP 3202 LT 3~ SmartRun 610 | Configuration Summary



Curve: ISO 9906

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances. Please consult your local Flygt representative for performance guarantees.

Patented self cleaning semi opened channel impeller, ideal for pumping in wastewater applications. Modular based design with high adaptation grade. Equipped with a user-friendly intelligent control unit, pre-programmed for wastewater pumping used for pumping station with one or two pumps.

Installation

Installation Type
P - Semi-Permanent, Wet

Motor

Rated Voltage
400 V

Motor Efficiency Class
Standard

Coupling
D

Rated Power
37 kW

Materials

Impeller Material
Grey Cast Iron

Volute Material
Grey Cast Iron

Performance

Explosion Proof
No

Impeller Diameter
396 mm

Max. Pumped Media Temp.
40 °C

Project:	Created By:	Created On:	Last Update:
NP 3202 LT 3~ SmartRun 610	-	10/05/2025	-

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NP 3202 LT 3~ SmartRun 610 | Product Details

Description

N 3202

Flygt Experior

Patented self cleaning semi opened channel impeller, ideal for pumping in wastewater applications. Modular based design with high adaptation grade. Equipped with a user-friendly intelligent control unit, pre-programmed for wastewater pumping used for pumping station with one or two pumps.
The Flygt N-series are equipped with the Flygt invented N-technology with its innovative self-cleaning impeller. Solid objects entering the pump will pass through the impeller between the impeller vanes. If an object gets caught on the leading edge of one of the vanes, it will slide along the backswept shape towards the perimeter of the inlet where it will be guided by a relief groove through the pump housing. This ensures a high sustained total efficiency over time. Due to the mechanical self-cleaning design, a sludge concentration of solids up to 8% can easily be pumped. The pump can easily be installed in either permanently or temporary submerged, or horizontally or vertically dry installations.

Flexible and Modular Design

The modular hydraulic design enables customization of the hydraulics to meet the requirements of many applications.

- Replaceable wear ring in two materials, grey iron or Hard Iron, for different operation conditions
- Hardened grey iron impeller for typical wastewater applications
- Hard-Iron impeller for heavy duty wastewater applications containing abrasive and corrosive content
- Chopper ring intended for tough wastewater applications where cutting is required due to long fibres and solid concentrations up to 10-12%
- Stainless steel impeller for special applications that require duplex stainless steel

Robust and Reliable

- Short shaft overhang reduces shaft deflection and increases seal and bearing life
- Class H Motor designed for submersible use. Heat is concentrated to the stator core for improved cooling properties.
- The Plug-in seal with Active Seal system eliminates the risk associated with incorrect installation and careless handling. All in one unit. Available in Tungsten carbide (WCCR) or Silicone carbide (SiC) depending on pumped media.
- Motor cable SUBCAB® specially developed for submersible use
- Offers flexible cooling systems, e.g. closed-loop cooling system, media cooled or external cooling that allows full motor potential in dry installations.
- Premium brand bearings, greased for life, ensures a minimum of 50 000 hours of duty
- Leakage sensor and motor temperature sensor as standard

The N 3202 is available with the following options

- ATEx, FM, CSA-approvals
- Premium efficiency motors
- Hard Iron hydraulic design
- Stainless Steel hydraulic design
- Vibration-sensor, extended motor temperature sensors, additional leakage-sensor, current-sensor and pump memory
- Compatible with SmartRun® Wastewater pump controller
- Compatible with MAS 801 monitoring system

Product Features

- State-of-the-art wastewater pump with N-technology
- Sustained high efficiency pumping with energy savings up to 25%
- Flexible and modular design
- Robust and reliable

Construction Materials

Impeller Material	Volute Material	Stator Cover Material
Grey Cast Iron	Grey Cast Iron	-

Motor

Rated Power	37 kW	Number Of Phases	3	Start Current Ratio	5.69	Motor Issue	11
Motor Denomination	30-29-6AA	Rated Motor Speed	980 RPM	Insulation Class	H	Locked Rotor Code	G
Motor Efficiency Class	Standard	Rated Voltage	400 V	Approval	Standard	Max starts per hour	30
Version Code	180	Rated Current	71 A	Total moment of inertia	0.7737 kgm^2	Power Factor 100%	0.83
Frequency	50 Hz	Start Current	405 A	Type of duty	S1	Power Factor 75%	0.79
Max P2 (1x)	36.06 kW	Starting Current, Direct Starting	405 A	Stator Variant	1	Power Factor 50%	0.68
Number Of Poles	6	Starting Current, Star Delta	135 A	Motor Module	170	Efficiency 100%	90 %
						Efficiency 75%	90.5 %
						Efficiency 50%	90.5 %

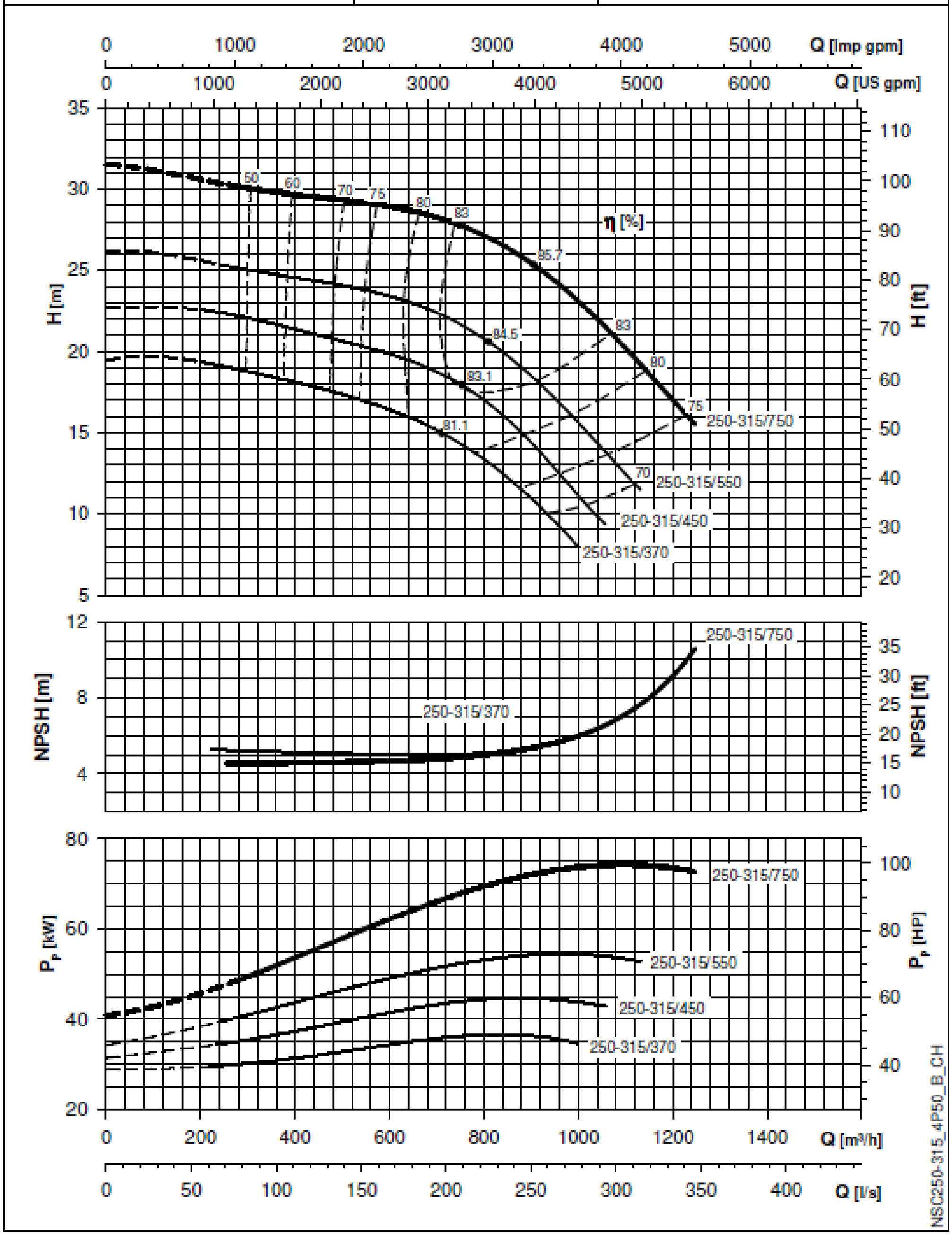
Project:	Created By:	Created On:	Last Update:
NP 3202 LT 3~ SmartRun 610	-	10/05/2025	-

Appendix B: LOWARA NSC 250-315 pump – Technical detail

LOWARA
a xylem brand

e-NSC SERIES
OPERATING CHARACTERISTICS AT 50 Hz, 4 POLES

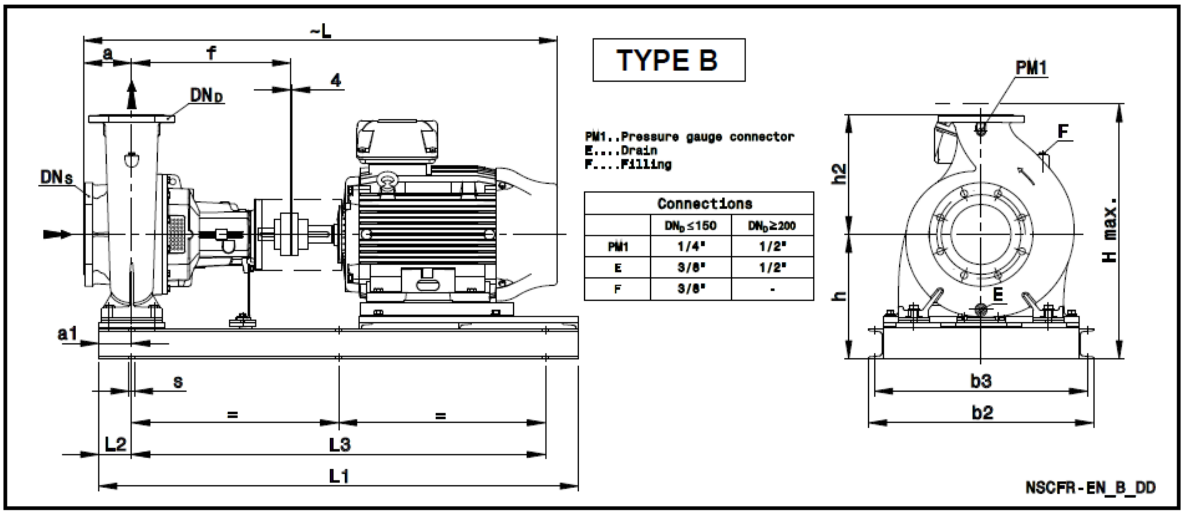
NSC 250-315	~ 1475 [rpm]	ISO 9906:2012 - Grade 3B



The NPSH values are laboratory values; for practical use we suggest increasing these values by 0,5 m.
These performances are valid for liquids with density ρ = 1,0 Kg/dm³ and kinematic viscosity ν = 1 mm²/sec.

a xylem brand

NSCF 150 SERIES (MOUNTED ON BASE)
DIMENSIONS AND WEIGHTS AT 50 Hz, 4 POLES



PUMP TYPE NSCF..4	TYPE	DIMENSIONS (mm)															WEIGHT (kg)	COUPLING TYPE
		DNS	DND	a	a1	b2	b3	f	h	h2	L	L1	L2	L3	H max	s FOR SCREWS	G	
250-315/370/W	B	300	250	250	165	850	810	530	525	500	1670	1700	165	1370	1025	6xØ19 (M16)	905	B140B

