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Feasibility Study Report

Support of the Water Reconciliation Strategy for the Algoa Water Supply System

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DEPARTMENT OF WATER AND SANITATION

Directorates: National Water Resource Planning and Options Analysis

Support of the Water Reconciliation Strategy for the Algoa Water Supply System

FEASIBILITY STUDY REPORT

Final: July 2020

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SUPPORT OF THE WATER RECONCILIATION STRATEGY FOR THE ALGOA WATER SUPPLY SYSTEM

APPROVAL

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Executive Summary

Introduction

The objective of the Feasibility Component of the *Support of the Water Reconciliation Strategy for the Algoa Water Supply System* study is to:

- Limit risks of shortfall in supply to the Nelson Mandela Bay Municipality (NMBM) and the Lower Sundays River Government Water Scheme (LSRGWS);
- Remove potential operating system constraints for the sustainable delivery of bulk Orange River water supply to the LSRGWS and NMBM, for water requirements up to 2040; and
- Limit operational risks to acceptable levels.

The existing Scheepersvlakte Balancing Dam is a balancing facility for water supply to the Lower Sundays River Water User Association (LSRWUA) and the Nelson Mandela Bay Municipality (NMBM), and for emergency supply. The capacity of the balancing dam (769 000 m^3 in 2014) is however not sufficient to meet the dual purpose of supplying the Nooitgedagt Water Treatment Works (WTW), as well as reducing the time of delivery to downstream irrigators. The lack of sufficient balancing capacity also negatively affects the maintenance of the canals.

The focus of the feasibility study investigation is on providing additional balancing storage in addition to the existing Scheepersvlakte Balancing Dam. The main purpose of the proposed new balancing dam, at the Lower Coerney site, is to eliminate the operational and balancing storage limitations imposed by Scheepersvlakte Dam.

After the investigation of several potential dam sites, the Lower Coerney site was found to be the most favourable site for the proposed new balancing dam for emergency water supply to NMBM. The proposed dam is referred to as ‘Coerney Dam’ in this report, as there is no Upper Coerney Dam.

This report documents the feasibility design of the Coerney Dam and related conveyance infrastructure.

Overview of the Scheme

Transferred Orange River water, which is released from Darlington Dam in the lower Sundays River, is diverted from Korhaansdrift Weir into the Kirkwood primary canal. The canal discharges freely into the existing Scheepersvlakte Dam, from where water is distributed further to irrigators and to NMBM, via the nine (9) km long supply pipeline to Nooitgedagt WTW. This pipeline is designed to convey 280 Mℓ/day.

The schematic layout of the proposed project is shown in **Figure E1**.

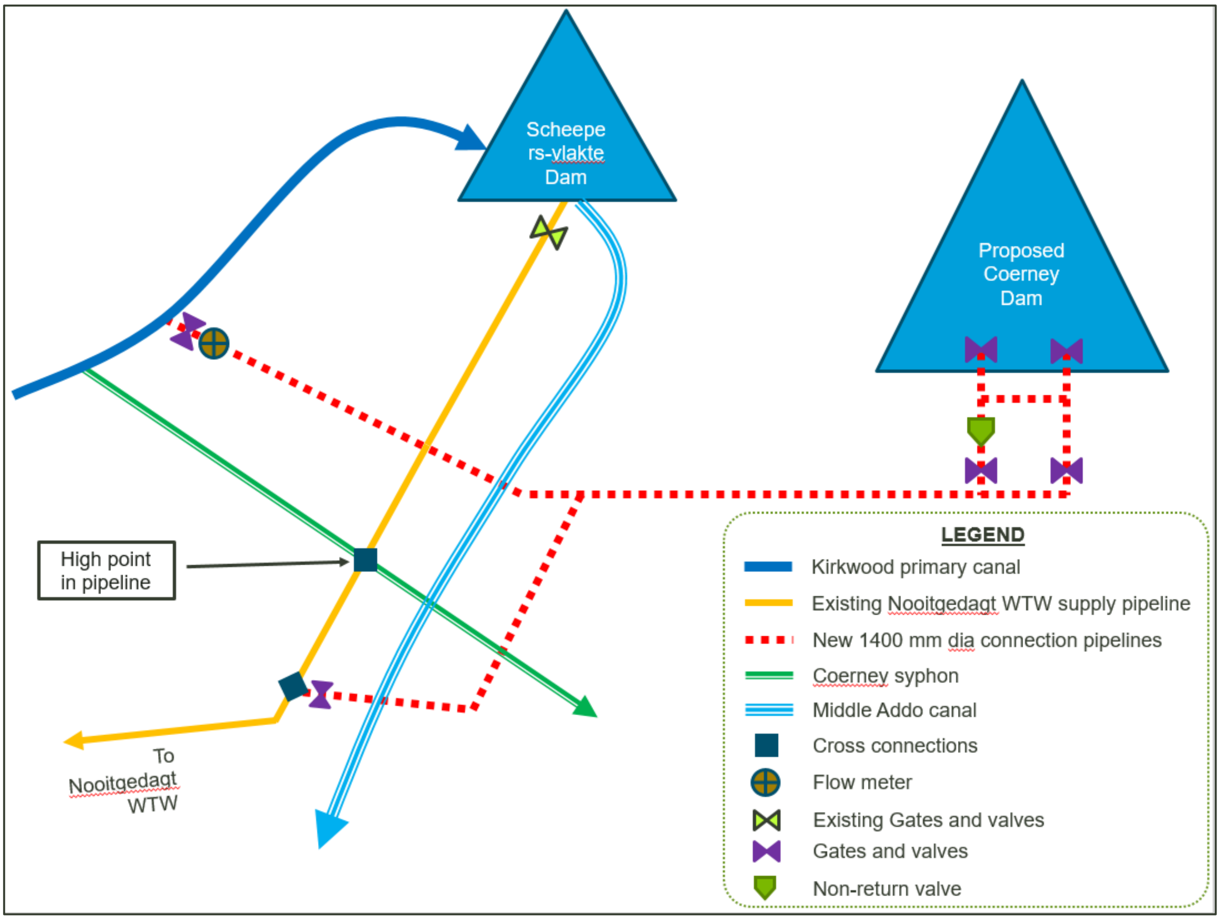

Figure E1: Schematic layout of the proposed Coerney Dam and connecting pipelines

The proposed location of the Coerney Dam is upstream of the Coerney Siphon outlet in a valley east of and adjacent to the existing Scheepersvlakte Dam. The proposed dam is an earthfill embankment dam with a side channel spillway. The dam is sited at a position with an elevation such that it can be filled under gravity via a new supply pipeline from the Kirkwood primary canal. It will also be able to supply the Nooitgedagt WTW under gravity via a new connecting pipeline to the existing 1 400 mm Nooitgedagt WTW pipeline. The main advantage of the scheme is that it can be operated under gravity.

The proposed Coerney Dam will be supplied with water from the Kirkwood primary canal, through a new 1 400 mm diameter steel pipeline, by abstracting water from a new offtake on the canal, between the existing long weir and the existing syphon intake. The head at the long weir will provide the maximum available head for filling the new Coerney Dam.