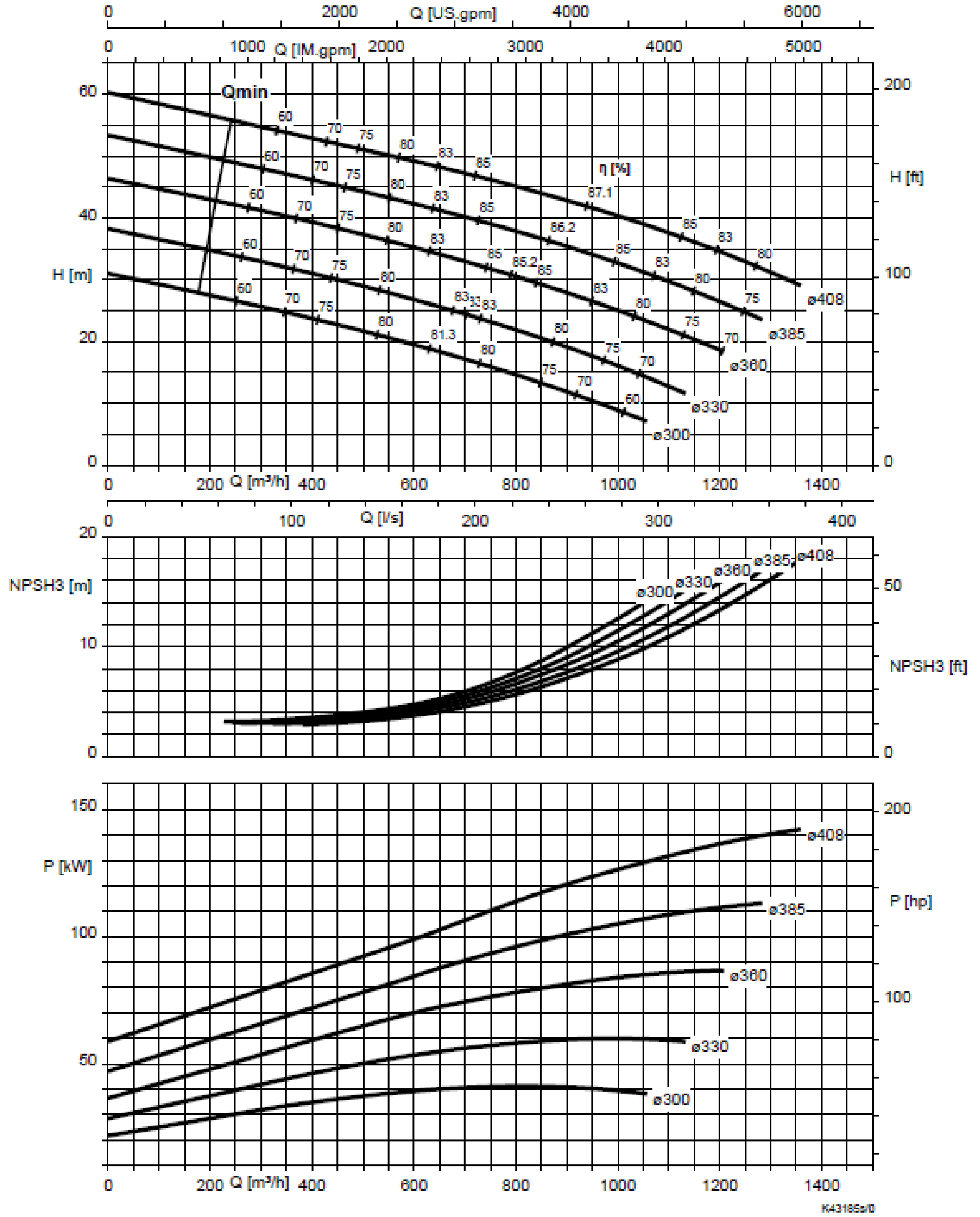
Appendix C: KSB KRT K 200-402 pump – Technical detail

KSB

Waste Water
Submersible Motor Pump

Amarex KRT K 200-402, n = 1450 rpm

Characteristic curves to ISO 9906 Class 3B, below 10 kW to § 4.4.2. n = nominal speed



Free passage = 80 mm

KSB

Waste Water
Submersible Motor Pump

General arrangement drawing S5, stationary on duckfoot bend, guide rail arrangement, single-level foundation, without foundation rail, small upper holder, motor version N

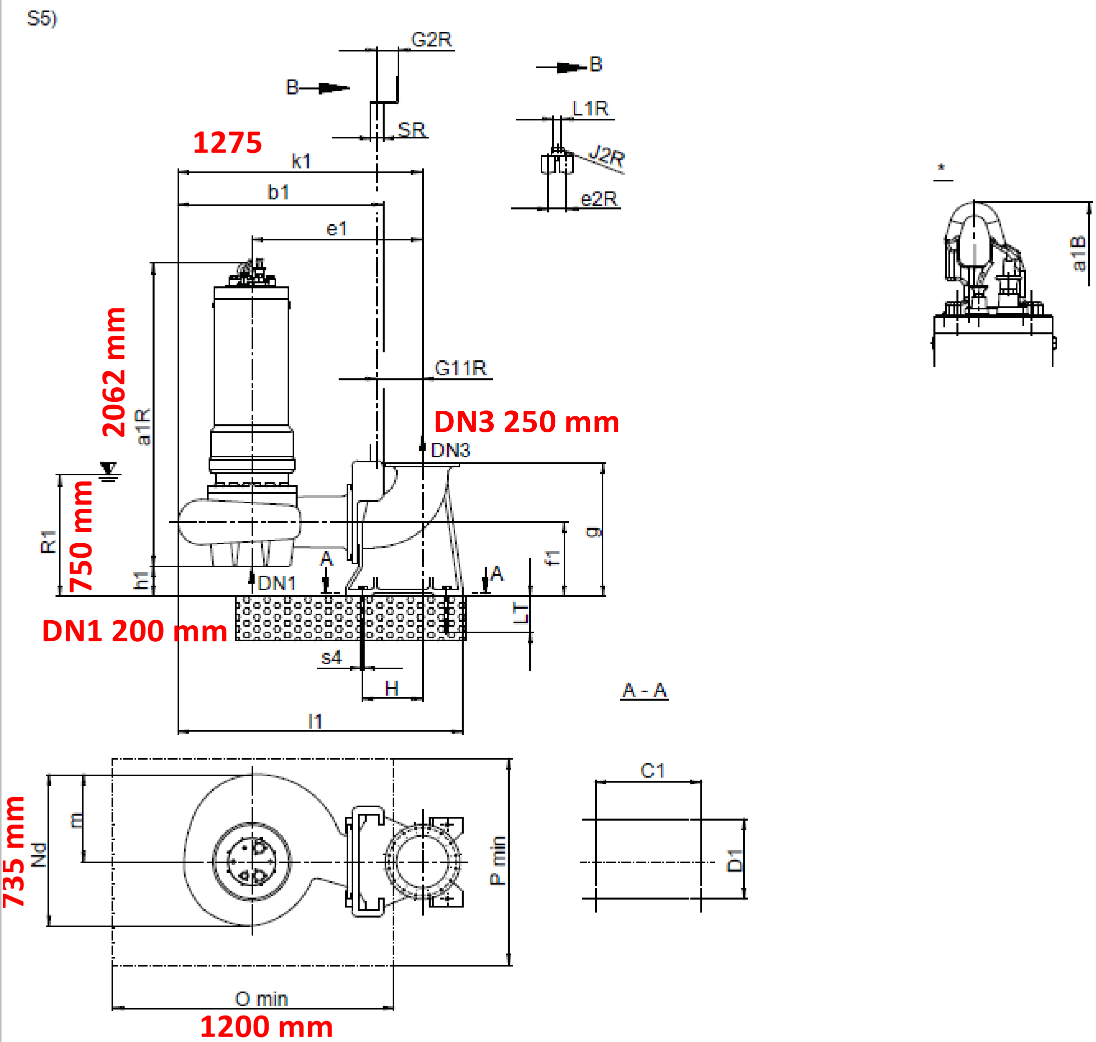


Fig. 6: General arrangement drawing S5, stationary on duckfoot bend, guide rail arrangement, single-level foundation, without foundation rail, small upper holder, motor version N

*: Optional

Size	Impeller type 50 Hz	Impeller type 60 Hz	Motor	Installation types K/S with guide wire arrangement Figure see page	Installation types K/S with guide rail arrangement Figure see page	DN1	DN3	Nominal diameter of bend	b1 [mm]	e1 [mm]	a1B [mm]	f1 [mm]	g [mm]	h1 [mm]	k1 [mm]	l1 [mm]	R1 (Motor version UN/UE/WN/WE/XN/XE/YN/YE) [mm]	R1 (Motor version ZN/ZE) [mm]	E12 [mm]	H [mm]	W1 [mm]	LT [mm]	s4 [mm]	A11 [mm]	E21 [mm]	m [mm]	Nd [mm]	C1 [mm]	D1 [mm]	C11 [mm]	C2 [mm]	O min [mm]	P min [mm]
200-402	K	K	130 4	24	14	200	250	200/250	1120	955	2062	400	745	216	1275	1478	750	750	-	350	-	125	18	-	-	410	735	490	300	450	-	1200	800

Appendix D: Jet-pump performance curves

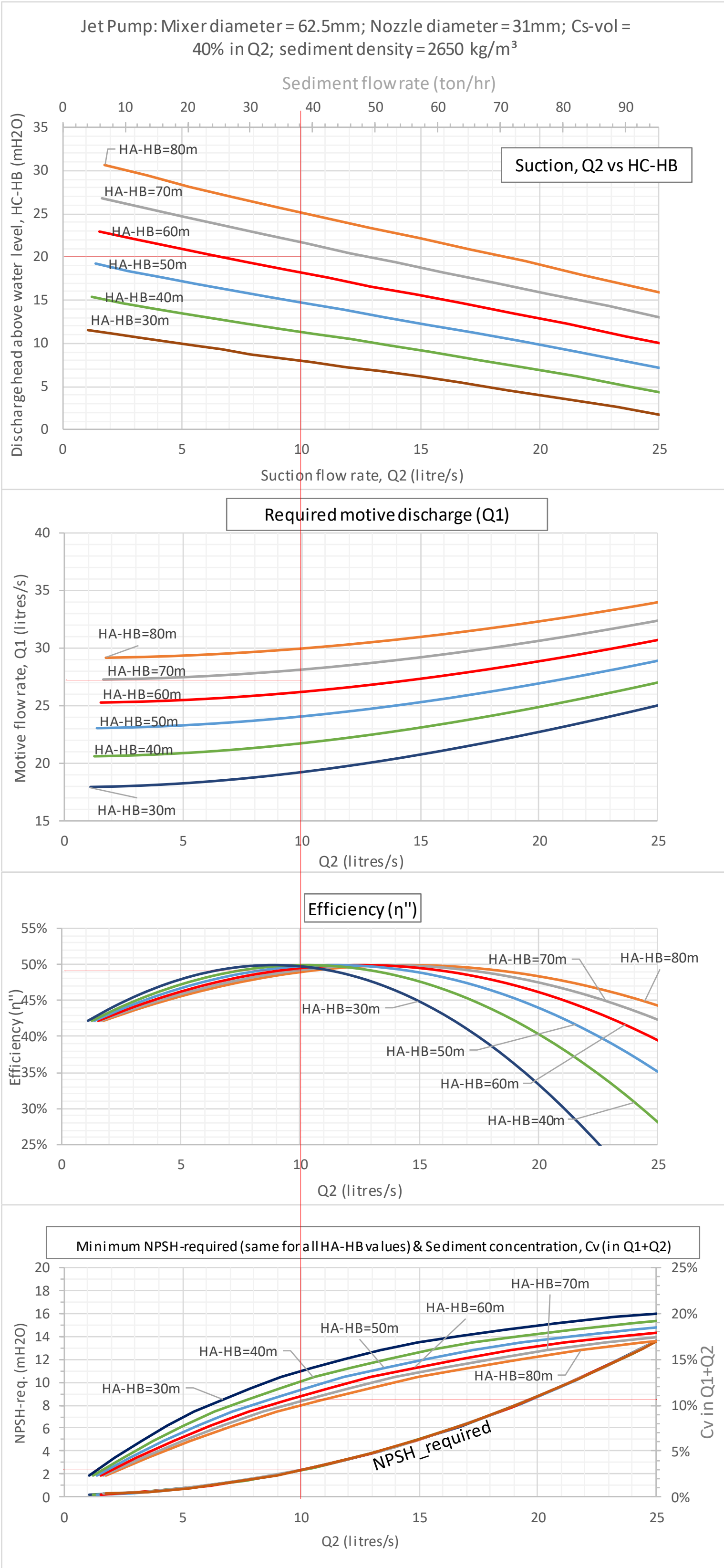


Figure D1: Performance curves of jet-pump with 62.5 mm diameter mixer and 31 mm diameter main nozzle (refer to Figure C2 for declaration of symbols)