The proposed balancing dam works will not have any impact on the operation of the current supply pipeline. Flow to the WTW will still be controlled from the downstream end of the pipeline. However, instead of being supplied from the outlet of the Scheepersvlakte Dam, it is proposed that the WTW be supplied from a combination of the proposed new intake at the long weir and from the Coerney Dam.

Topographical survey

A topographical survey was completed by DWS Survey Services: Southern Operations (National Water Resource Infrastructure) for the Coerney Dam site. The survey was updated and expanded to include the immediate surrounding infrastructure. The available survey data is considered sufficient to undertake the feasibility, preliminary and detailed designs of the proposed infrastructure.

The contours of the existing 1 m contour plans from 1977 and 1984, which were compiled from aerial photography for the design of the Lower Sundays River Government Water Scheme (LSRGWS), were digitised. Nine test sections were surveyed for the Lower Coerney site, to compare and verify the digitised data to the actual ground data, which resulted in a good match.

Geotechnical Investigation

Geotechnical investigations for the options analysis comparison between the two most favourable dam sites, namely the Upper Scheepersvlakte Dam and Lower Coerney Dam sites were conducted to inform the selection of the preferred dam site. The geotechnical investigations included the following elements:

- Geophysical (resistivity) surveys;
- Test pitting including the additional test pitting for the supplementary investigation using a tracked excavator;
- Rotary core drilling;
- In-situ field testing including standard penetration tests (SPTs);
- Packer water loss (Lugeon) testing;
- Sampling and laboratory testing; and
- Interpretation, analysis and reporting.

Following the selection of the preferred site (Lower Coerney) a more detailed test pit investigation was conducted at the site using a tracked excavator, to collect supplementary and supportive data required for the feasibility design of the Coerney Dam.

Coerney Dam feasibility level design

Dam characteristics

The proposed dam is a homogeneous earthfill embankment dam. There is some zoning of the embankment fill for slope protection, rip-rap on the upstream face and crushed stone on the downstream face, and internal filter drains. The upstream face is 1V:3H and the downstream face is 1V:2H. The lowest level at the valley bottom is 81.5 masl, which with a non-overspill crest (NOC) level of 102.0 masl results in a maximum wall height of 20.5 m. The full supply level (FSL) is 98.2 masl which gives a maximum water depth of 16.2 m and a storage capacity of 4.69 million m^3. This wall height (*medium* size dam), along with the expected *high* hazard rating, results in a Category III dam safety classification. The basin storage volume excludes the volume of material proposed to be excavated from the basin for the main fill material.

Geotechnical findings

The geotechnical investigations have shown that the material in the basin does not have enough differentiation between core and general fill shell zones for zoned embankment construction. A homogeneous embankment design, which makes use of a semi-pervious to impervious fill for the entire embankment fill, is therefore the only option for this site. Sand, aggregate and rock are not available on site for the filter zones, embankment protection and concrete aggregates, and will need to be imported.

The geotechnical investigations at the dam site have identified that the core trench excavation should extend past a potential seepage path layer of reworked terrace gravels. The depth ranges from 7 m to 8 m on the left abutment, to 4 m in the river section and 3 m to 5 m deep on the right abutment.

Spillway and floods

The foundation of the spillway was also investigated, focusing on the left abutment, which has deep foundations to suitable bedrock. The limited geotechnical data on the right abutment indicated that siting the spillway here could have shallower founding depth. The siting of the spillway on the right abutment was also considered and found to result in lower construction costs. However, there are some drawbacks to this arrangement, most notably; the spillway would need to cross the access road and supply pipeline. This arrangement has increased risks, and it would complicate operation and maintenance of the pipeline. This option is also not supported by LSRWUA or NMBM. It is therefore recommended that the spillway be sited on the left abutment.

The dam, which is classified as Category III, has a recommended design flood (RDF) equal to the 1:200 year flood. This flood has an incoming flow peak of 143 m^3/s, which will be attenuated to 110 m^3/s, after level pool routing through the dam basin and spillway. The SEF is equal to the probable maximum flood with a peak inflow of 835 m^3/s, which will be attenuated to 753 m^3/s.

Two spillway types were considered, an in-line ogee overflow spillway and a side channel ogee overflow spillway. The side channel spillway, sited on the left abutment, was found to be the most favourable option.

The chosen side channel spillway has an ogee shaped overflow crest with a crest length of 50 m. The side channel has a trapezoidal cross-section with a base width of 20 m and side slopes of 1V:0.5H. Water then flows into a trapezoidal discharge channel with a width of 20 m and side slopes of 1V:1H. The discharge channel is lined with reinforced concrete to a depth of 1.7 m, which is equal to the flow depth of the safety evaluation flood (SEF) plus freeboard. The spillway terminates in a stilling basin at the foot of the abutment slope, which then returns subcritical flow to the low point in the river channel.

Freeboard

The required freeboard of the dam was calculated to be 3.64 m. The freeboard was determined for a category III embankment dam using the maximum flood levels of the attenuated floods. The freeboard provided is 3.8 m.

Supply pipeline and inlet/outlet works

A new 1400 mm diameter steel pipeline will connect the dam to the existing water supply scheme pipeline of 1400 mm diameter, which conveys water to the Nooitgedagt WTW. This new pipeline will link the new offtake, which is to be located on the Kirkwood primary canal, to the existing supply pipeline and the dam. The supply pipeline to the dam will bifurcate into an inlet and outlet branch at the dam wall and reduce to a 1200 mm diameter at the downstream outlet chamber. The pipes will then reduce again to 1000 mm diameter before being encased in reinforced concrete through the embankment. The pipes will connect to a wet well tower in the dam basin. The tower is accessible from the embankment crest via a pedestrian walkway. The outlet tower will have two inlet levels, one at 86.0 masl and another at 92.0 masl. The inlets can be isolated with gates operated from the tower.

Access, river diversion and legislative requirements

The dam will be accessed via a road extending from the downstream end of Scheepersvlakte Dam after crossing the river downstream of Scheepersvlakte Dam and its spillway.

The river diversion strategy for the construction of Coerney Dam should be greatly simplified due to the apparent absence of regular flow in the river channel. It is expected that no regular river flows will need to be diverted during construction. Provision is made for a coffer dam with diversion.

Conveyance Infrastructure feasibility level design

Pipeline Design

The proposed conveyance infrastructure comprises two gravity pipelines, namely a pipeline supplying water from the Kirkwood Primary canal to the proposed Coerney Dam, and a pipeline supplying water from the proposed Coerney Dam to a tie-in point on the existing Nooitgedagt WTW pipeline (refer to **Figure E1**).

The main advantages of the proposed scheme are that the proposed Coerney Dam would increase the raw water storage capacity of NMBM. The high point in the existing Nooitgedagt WTW gravity main would be bypassed, to increase the hydraulic capacity during periods with low water levels in the dam.

The hydraulic calculations of both pipelines are based on a design capacity of 280 Mℓ/d (3.24 m^3/s) and the Coerney Dam water levels at a minimum operating level of 86 masl and a full supply level of 98.2 masl. The Hazen-Williams equation was used to determine the operating level required for a flow of 280 Mℓ/d. The results were compared against the Depth-Storage Curve for the dam to compare the percentage storage versus the minimum water level required to discharge the maximum flow of 280 Mℓ/d. A storage capacity of only 17% would be required for a DN 1400 pipeline to deliver this required maximum flow. A flow of 106.6 Mℓ/d (1.85 m^3/s) can be discharged through a DN 1400 pipeline with the dam level at the minimum operating level, i.e. almost 40% of the maximum flow rate.

Based on the hydraulic gradient lines it would be possible to discharge 280 Mℓ/d from the Kirkwood Canal to the Coerney Dam, even when the dam is at full supply level. A residual pressure of approximately 3 m would be available at the tie-in point to the existing Nooitgedagt WTW supply pipeline.

Glass reinforced polyester (GRP), ductile iron and steel pipes were considered as suitable pipe materials, based on the pipeline diameter and expected working pressures. Given the advantages of steel pipes, they are recommended as the preferred pipe material for the proposed pipelines.

A preliminary wall thickness calculation was undertaken based on limited geotechnical information, hydraulic analyses and external loads. Based on the assumptions and calculations, the proposed pipelines will be DN 1400, Grade X52 steel with a yield strength of 358 MPa and a recommended wall thickness of 10 mm. The maximum soil cover of 3.4 m will have to be adhered to during the detailed design of the vertical alignment of the pipelines. A wall thickness of more than 10 mm might be required if the E-value of the native soil is worse than expected or if the E-value of the bedding material is lower than anticipated.

Connections and associated impacted infrastructure

The proposed Coerney Dam will be supplied from the Kirkwood primary canal through a DN 1400 pipeline, which will also be used to release water from the dam to the tie-in point on the existing Nooitgedagt WTW pipeline. The pipe for supplying water to and from the dam will bifurcate into an inlet and outlet branch at the outlet chamber at the downstream toe of the embankment. The inlet branch will have an isolation valve for shutting off supply when the dam is full, and the outlet branch will be fitted with a non-return valve and an isolation valve upstream and downstream.

The offtake from the Kirkwood primary canal will be located downstream of the Coerney syphon intake, and just upstream of the long weir, which will provide head to the new intake. It is proposed that the new offtake comprises an adjustable weir that would allow regulating of the flow that could be discharged from the canal to the WTW or the Coerney Dam.

A connection will be made into the existing Nooitgedagt WTW supply pipeline downstream of the cross connection with the Scheepersvlakte pipeline, and downstream of the existing high point in the existing supply pipeline.

The Middle Addo canal will have to be crossed at two locations. It is recommended that the pipeline be installed over the canal by means of a pipe bridge, not to impact the integrity or operation of the canal, and to facilitate easier maintenance.

The existing Nooitgedagt WTW supply pipeline will also have to be crossed with the proposed new pipeline.

An additional syphon under the Sundays River on the existing Nooitgedagt WTW supply pipeline is recommended. The purpose is to reduce the risk of supply failure and to mitigate the risk of the new balancing storage being located on the opposite side of the river, relative to the WTW. It is recommended that the new syphon be located upstream and separate from the existing syphon. Apart from doubling the syphon it is also recommended that an adequate stockpile of replacement pipes be kept on site, to enable quick repair of the pipeline in case of failure.

Feasibility-level Cost and Implementation Analysis

Coerney Dam feasibility level cost estimate

The costing and main design assumptions are discussed in this report. The cost estimate for the construction of the proposed Coerney Dam and associated works is shown in **Table E1**.

Table E1: Coerney Dam cost estimate (excluding 15% VAT)

No	Description	Amount, rounded (Rand million)
1	Construction cost	156.82
2	Preliminary and General Items (50%)	78.41
	Sub-total	**235.24**
3	Access and electrical supply	2.75
	Sub-total	**237.99**
4	Contingencies (25%)	59.50
	Total	**297.48**

The operation and maintenance costs are estimated as a percentage of the construction cost, divided into three categories: civil, mechanical and the dam wall. The cost of the river diversion, preliminary and general cost items and professional fees are excluded from the construction values below. The estimated annual operation and maintenance costs are as shown in **Table E2**.

Table E2: Dam Operation and Maintenance costs (excluding 15% VAT)

No	Description	Percentage	Construction Cost (million)	Annual Cost (Rand million)
1	Civil works	0.5%	89.653	0.448
2	Mechanical works	4.0%	18.750	0.750
3	Dam (embankment)	0.25%	78.564	0.196
Total			**186.967**	**1.394**

Conveyance infrastructure cost estimate

The costing and main design assumptions are discussed in this report. The cost estimate for the construction of the conveyance infrastructure is shown in **Table E3**.

Table E3: Conveyance infrastructure cost estimate (excluding 15% VAT)

No	Description	Amount, rounded (Rand million)
1	Kirkwood Canal off-take	1.87
2	Inlet/outlet to Coerney Dam	6.83
3	Main connecting pipeline	32.76
4	Tie-in to Nooitgedagt pipeline	1.28
5	Crossing of Middle Addo canal	0.81
6	Syphon under Sundays River	1.21
	Sub-total (a)	**54.76**
7	Preliminary and General Items (50%)	27.38
	Sub-total (b)	**82.14**
8	Contingencies (15%)	12.32
	Total	**94.46**

The operation and maintenance costs are estimated as a percentage of the construction cost, divided into two categories: civil and mechanical. The cost of contingencies is included, and the costs of the professional fees and preliminary and general items are excluded from the values below. The operation and maintenance costs for the conveyance infrastructure are shown in **Table E4.**

Table E4: Conveyance Infrastructure Operation and Maintenance Costs (excluding 15% VAT)

No	Description	Percentage	Construction Cost (Rand million)	Annual Cost (Rand million)
1	Civil works	0.5%	56.809	0.284
2	Mechanical works	4.0%	6.164	0.247
	Total		**62.972**	**0.531**

Other miscellaneous project costs

The other project costs include land acquisition for the dam and pipeline, and professional fees (design, applications and licencing, etc.) are shown in **Table E5**.

Table E5: Estimate of other miscellaneous costs (excluding 15% VAT)

No	Description	Amount (Rand million)
1	Land acquisition	21.950
2	Professional fees for dam (10% of construction cost)	29.748
3	Professional fees for conveyance infrastructure (10% of construction cost)	9.446
	Total	**61.144**

Total project cost estimate

The total estimated project costs are shown in **Table E6**.

Table E6: Total estimated project costs

No	Description	Amount (Rand million)
1	Balancing dam	297.481
2	Conveyance infrastructure	94.456
3	Professional fees	39.194
	Value Added Tax (15%)	64.670
4	Land acquisition	21.950
	TOTAL (January 2020 prices)	**517.753**

The project completion cost estimate for commencement of construction in 2025, with escalation of 6.5% per annum from the base year of 2020, is shown in **Table E7**.