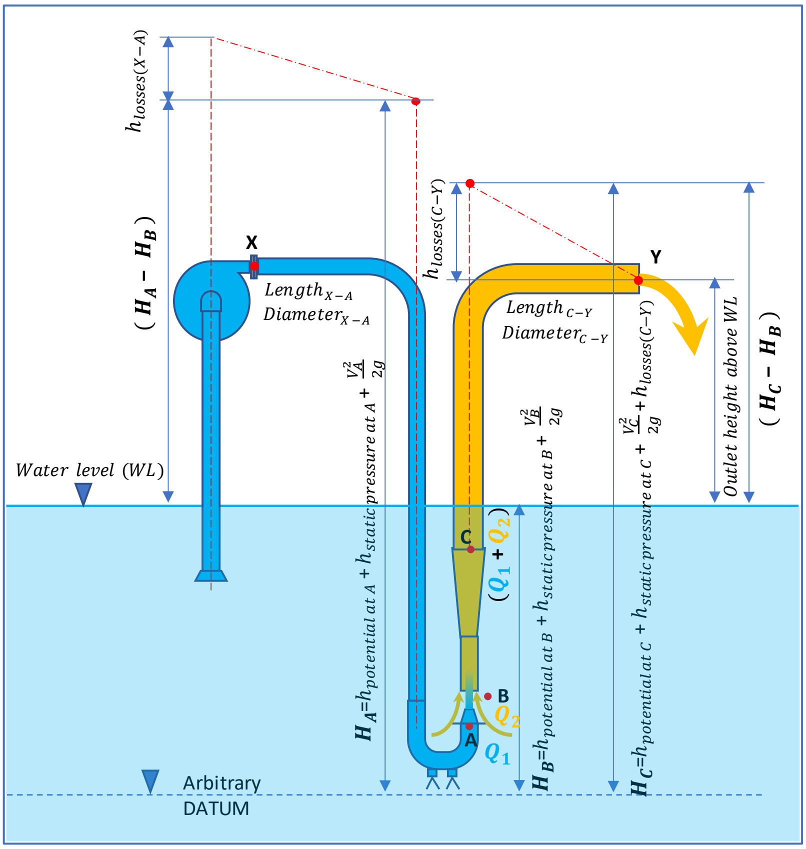
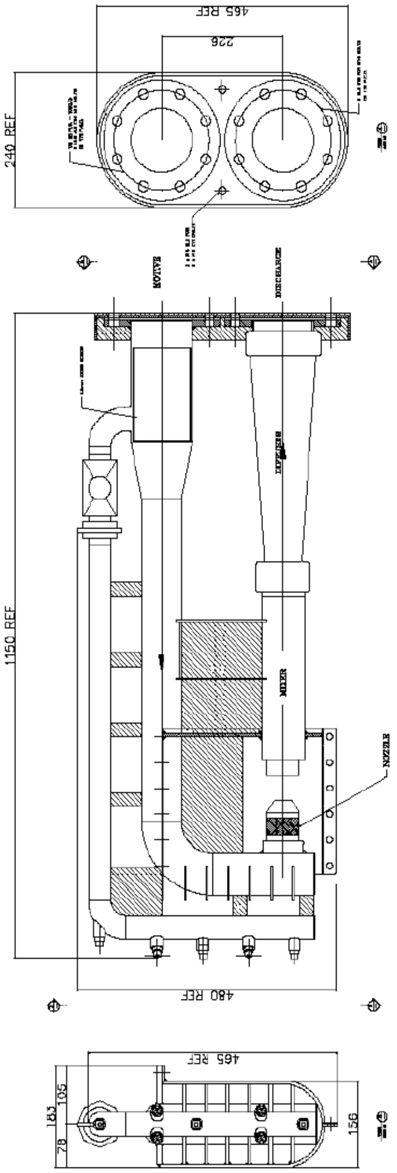


Figure D2: Declaration of symbols used in Figure C1.

Appendix E: Jet-pump examples

Example of GENFLO SANDBUG jet-pump



Half cowl removed with main jet and mixer visible

Appendix F: Mobile Jet-pump in action during WRC field evaluation research project (WRC, 2002)

EXAMPLE OF MOBILE JET PUMP REMOVING SAND FROM A RIVER INTAKE PUMPSTATION

Appendix G: Alternative arrangement for a permanently installed jet-pump in a hopper of as pump forebay

Possible alternative of using rubber supply and discharge pipes for jet pump instead of rigid supply and discharge pipes

OPTION FOR INSTALLATION AND RETRIEVAL CONFIGURATION OF DREDGED TYPE JET PUMP IN HOPPER OF RIVER INTAKE PUMP STATION EMPLOYING RUBBER HOSE PIPES

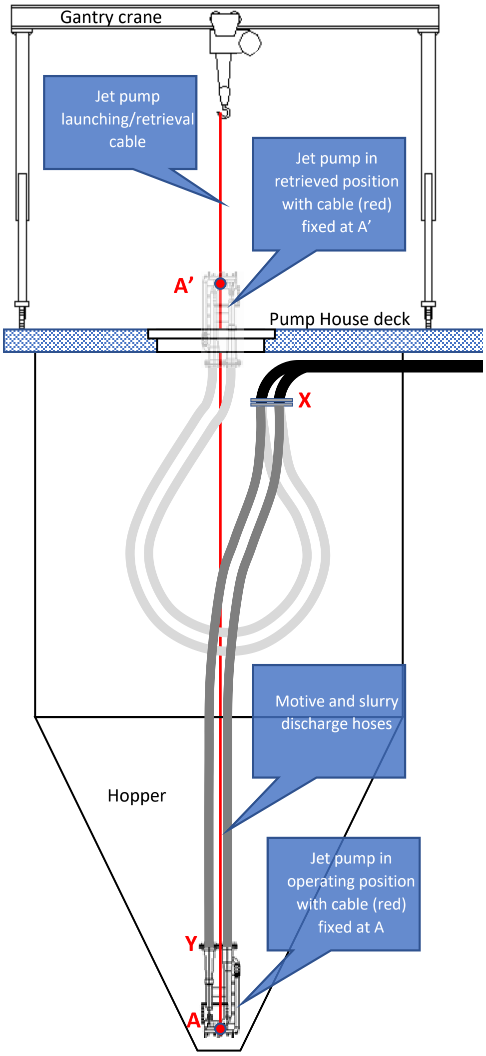


Figure 1: Schematic sketch of installed jet pump (dark grey) in hopper and retrieved for maintenance (light grey) with dredge type hoses between X and Y.

Description of rubber hoses

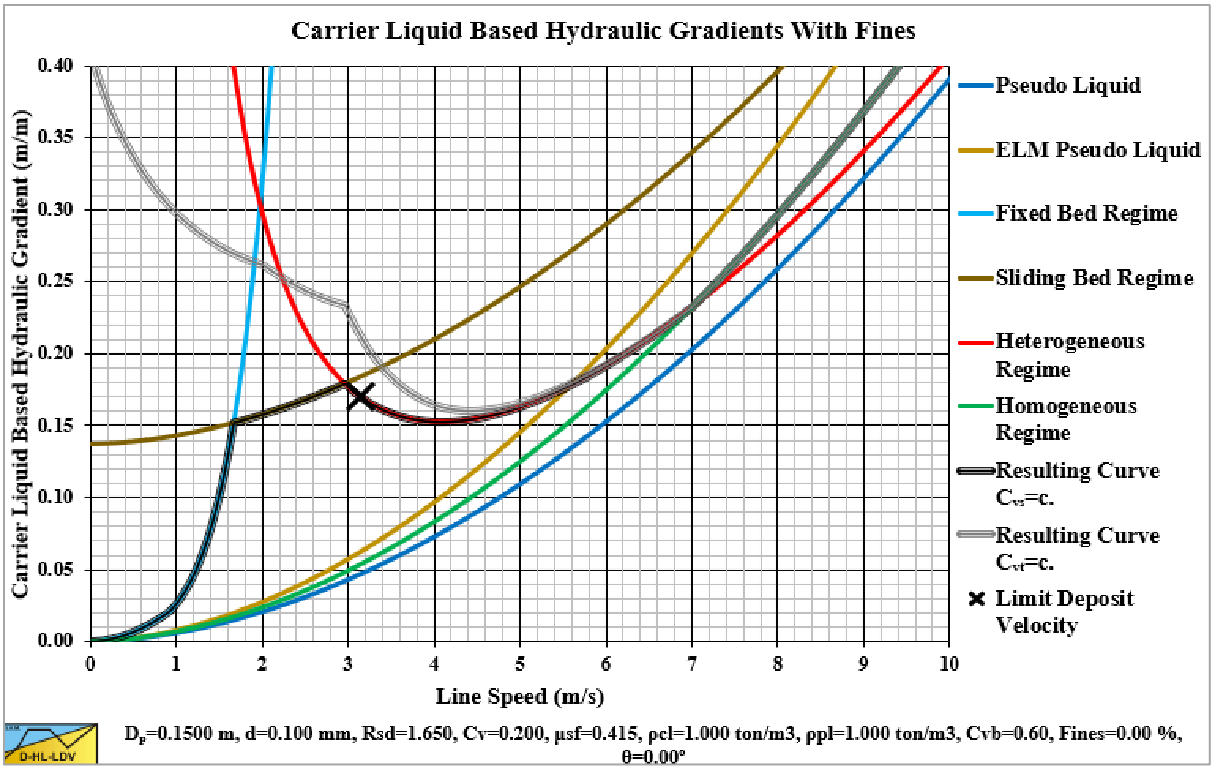
As an alternative to steel pipe for the vertical pipes in the hopper compartment of the intake pump station, dredge type rubber hoses could be considered. This will enable easier retrieval of the jet pump when maintenance on it is required. Also, the jet pump can be activated before reaching the invert of the hopper during launching so that it could dredge itself into position at the hopper invert. Figure 1 demonstrates the installed position of the jet pump on the invert of the hopper (with rubber hose indicated in dark grey) and at its retrieved position at the pump house operating deck (in light grey).

A suitable rubber hose is the dredge type rubber hoses with a LINATEX weir resistance lining as manufactured by WEIR (refer Figure 2 - https://www.global.weir/industries/mining/), or similar type of dredge hose.

The hose flanges should be of swivel flange ends enabling rotation of the backing flange for ease of alignment of the bolt holes as shown in Figure 2. The flange ends should be of the type that can resist significant *axial* stresses to ensure robustness and should therefore be appropriately integrated with the hose material.

Figure 2: Swivel flange ends of LINATEX dredge type hose manufactured by WEIR (https://www.global.weir/industries/mining/)

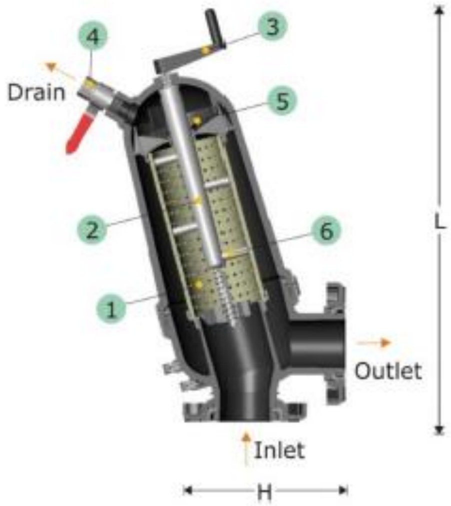
Appendix H: Head loss and sediment limit deposit velocity graph for jet-pump slurry pipe (Miedema et al, 2016)



Appendix I: Example of a screen filter for jet-pump motive water

Operation Description - Filtration Mode

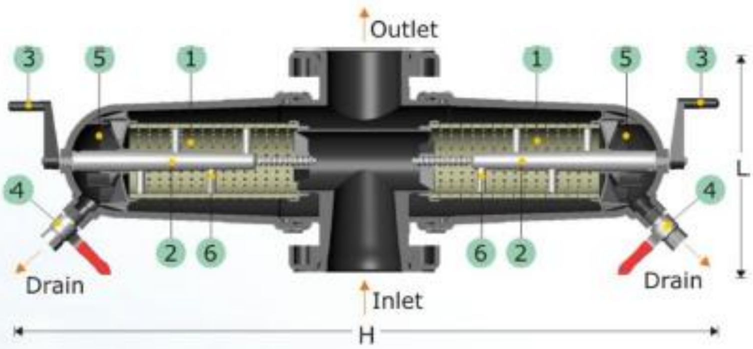
Water enters the filter through the "Inlet", reaching the screen 1, which purifies the water by taking out smaller particles. As more water flows through impurities build up on the screen creating differential pressure between the internal section of the screen 1 and the external section of the screen.



Description

Arkal's 3", 4" and 6" Semi-Automatic filters consist of the following major parts:

1 Screen
2 Dirt Collector
3 Handle
4 Flush Valve
5 Flush Chamber
6 Nozzles



Cleaning Process in Suction Type Filter

When the differential pressure (ΔP) reaches the determined value (no more than 0.8 bar), or according to the timetable specified by the operator, a series of events occurs as the water continues to flow into the system's units.

Before opening the flush valve 4 the operator should check that the dirt collector 2 is fully withdrawn by turning the handle 3 clockwise until it stops. When the flush valve 4 opens, water flows out.

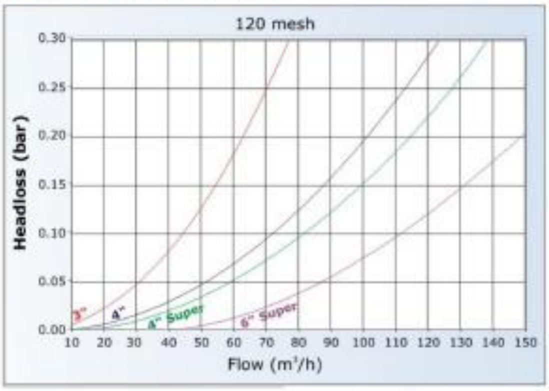
The operator should also check that the pressure in the flush chamber 5 and the dirt collector 2 is significantly reduced, which will create suction through the collector's nozzles 6. Rotating the dirt collector 2 by the handle will cause full scanning of the screen 1 by the nozzles 6. The combination of linear movement and rotation cleans the whole internal screen 1 surface. The flushing cycle will take only a few seconds.

General Technical Data

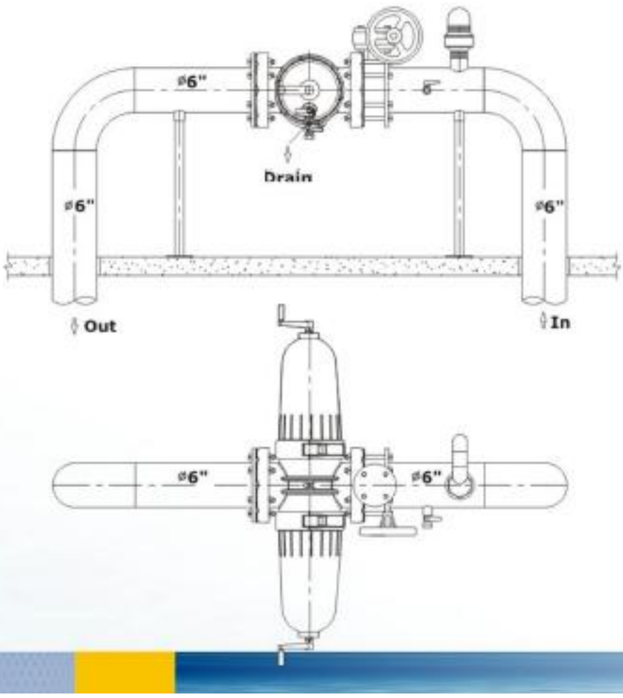
- Max. Pressure: 10 bar / 145 psi
- Max. Temperature: 60°C / 140°F
- Filtration Grades (currently): 400μ, 200μ, 120μ

Model Number	Connection Size (inch)	Screen Area (cm^2)	Max. Flow Rate (M^3/h)	L (mm)	H (mm)	Weight (kg)
AKSP3LT	3	1250	60	825	509	12
AKSP3LV	3	1250	60	789	509	12
AKSP3LF	3	1250	60	789	509	13
AKSP4LV	4	1250	90	789	509	13
AKSP4LF	4	1250	90	789	509	14
AKSP4S	4	2500	110	445	1368	26
AKSP6S	6	2500	140	415	1368	28

Headloss Chart



AKSP = Arkal Semi Automatic Polypropylene
L = Angel filter connection
T = Threaded filter connection
V = Victaulic filter connection
F = Flanged filter connection
S = Super leader filter (inline filter connection)



Appendix J: Proposed motive pump to drive jet-pumps

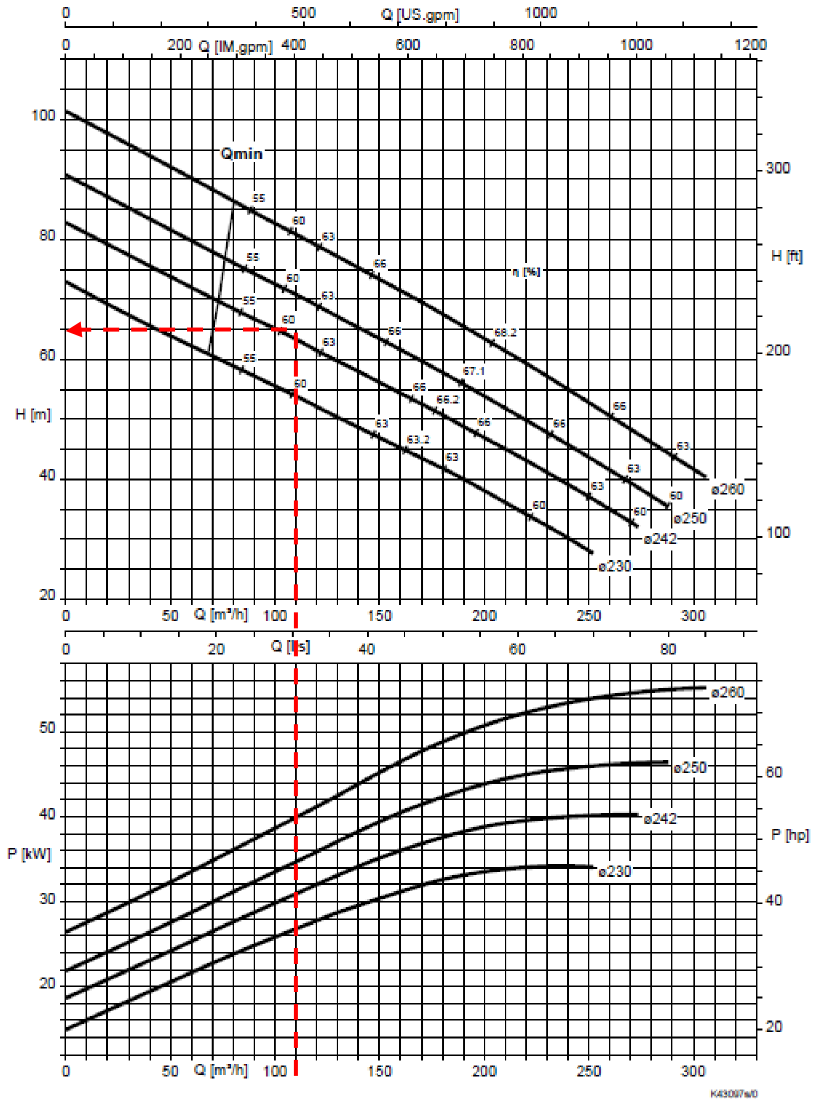
KSB

Waste Water
Submersible Motor Pump

D impeller

Amarex KRT D 80-315, n = 2900 rpm

Characteristic curves to ISO 9906 Class 3B, below 10 kW to § 4.4.2. n = nominal speed



Free passage = 65 mm

KSB

Waste Water
Submersible Motor Pump

General arrangement drawing S14, stationary on duckfoot bend, guide rail arrangement, foundation with step, without foundation rail, motor version E

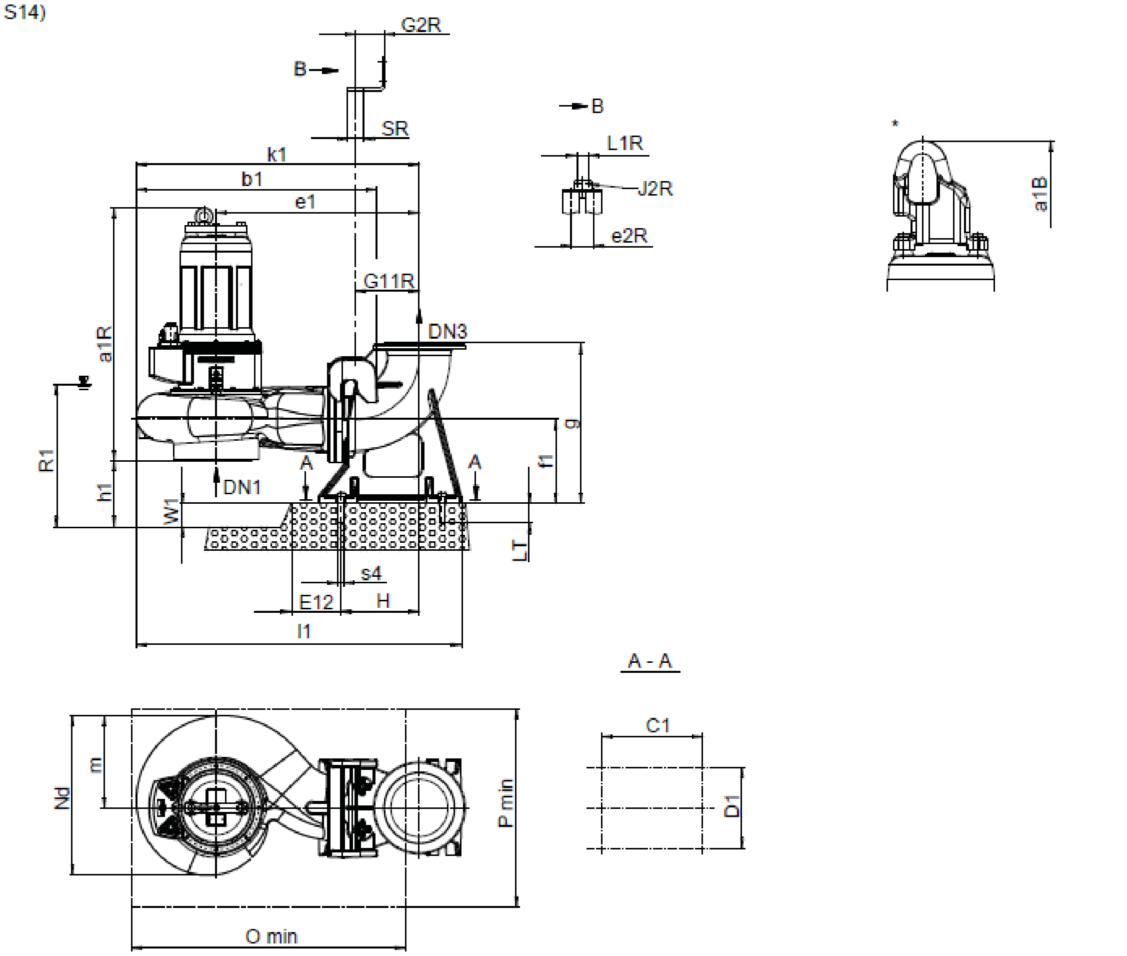


Fig. 9: General arrangement drawing S14, stationary on duckfoot bend, guide rail arrangement, foundation with step, without foundation rail, motor version E