Table E7: Total estimated project completion costs (including 15% VAT)

Year	Escalation rate	Notes	Present cost (R million)	Future cost (Base year 2020) (R million)
2023	6.5%	Land acquisition + 40% of Professional fees	39.979	48.293
2024	6.5%	33% Construction value + 20% Professional fees	159.257	204.879
2025	6.5%	33% Construction value + 20% Professional fees	159.257	218.196
2026	6.5%	33% Construction value + 20% Professional fees	159.257	232.379
		TOTAL	**517.751**	**703.747**

Legislative Compliance

Water use licencing and dam safety

A water use licence will need to be obtained for storing water, and for affecting and altering the banks of a river (Section 21, National Water Act, 1998). This licence application is included in the scope of work for the EIA study.

In terms of Chapter 12 of the National Water Act (NWA), the dam will be a "*dam with a safety risk*". This means the design and construction of the dam must comply with the dam safety regulations (2012).

Ecological Water Requirement

The proposed Coerney Dam will be situated in a small ephemeral tributary of the Coerney River, which joins the lower Sundays River near the Nooitgedagt WTW. This tributary has no defined channel or any evidence of flow and has seemingly not seen flows for over 20 years. DWS Directorate: Resource Directed Measures should determine whether to undertake an EWR determination study for non-perennial systems for this tributary.

Environmental compliance

In terms of the National Environmental Management Act (No. 107 of 1998, as amended) (NEMA), an Environmental Authorisation for the proposed project will be required. The Environmental Impact Assessment process for the proposed dam is expected to start in 2021.

It follows a multi-staged approach to environmental impacts, public participation and stakeholder engagement stipulated by these regulations, as well as various specialist studies.

Implementation Arrangements

Affected Land and Land Acquisition

The portion of land upon which the dam is to be located is known as Portion 7 of Scheepersvlakte No. 98, owned by Scheepersvlakte Farms (Pty) Ltd. The footprint of the proposed Coerney Dam overlaps with portions of the planned future Scheepersvlakte Farms development. The overlaps occur within the full supply level of the dam, the 1:100 year flood line in the basin, as well as portions of the dam wall and the spillway. The total area where the future planned orchards and dam infrastructure and 1:100 year flood line overlap is estimated at 36 ha. The layout of the new citrus developments would need to be revised.

The proposed storage capacity of the Coerney Dam includes an allowance for the farm dam, namely one week of storage for Scheepersvlakte Farms, equal to 150 000 m^3 of storage.

During the detailed design of the pipeline route the various landowners who could be affected should be consulted.

Operation and maintenance

The proposed Coerney Dam will be filled, and topped up, over a lengthy filling period through gravity supply. The existing Scheepersvlakte Dam and proposed Coerney Dam, although filled from the same source, should be operated separately under normal operation. The proposed Coerney Dam will be used as balancing storage for NMBM and Scheepersvlakte Farms. Scheepersvlakte Dam will revert to its original function and will only be used as balancing storage for irrigators.

Institutional Arrangements

The proposed Coerney Dam will be implemented and owned by DWS. The use of the dam is solely for the NMBM, with the additional storage for use by the Scheepersvlakte Trust. It has been agreed that the new balancing dam should be operated by the LSRWUA.

Further Investigations for Detailed Design

The accuracy of the contour surveys undertaken for the dam site and adjacent area is considered suitable for detailed design of the dam. However, it is recommended that a centreline survey of the final pipeline routes be undertaken prior to design and construction.

The current level of information of the geotechnical investigations undertaken should be adequate if a conservative design approach is followed.

Investigation of the dam basin indicates that there is sufficient suitable material available for the construction of a homogeneous embankment.

A geotechnical investigation should be undertaken along the proposed pipeline routes and at the proposed chamber locations to inform the detailed design.

Other construction materials, such as coarse aggregate for concrete, sand for filters and riprap for slope protection are not found in the basin or near the site, and will have to be imported from commercial sources.

The geotechnical investigations along the pipeline route are required to determine the amount of bedding material that is available on site and the volume that will need to be imported from a commercial source.

A hydraulic model study of the spillway configuration is required to optimise the detailed design of these components.

Site specific determination of the magnitude of the recommended design flood and safety evaluation flood should be undertaken for the proposed dam site.

Project Implementation

The project has been declared as an Emergency Works on 10 July 2020 by the Minister. This will enable the detailed design to be undertaken by DWS in parallel with the environmental impact assessment process, thereby greatly reducing the time for implementation.

The proposed Coerney Dam and associated works will form part of the Lower Sundays River Government Water Scheme and will therefore be implemented as a Government Waterworks and funded by National Treasury.

Conclusions

It is concluded that it is feasible to construct the 4.69 million m^3 earthfill embankment Coerney Dam, with a side channel spillway and intake tower at the 'Lower Coerney' dam site, to improve balancing storage of transferred Orange River water to the Nooitgedagt WTW of NMBM. The dam will be filled from the Kirkwood primary canal, by abstracting water from a new offtake on the canal, between the long weir and syphon intake. A new 1 400 mm diameter steel pipeline will supply the dam with water from the offtake under gravity. In addition to the alterations around the existing Scheepersvlakte Dam, an additional syphon on the supply pipeline to the Nooitgedagt WTW under the Sundays River should be constructed.

The available survey data is considered sufficient to undertake the preliminary and detailed designs of the proposed infrastructure, but further geological investigations are required.

The joint use of water from the dam by NMBM and the private Scheepersvlakte Farm developer has been agreed by the parties concerned.

Coerney Dam will be operated and maintained by LSRWUA as part of the existing LSRGWS.

Recommendations

The following recommendations are applicable to the detailed design and construction phases of the project:

Topographical Survey

1) A ground centreline survey should be done along the final pipeline centreline route, prior to construction commencing. This will serve as a final check on the pipeline vertical alignment and soil cover depths.

Geotechnical Investigations

2) Involvement of a geotechnical specialist during construction is essential. Activities would include regular inspection of all excavated faces and cut slopes from a stability point of view, oversight of any further geotechnical exploration and quality assurance testing, confirmation of bedrock depth at the spillway, engineering geological mapping of the cut-off trench and recording of the as-built details, etc.
3) A geotechnical investigation should be undertaken along the proposed pipeline routes and at the proposed chamber locations to inform the detailed design. The geotechnical investigation should also include a soil resistivity survey and testing for sulphate reducing bacteria to inform the coating selection and the need for cathodic protection.

Coerney Dam Design

4) The assumptions made in the determination of the desired dam storage volume (e.g. siltation both from the canal and catchment, "normal use" volume and resulting siltation, infiltration losses) should be checked and refined.
5) Filter sand, aggregate and rock sources, other than what has been identified thus far, could be investigated and identified. There was no investigation into site specific borrow pits outside of the dam basin.
6) The embankment zoning and dimensions are based on typical values for embankment dams of this size using similar materials. The zoning dimensions must thus be designed based on the actual material properties and design constraints for the zones. These

include elements such as: filter zone thicknesses and spacing depending on the target filter sand and permeability, the core trench bottom width depending on the permeability of the target fill material as well as the permissible seepage losses (to be clarified), and the embankment slopes and slope stability.