*: Optional

080-315, installation types K/S

080-315, installation types K/S, dimensions and weights depending on the material variant, part 1

Size	Impeller type 50 Hz	Impeller type 60 Hz	Motor	Installation types K/S with guide wire arrangement Figure see page	Installation types K/S with guide rail arrangement Figure see page	DN1	DN3	Nominal diameter of bend	b1	e1	a1B	f1	g	h1	k1	l1	R1 (Motor version UN/UE/WN/WE/XN/XE/YN/YE)	R1 (Motor version ZN/ZE)	E12	H	W1	LT	s4	A11	E21	m	Nd	C1	D1	C11	C2	O min	P min
									[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]
080-315	D	-	55 2	27	17	100	100	100/100	771	530	1633	240	430	102	885	1011	590	765	200	139	80	125	18	-	-	203	383	220	150	-	-	900	500

Appendix K: 2D Hydrodynamic modelling results

2-year flood scenario

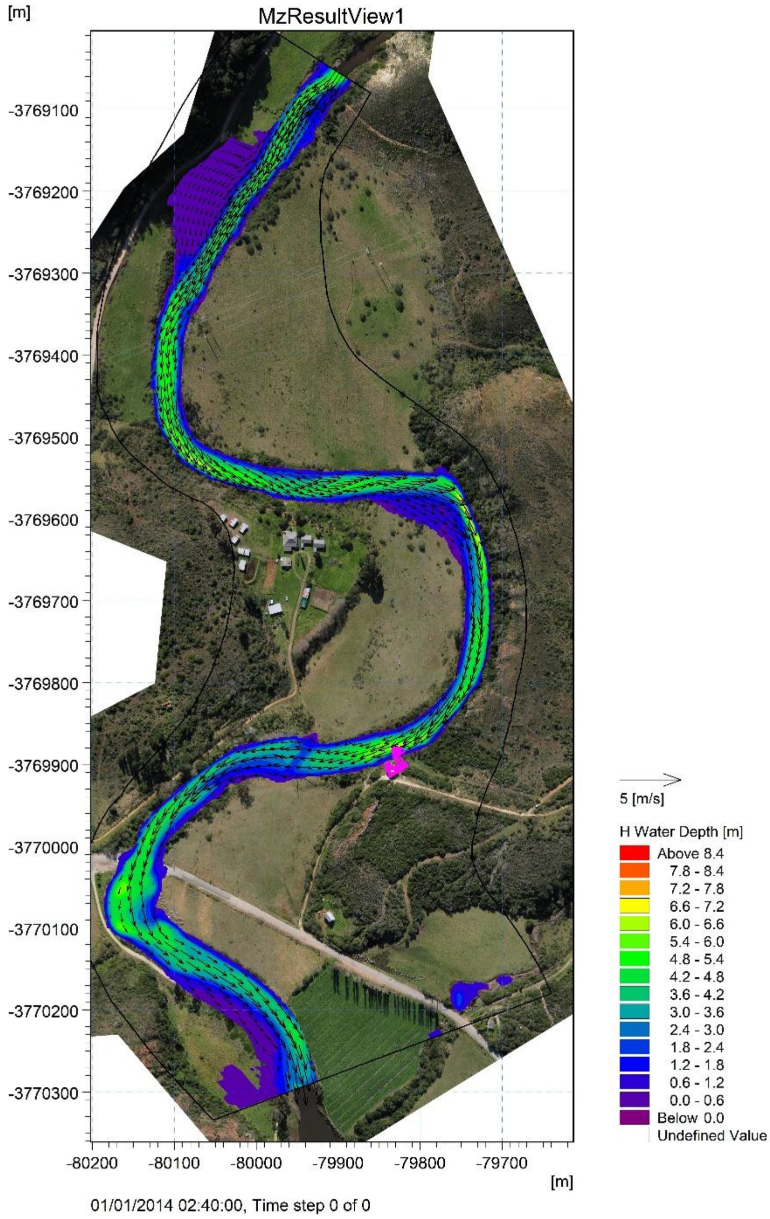


Figure J-1: Simulated flow depths during 2-year flood peak

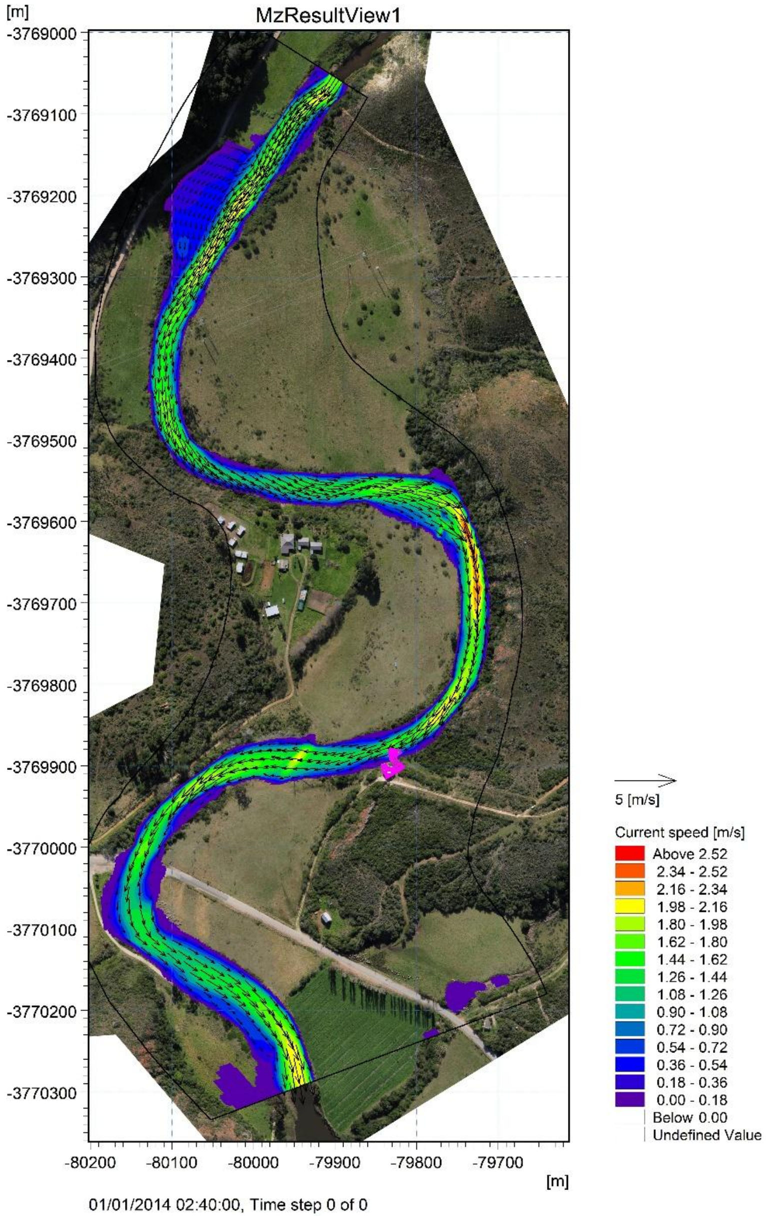


Figure J-2: Simulated flow velocities during 2-year flood peak

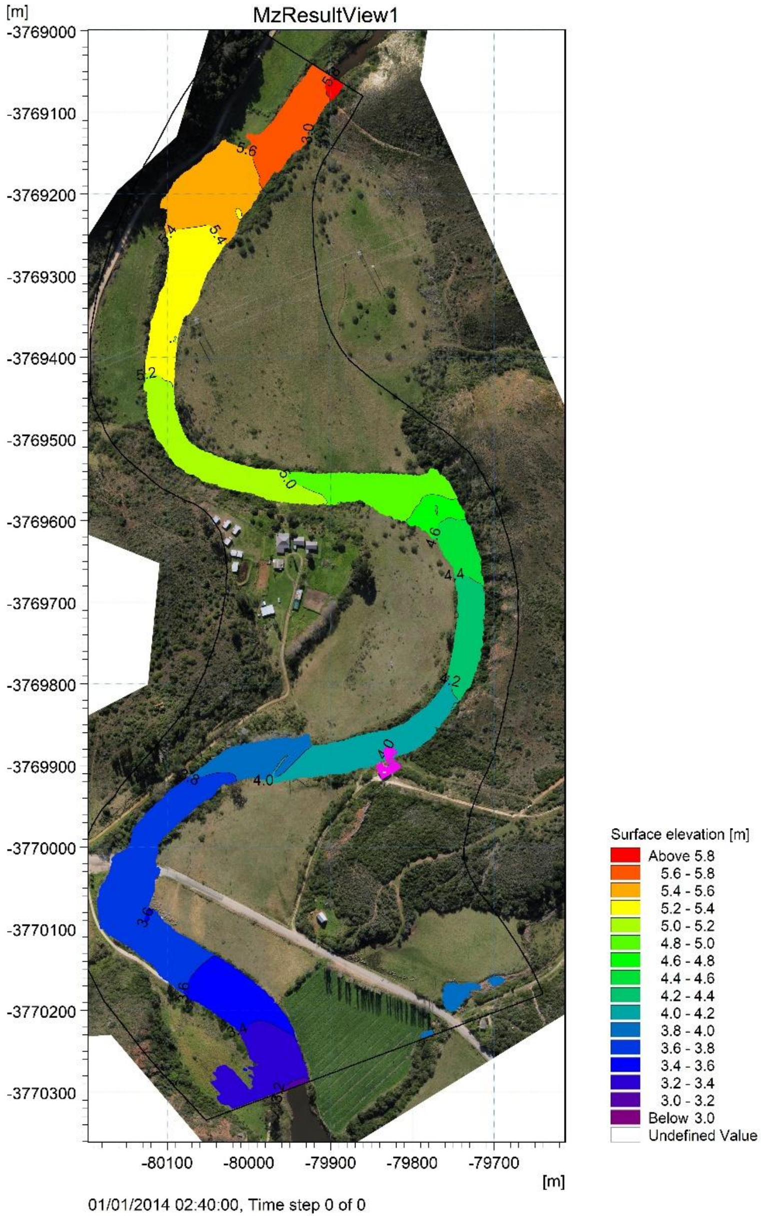


Figure J-3: Simulated water levels during 2-year flood peak

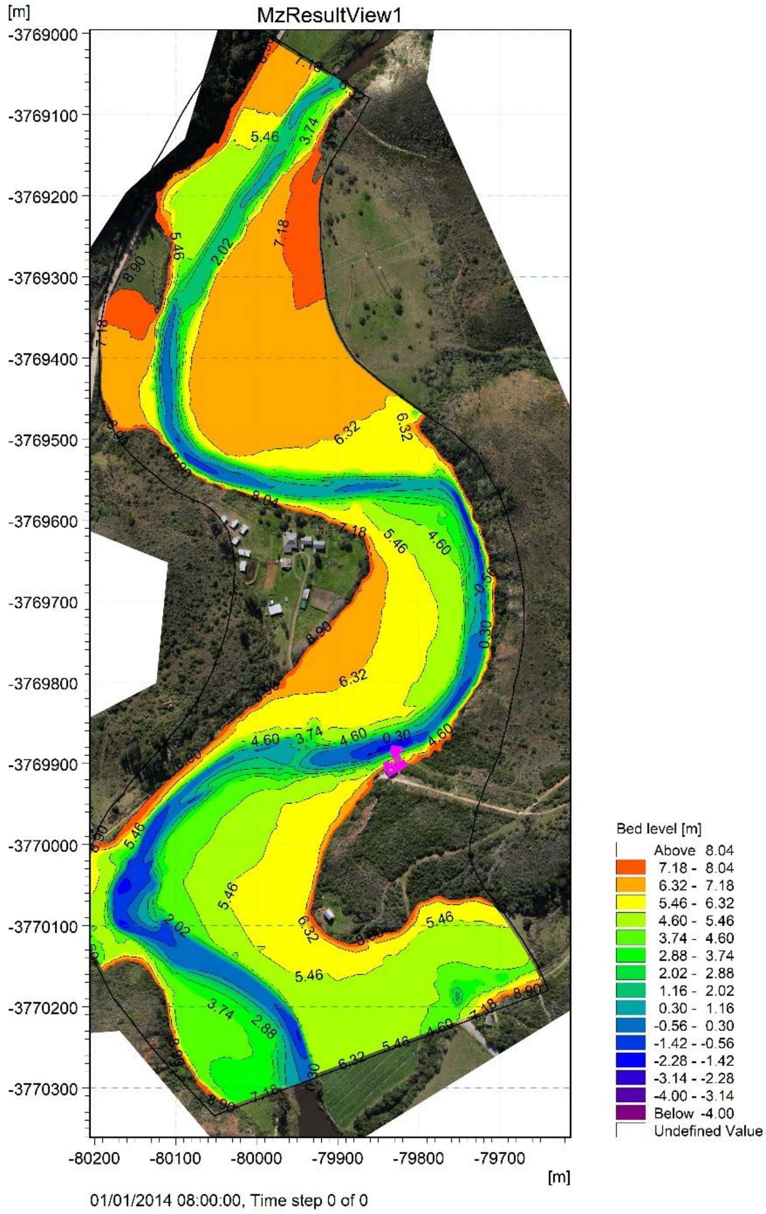


Figure J-4: Simulated bed levels at the end of 2-year flood peak

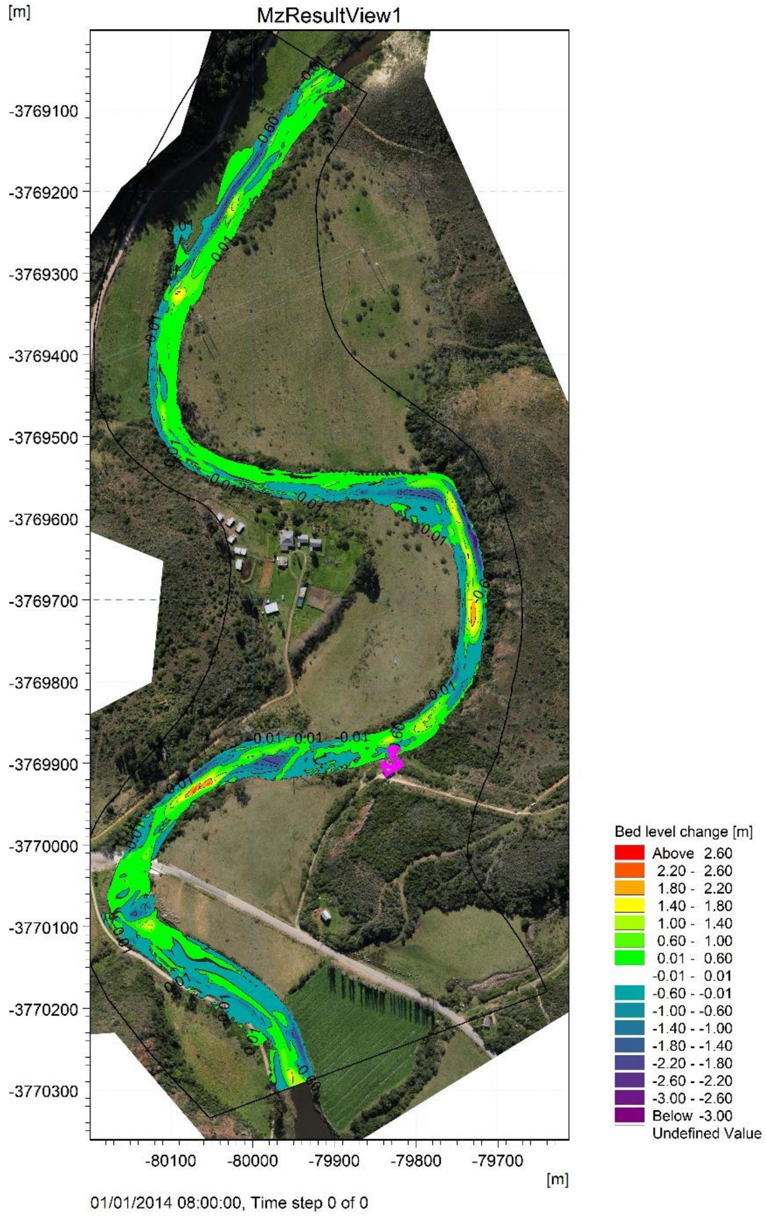