7) The current level of study compared in line and side channel ogee spillways on the left and right abutments based on the data available. This is mostly based on limited investigations on the right abutment and more in-depth investigations on the left abutment. The assumptions and inferences made in the current proposed design must thus be refined.

8) Additional consideration and investigation into the possibility of providing an auxiliary and service spillway arrangement should be done. The service spillway would contain the Recommended Design Flood and an auxiliary spillway, which can be unlined, would accommodate the Safety Evaluation Flood.

9) Determination of a site-specific RDF and SEF for detailed design of the dam and spillway.

10) Undertake a hydraulic model study of the spillway configuration.

Conveyance Infrastructure Design

11) An estimate is required of the volume of suitable pipeline bedding material that will need to be imported, as well as locating suitable sources.

12) The wall thickness calculations of the proposed pipelines should be confirmed once the additional geotechnical information becomes available.

13) During the detailed design, the pipeline route will need to be confirmed after discussions with affected land owners and authorities. Some refinements to the routes may be required due to developments subsequent to the feasibility design.

14) A decision is required on the preferred pipe lining system to be used (i.e. cement mortar or epoxy), as well as the coating system (i.e. polyurethane, medium density polyethylene (i.e. Sintakote), 3LPE and polymer modified bitumen).

15) The exact positions of all connections and impacted associated infrastructure must be verified during the detailed design of the pipelines.

16) Independent quality control inspections of the pipes, at the factory and on site, must be included in the construction tender documents.

17) As-built drawings and/or information of the existing syphon under the Sundays River will need to be obtained during the detailed design phase of the project.

18) The position of the new syphon, upstream of the existing syphon, needs to be determined.

Legislative Compliance

19) The water use licence application for storing water and affecting and altering the banks of a river (Section 21 (b), (c) and (i), National Water Act, 1998) is included in the scope of work for the EIA study Chapter 4.8.

20) Dam safety regulation requirements:
    - Applications for licences for complying with the dam safety regulations will need to be completed before certain tasks may continue.
    - A licence to construct must be issued by the Dam Safety Office (DSO) before any construction may commence.
    - Before the bottom outlets of the dam are closed, thereby commencing the impounding of water, the licence to impound must be obtained from the DSO.

21) The DWS should consider the merit of undertaking an EWR determination study for non-perennial systems for the small ephemeral tributary of the Coerney River, in which the Coerney Dam will be situated.

22) DWS should undertake a comprehensive EIA process in accordance with the National Environment Management Act (No. 107 of 1998, as amended) (NEMA) and the 2014 EIA Regulations (GN R982 – 985, as amended). The EIA process is a legal requirement to obtain Environmental Authorisation for implementation of the project from the Department of Environment, Forestry and Fisheries.

Implementation Arrangements

23) The irrigation development on Scheepersvlakte Farm and the implementation of the dam need careful planning and coordination. An agreement for the joint use of water will need to be agreed. The location of the Scheepersvlakte Farm offtake from the dam should be confirmed.

24) Further investigations for detailed design should be undertaken, inclusive of topographical surveys, geotechnical investigations, construction materials, hydraulic model study, electrical power supply and site-specific flood determination.

25) The revision of contractual arrangements with LORWUA should be considered.

26) The project has been declared as an Emergency Works by the Minister to enable the detailed design to be undertaken by DWS in parallel with the EIA process. The RID needs to be completed as soon as possible and issued to DWS Infrastructure

Development to formalise the early implementation of the project as an Emergency Works.

27) Funding needs to be secured from National Treasury to enable construction of the project to commence as soon as the detail design is complete and Environmental Authorisation has been received.

28) An Addendum to the RID needs to be issued after the Environmental Authorisation has been received.

Contents

Appendices

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Abbreviations

APP	Approved Professional Person
AWWA	American Water Works Association
DN	Nominal diameter
DEFF	Department of Environment, Forestry and Fisheries
DSO	Dam Safety Office
DWAF	(Previous) Department of Water and Forestry
DWS	Department of Water and Sanitation
ECPHRA	Eastern Cape Provincial Heritage Resources Agency
EIA	Environmental Impact Assessment
EMPr	Environmental Management Programme
EWR	Ecological water requirements
FSL	Full supply level
GN	Government Notice
GPS	Global Positioning System
GRP	Glass Reinforced Polyester
GWS	Government Water Scheme
ha	hectares
HGL	Hydraulic gradeline
HW	Hazen-Williams friction coefficient
ID	Internal diameter
LSRGWS	Lower Sundays River Government Water Scheme
LSRWUA	Lower Sundays River Water User Association
m	metre
Mℓ	Megalitre
Mℓ/d	Megalitre per day
masl	Metres above mean sea level
mamsl	Metres above mean sea level
mm	millimetre
MOL	Minimum outlet level
MPa	Megapascal
NEMA	National Environmental Management Act of 1998
NEM:BA	National Environmental Management: Biodiversity Act
NFA	National Forests Act
NHRA	National Heritage Resources Act
NMBM	Nelson Mandela Bay Municipality
NOCL	Non-Overspill Crest Level

NOC	Non-overspill crest
NWA	National Water Act of 1998
ORP	Orange River Project
PGA	Peak Ground Acceleration
PI	Plasticity Index
PMF	Probable Maximum Flood
RDF	Recommended Design Flood
RID	Record of Implementation Decisions
RL	Related level
SANCOLD	South African National Committee on Large Dams
SANS	South African National Standards
SCS Method	Runoff Curve Number Soil Conservation Service method
SEF	Safety Evaluation Flood
SPT	Standard Penetration Test
WAR	Water Allocation Reform
WSS	Water Supply Scheme
WTW	Water Treatment Works
WUA	Water User Association
WUL	Water Use Licence
3LPE	Trilaminate polyethylene coating

1 Introduction

1.1 Study Objective

The objective of the Feasibility Component of the *Support of the Water Reconciliation Strategy for the Algoa Water Supply System* study is to:

- Limit risks of shortfall in supply to the Nelson Mandela Bay Municipality (NMBM) and the Lower Sundays River Government Water Scheme (LSRGWS);
- Remove potential operating system constraints for the sustainable delivery of bulk Orange River water supply to the LSRGWS and NMBM, for water requirements up to 2040; and
- Limit operational risks to acceptable levels.

The focus of this feasibility study investigation is on providing dedicated balancing storage for water supply to the Nooitgedagt water treatment works (WTW), which provides potable water to NMBM. The main purpose of the proposed new balancing dam, at the Lower Coerney site, is to improve operation and provide adequate balancing storage for NMBM.

1.2 Purpose of this Report

The purpose of this report is to document the feasibility design of the Coerney Dam and related conveyance infrastructure, as well as to summarise the topographical survey and geotechnical and materials investigations undertaken. This report is a synthesis of the following reports and sub-reports that have been incrementally produced and approved during the feasibility study:

- Coerney Dam Contour Survey Report;
- Geotechnical Report: Lower Coerney Dam Site Supplementary Investigations;
- Layout and Affected Land and Infrastructure Sub-Report;
- Feasibility-level Engineering Design - Balancing Dam Sub-Report;
- Feasibility-level Engineering Design - Conveyance Infrastructure Sub-Report;
- Feasibility-level Cost and Implementation Analysis Sub-Report; and
- Implementation Support Sub-Report.

1.3 Need for the scheme

The existing Scheepersvlakte Balancing Dam is a balancing facility for water supply to the Lower Sundays River Water User Association (LSRWUA), which also supplies NMBM and for emergency supply. The dam has partly silted up, and by 2014, the dam had a capacity of 769 000 m^3. The capacity of the balancing dam is currently too small to adequately supply NMBM and irrigators, presenting a risk of shortfall in supply, should there be any interruption to the supply. The lack of sufficient balancing capacity also negatively affects the maintenance of the canals. The Scheepersvlakte Dam Remedial Works Project (Naidu Consulting, 2016) concluded that the current operation of the Scheepersvlakte Dam leads to high operational risks for water supply to the NMBM.

Following the expected completion of the Nooitgedagt Water Treatment Works (WTW) Phase 3 in 2022, the WTW will have a maximum capacity of 210 Mℓ/day. The scheme has been designed to cater for peak/back-up supplies from the Nooitgedagt WTW at times when the older infrastructure, from sources to the west of Port Elizabeth, will be requiring maintenance or emergency repairs; in other words, the dam is a balancing dam for emergency water supply to NMBM.

After investigation of several potential dam sites, as documented in the *Options Analysis Report* of this study, 2019 (Report No P WMA 15/N40/00/2517/3), the Lower Coerney Dam site was found to be the most favourable site for the proposed new balancing dam. The proposed dam is referred to as 'Coerney Dam' in this report and future reports, as there is no Upper Coerney Dam. The site was approved for further evaluation and recommended for feasibility design.

1.4 Content of this Report

The various chapters in this report and their content are briefly described hereunder.

Chapter 1: Introduction

Provides a brief introduction to the project and report.

Chapter 2: Background

Describes the bulk conveyance and storage infrastructure conveying Orange River water to NMBM and provides an overview of the Coerney Dam Scheme and its components.

Chapter 3: Topographical Survey

Overview of the topographical survey undertaken at the chosen dam site.

Chapter 4: Geotechnical and Construction Materials Investigation

Overview of the geology and the outcomes and conclusions of the geotechnical investigations undertaken at the chosen dam site.

Chapter 5: Coerney Dam Design

Presents the characteristics and design of the Coerney Dam and appurtenant structures.

Chapter 6: Conveyance Infrastructure Design

Presents the characteristics and design of the connecting pipeline to Nooitgedagt WTW and appurtenant structures, as well as the additional syphon through the Sundays River.

Chapter 7: Cost Estimate

Discusses the dimensions, assumptions and other factors affecting the costing of the various components of the scheme and presents capital, operational and other estimated costs.

Chapter 8: Legislative compliance

Describes water use licensing and dam safety legislation and need for compliance, the potential need for the determination of an ecological water requirement and the environmental requirements and processes that are required to make the proposed option implementation ready.

Chapter 9: Implementation Arrangements

Briefly identifies the various legislative considerations required for the dam and the status of each process, namely the environmental impact assessment, ecological water requirement, water use licencing, and dam safety requirements.

Chapter 10: Further Investigations for Detailed Design

Describes the further investigations that are required to successfully undertake the detailed design.

Chapter 11: Project Implementation

Provides information on the recommended implementation process, as well as the possible timeframe and milestones dates.

Chapter 12: Conclusions

Summarises the conclusions from the feasibility design.

Chapter 13: Recommendations

Lists the recommendations emanating from the feasibility design.

2 Background

This chapter describes the bulk conveyance and storage infrastructure conveying Orange River water to NMBM and provides an overview of and introduction to the Coerney Dam Scheme and its components.

2.1 Bulk Infrastructure

2.1.1 Introduction

The existing water supply system (WSS) that conveys transferred Orange River water from the Sundays River comprises a number of components to abstract, convey, store and distribute the water to the users, both the irrigators of the LSRWUA and the NMBM. The overall WSS layout is shown in **Figure 2.1**. The provision of the additional balancing storage will have an impact mainly on the infrastructure around the existing Scheepersvlakte Balancing Dam, to connect the new dam to the WSS, and to provide the operational functionality and flexibility required.

In the following sections the salient components of the existing WSS are described.

2.1.2 Korhaansdrift Weir

Following the transfer of Orange River water from Gariep Dam via the Great Fish, Little Fish and Sundays rivers, the transferred water flows into Darlington Dam on the Sundays River, from where water is released downstream and then diverted from the Korhaansdrift Weir into the Kirkwood primary canal. The weir has lost most of its limited storage capacity to siltation and can no longer provide balancing storage. The proposed new balancing dam and infrastructure does not have any impact on the weir.