Figure J-5: Simulated bed level change at the end of 2-year flood peak

10-year flood scenario

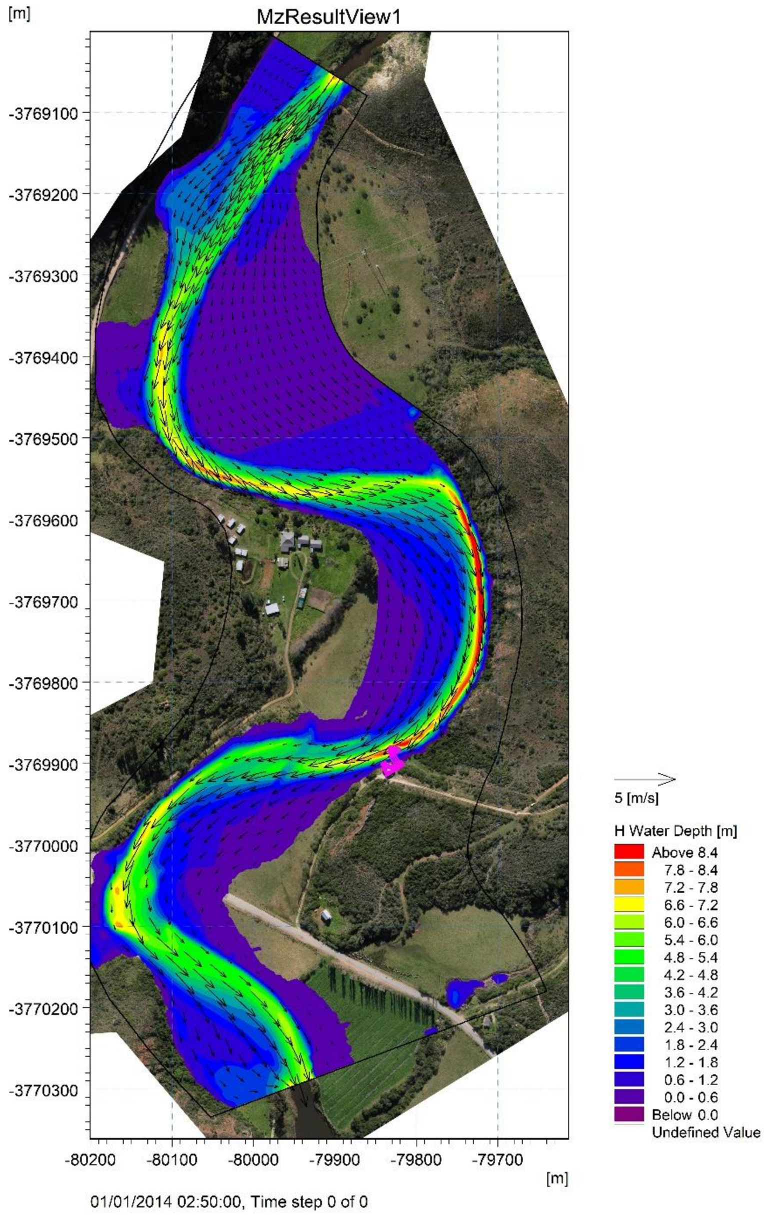


Figure J-6: Simulated flow depths during 10-year flood peak

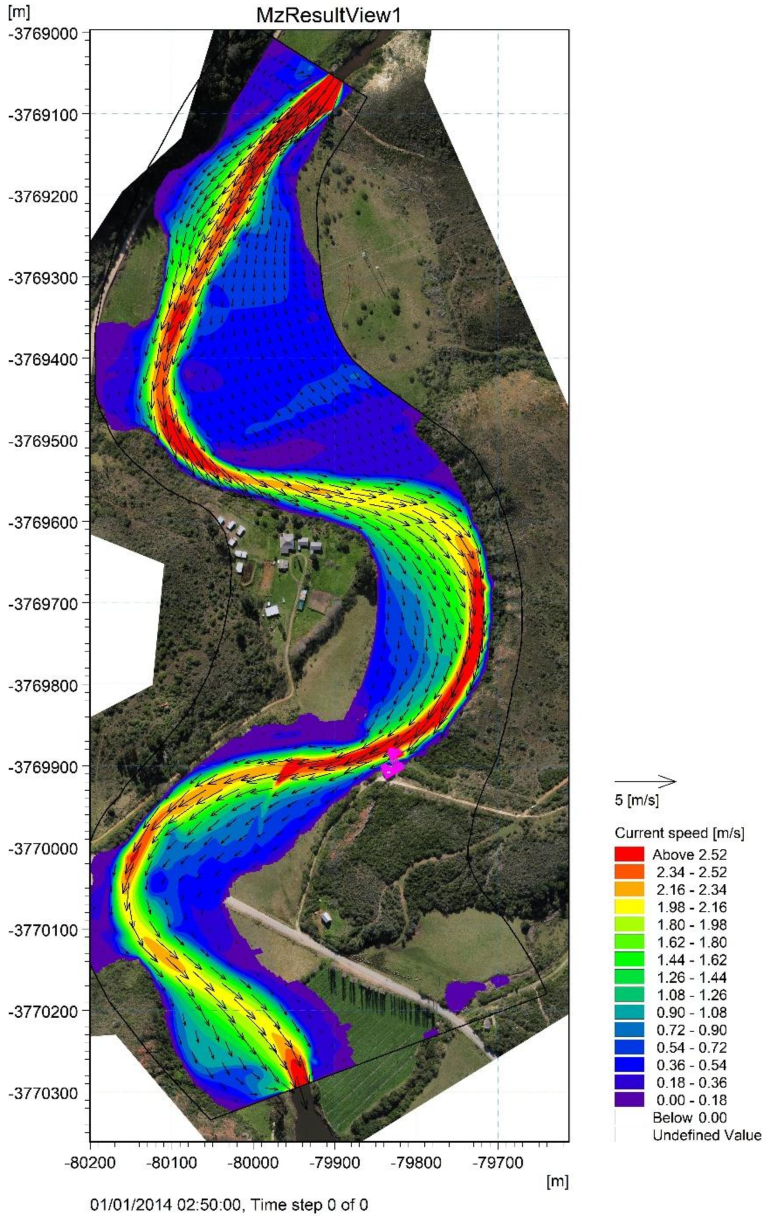


Figure J-7: Simulated flow velocities during 10-year flood peak

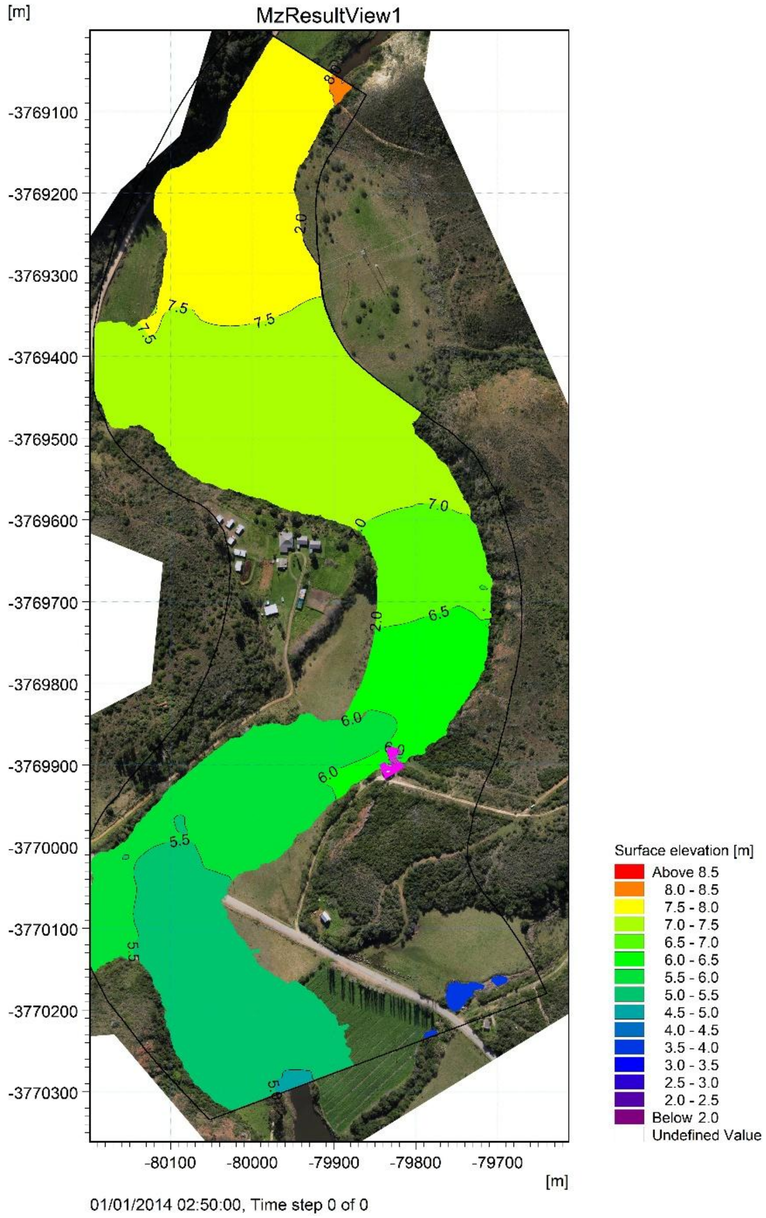


Figure J-8: Simulated water levels during 10-year flood peak

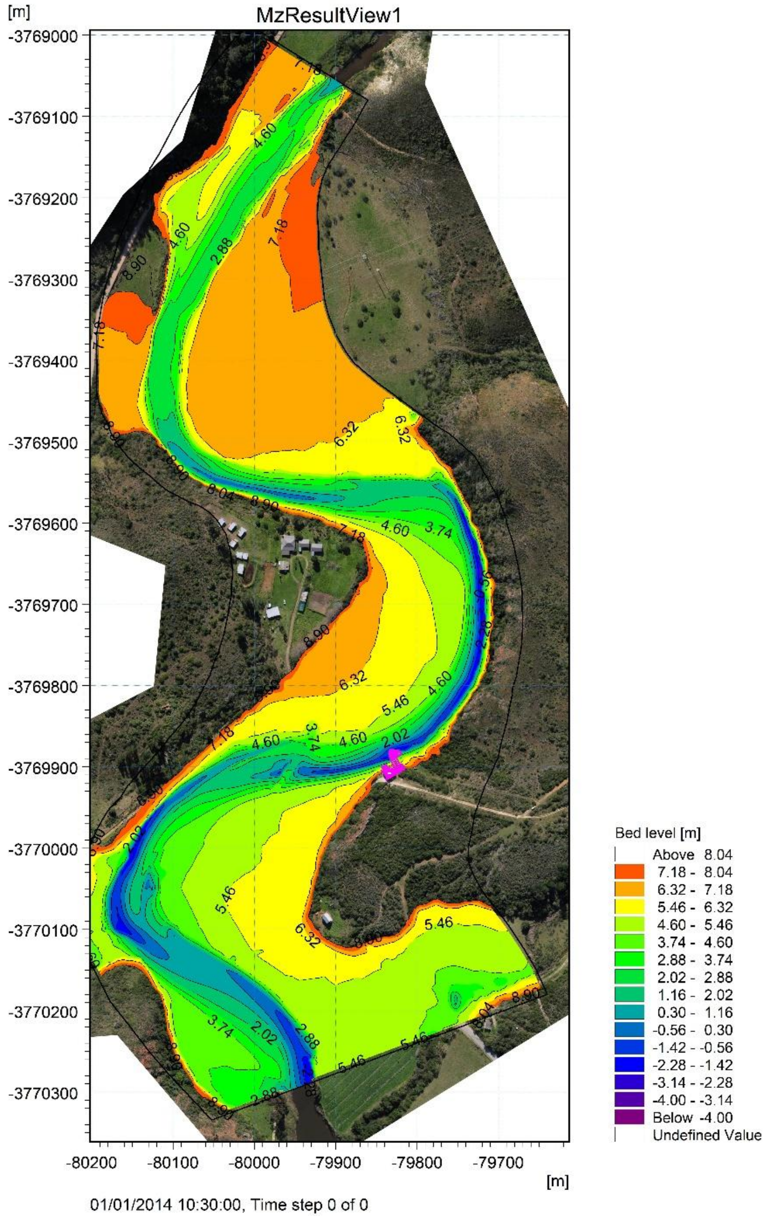


Figure J-9: Simulated bed levels at the end of 10-year flood peak

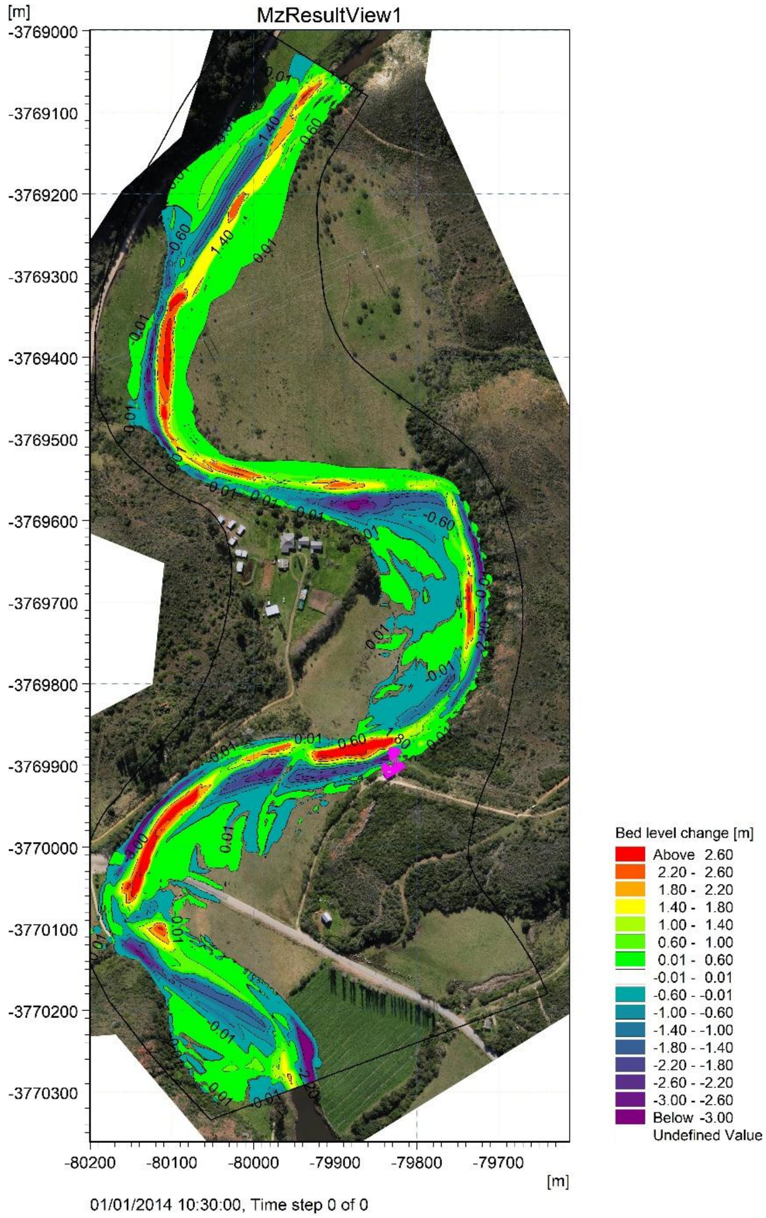


Figure J-10: Simulated bed level change at the end of 10-year flood peak