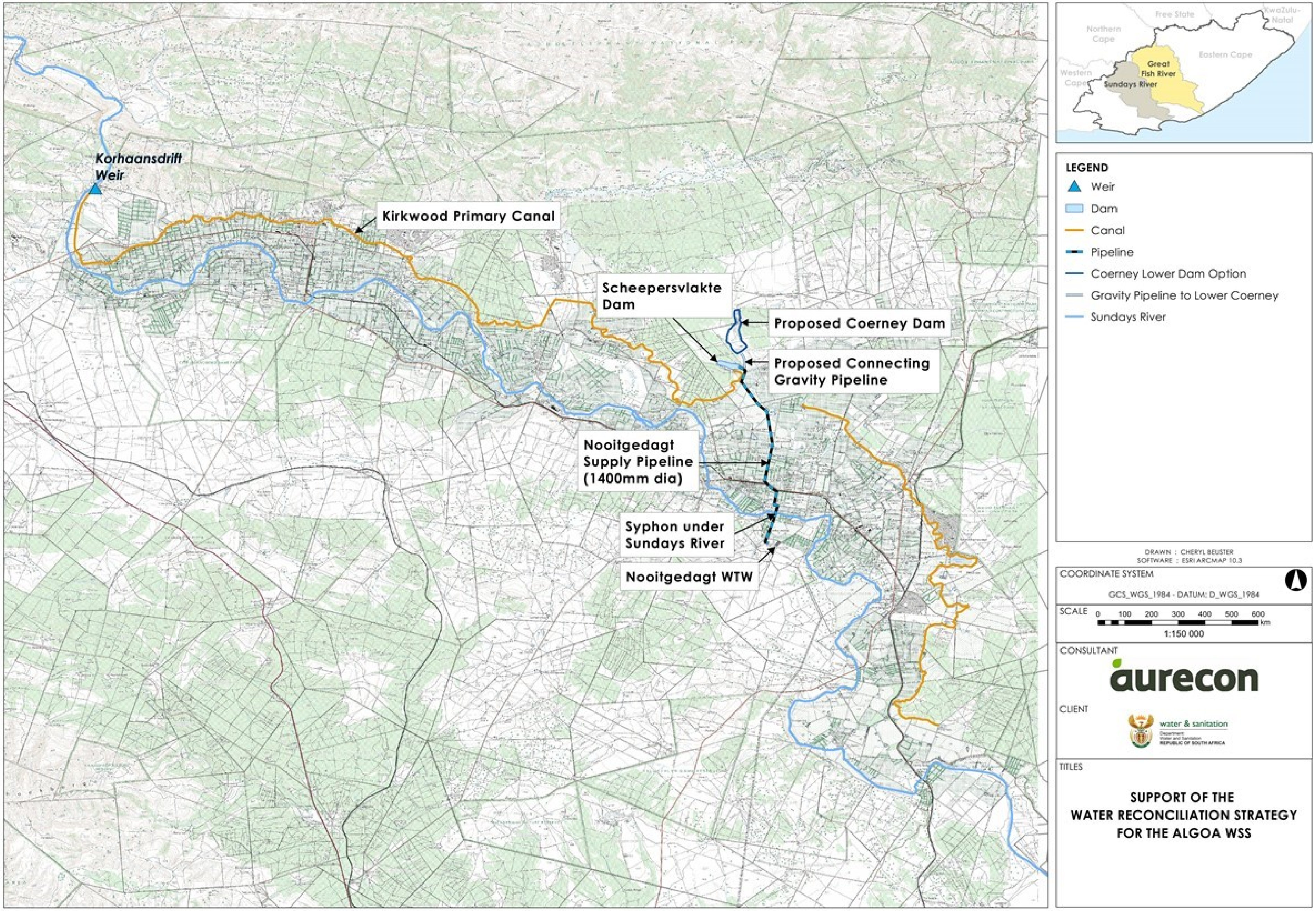

Figure 2.1: **Layout Plan of the water supply system between Korhaansdrift Weir and Nooitgedagt WTW**

2.1.3 Lower Sundays River Canal System

Water diverted at the Korhaansdrift Weir is transported via the main canal, also known as the Kirkwood primary canal, to the Scheepersvlakte Balancing Dam. The design capacity of this canal at the offtake at Korhaansdrift Weir is 22.7 m^3/s. There are three secondary canals, which branch from the Kirkwood canal, to supply the various areas of the scheme, namely Wesbank, Mistkraal and Tregeron/Selborne canals.

The main canal discharges freely into the existing Scheepersvlakte Balancing Dam. Thereafter the water is supplied through the dam's outlet pipes to both the Middle Addo irrigation canal and the Nooitgedagt WTW supply pipeline.

Shortly before the Scheepersvlakte Balancing Dam there is a long weir (at elevation 105.8 m) for controlling the back water in the canal and to provide adequate head for a syphon offtake. The syphon allows abstraction from the canal and crosses under the Middle Addo canal, to supply water to the Coerney canal. The syphon is also cross-connected to the supply pipeline to the Nooitgedagt WTW, which provides a bypass of the dam and its outlet during emergencies or maintenance.

The proposed new balancing dam will be supplied from the main canal, by abstracting water from a new offtake on the canal, between the long weir and the existing syphon intake. The head at the long weir will provide the maximum available head for filling the new Coerney Dam.

2.1.4 Scheepersvlakte Balancing Dam

The Scheepersvlakte Balancing Dam was sized to provide balancing capacity for irrigation water supply only, with an original volume of 820 000 m^3. The dam has since partly silted up, and by 2014, the dam had a capacity of 769 000 m^3. The balancing dam reduces the time it takes to convey water to downstream water users, while balancing any irrigation spills from upstream water users, and is currently also storing water for supply to NMBM. The dam still provides significant operational flexibility to the LSRWUA, depending on the distribution of users requesting water for the following week, downstream of the balancing dam, by reducing the delivery time to the downstream irrigators.

The capacity of the balancing dam is however not sufficient to meet the dual purpose of supplying the Nooitgedagt WTW, as well as reducing the time of delivery to downstream irrigators. The lack of sufficient balancing capacity also negatively affects the maintenance of the canals. The Scheepersvlakte Dam Remedial Works Project (Naidu Consulting, 2016) concluded that the current operation of the Scheepersvlakte Dam leads to high operational risks for water supply to the NMBM. The report made recommendations to alleviate some risks, but this cannot solve the main problem, which is the limited storage capacity.

The proposed balancing storage of the proposed Coerney Dam will thus provide a significantly improved storage buffer. In addition, the use of the full Scheepersvlakte Dam volume for irrigation balancing storage, as originally intended, will provide operational flexibility for the canal system. It will also alleviate the current time constraints for canal maintenance.

2.1.5 Pipeline to Nooitgedagt Water Treatment Works

Water is currently conveyed from the Scheepersvlakte Balancing Dam to the Nooitgedagt WTW by a 9 km long DN 1400 mm diameter steel gravity main, designed to convey 280 Mℓ/day.

The original capacity of the Nooitgedagt WTW was increased to 125 Mℓ/day average and 140 Mℓ/day peak (Phase 2) supply. Phase 3 of the Nooitgedagt WTW upgrade is currently under construction with a final capacity of 160 Mℓ/day average and 210 Mℓ/day peak, which is expected to be available by 2022. From the Nooitgedagt WTW, water is pumped to the NMBM, either via the High-Level Scheme or the Low-Level Scheme.

2.2 Project Layout and Operation

2.2.1 Location and Layout

The scheme is located between Addo and Kirkwood in the Eastern Cape Province of South Africa. The proposed location of the Coerney Dam (**Figure 2.2**) is upstream of the Coerney Siphon on the privately-owned Scheepersvlakte 98 Portion Number 7 of Scheepersvlakte Farms Pty Ltd. It is in the vicinity of the site proposed by Scheepersvlakte Farms for a balancing dam for their proposed irrigation activities.

The project locality and layout are shown on Drawing 112546-0000-DRG-CC-001 in **Appendix A-1** and Drawing 112546-0000-DRG-CC-002 in **Appendix A-2**.

The main advantage of the chosen dam site is that it would be operated under gravity. The dam will be filled by gravity from the Kirkwood primary canal via a new pipeline. The dam will also supply the Nooitgedagt WTW via a new connecting pipeline to the existing 1 400 mm Nooitgedagt pipeline.