100-year flood scenario

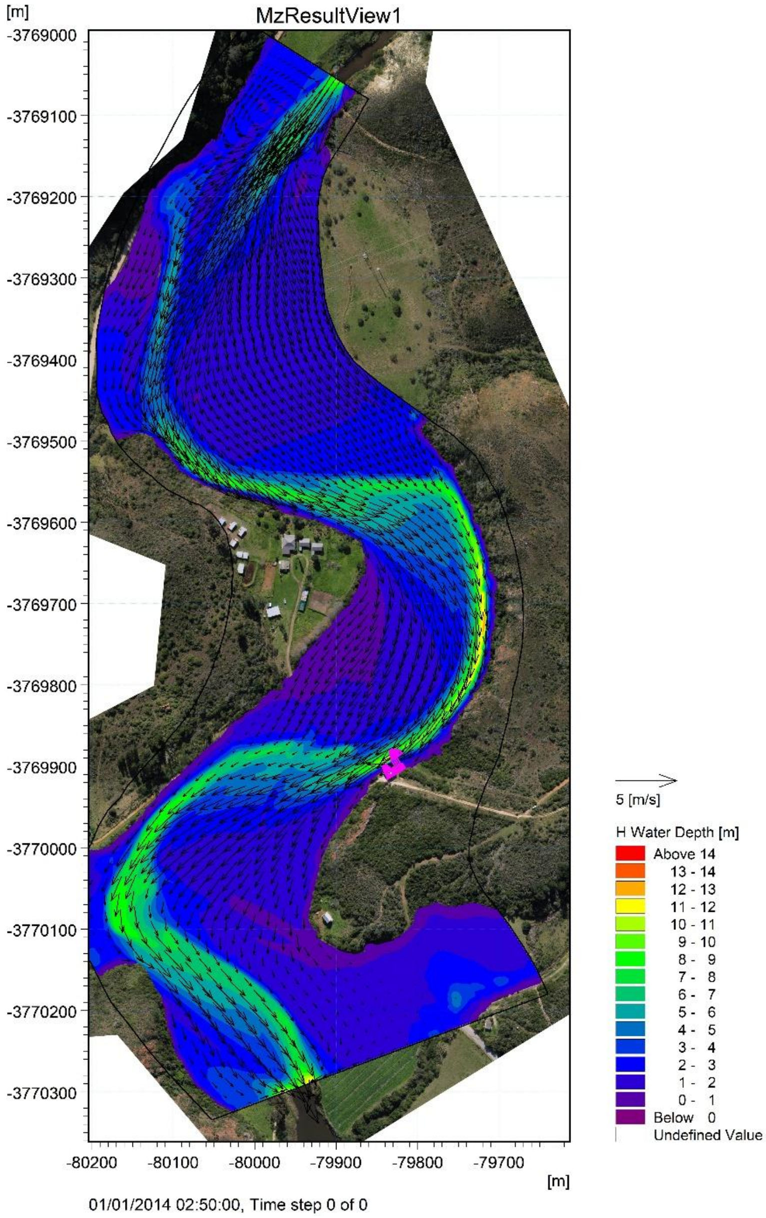


Figure J-11: Simulated flow depths during 100-year flood peak

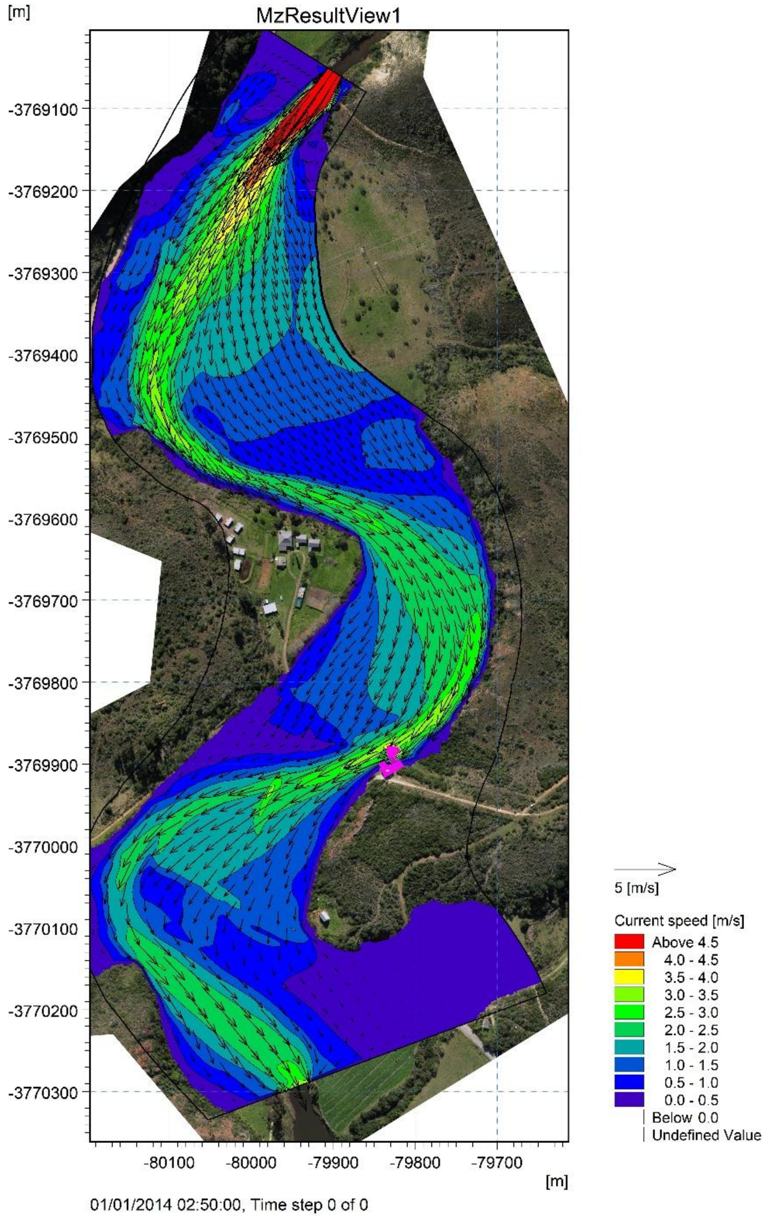


Figure J-12: Simulated flow velocities during the 100-year flood peak

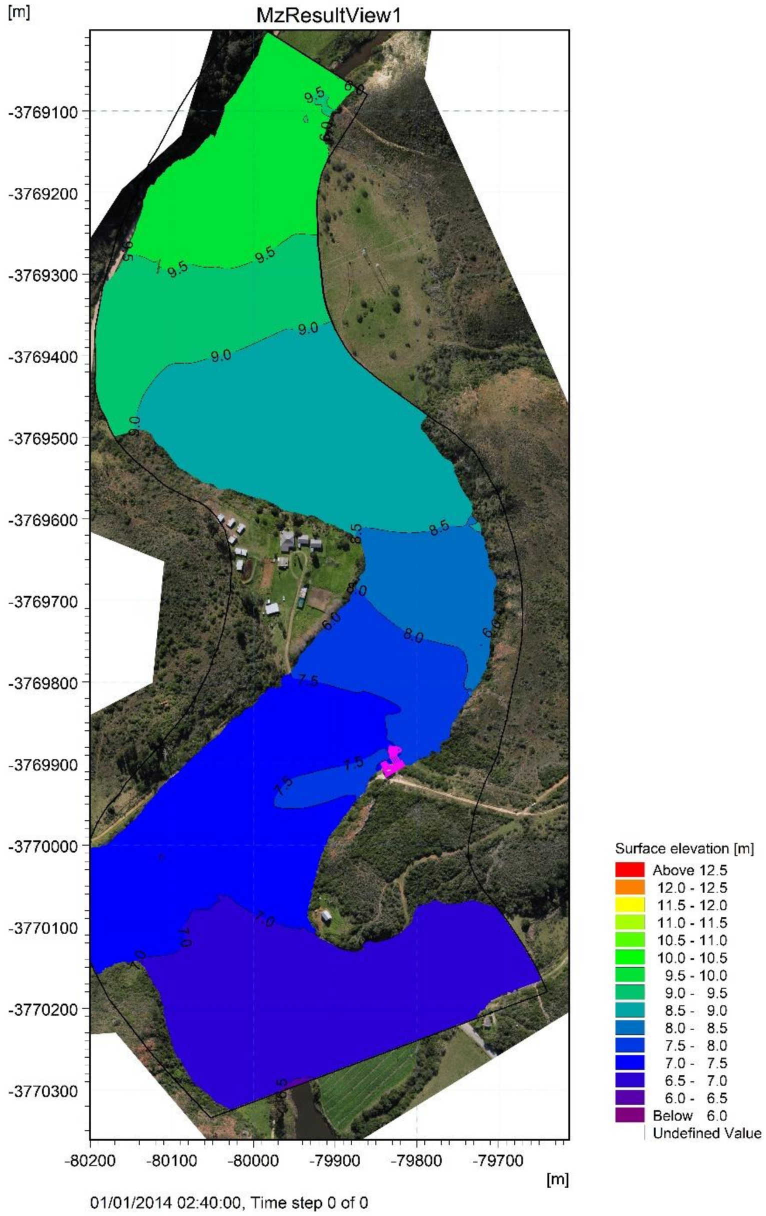


Figure J-13: Simulated water levels during the 100-year flood peak

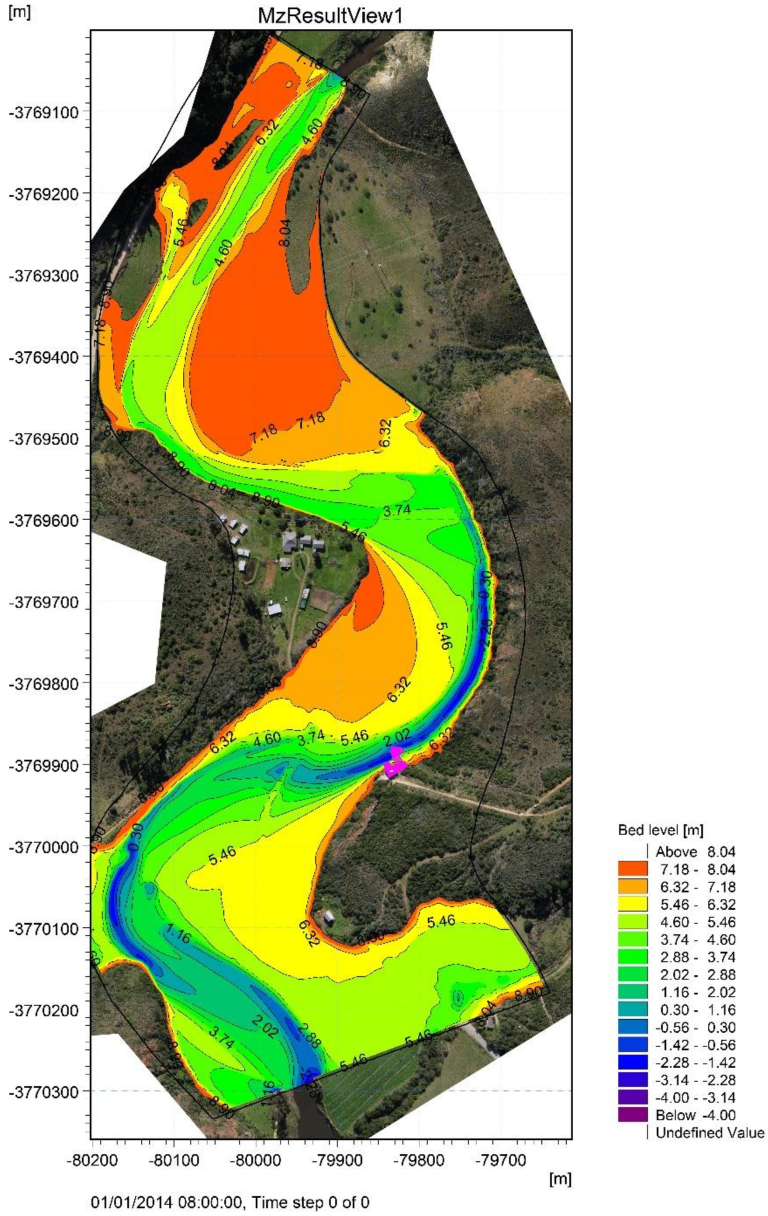


Figure J-14: Simulated bed levels at the end of 100-year flood peak

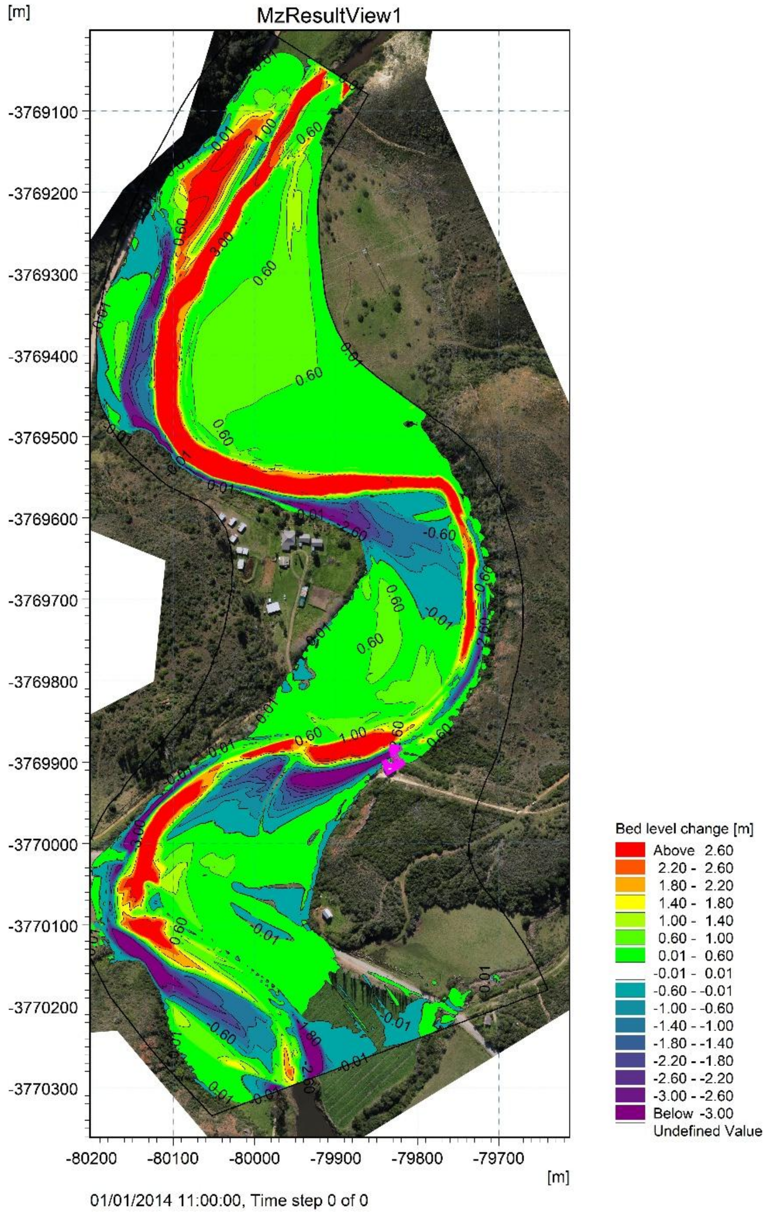


Figure J-15: Simulated bed level change at the end of 100-year flood peak

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