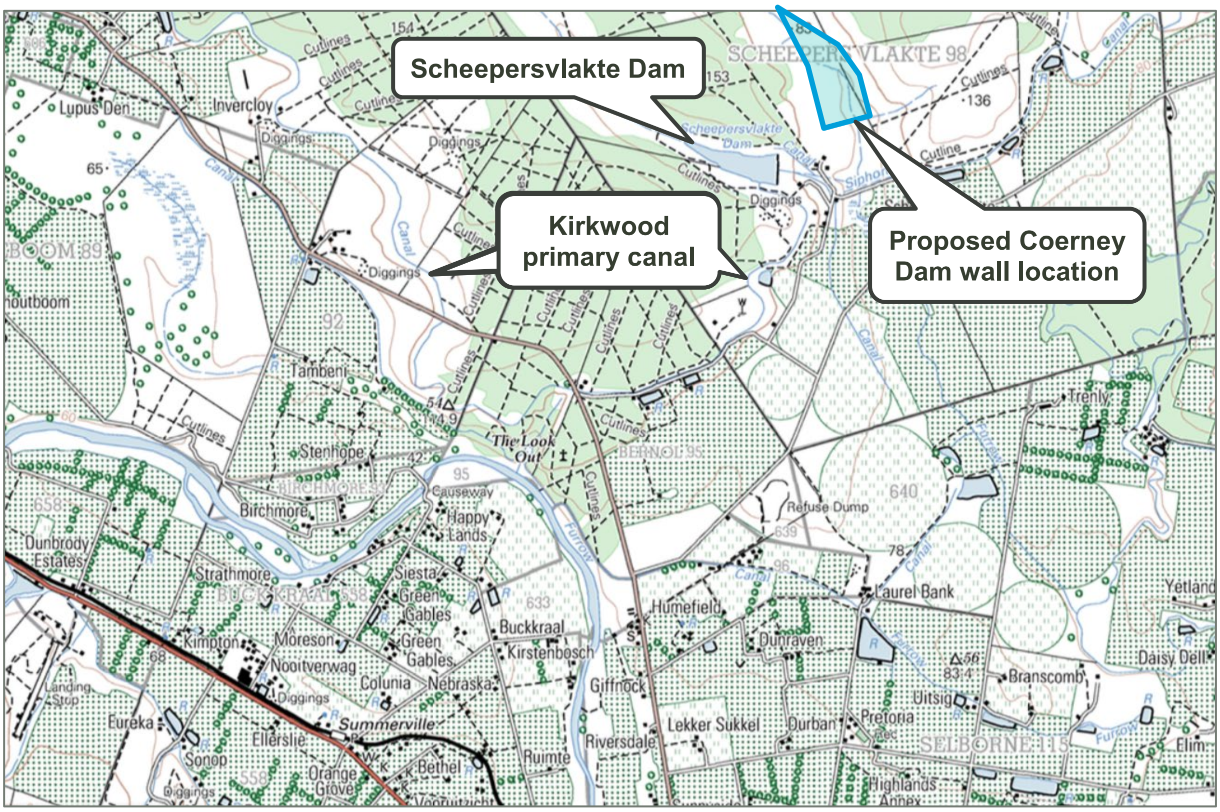

Figure 2.2: Location of the existing Scheepersvlakte and proposed Coerney Dam

The schematic layout of the new infrastructure at the balancing dam site, including existing infrastructure, is shown in **Figure 2.3**.

2.2.2 Proposed Coerney Dam

The proposed Coerney Dam will be situated in a valley to the east of Scheepersvlakte Dam. The proposed dam is an earthfill embankment dam with a side channel spillway. The dam is sited at a position with an elevation such that it can be filled via the proposed supply pipeline from the Kirkwood primary canal under gravity. It will also be able to supply the Nooitgedagt WTW under gravity. This means that no pumping will be necessary for supply to the dam or the WTW.

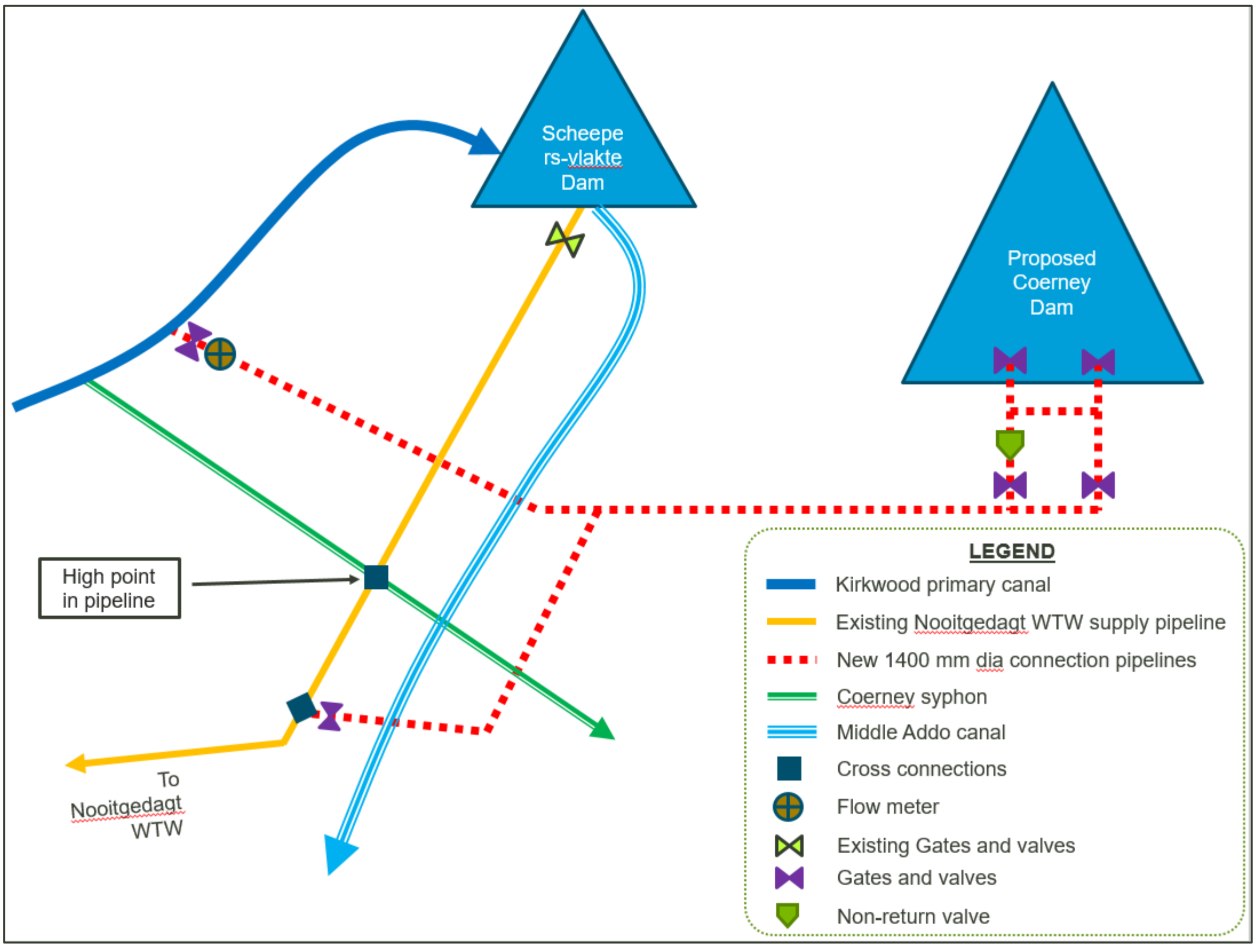

Figure 2.3: Schematic layout of the proposed Coerney Dam and connecting pipelines

The proposed balancing dam's full supply level (FSL) for the proposed storage of 4.69 million m^3 is at 98.2 masl and the minimum outlet level (MOL) is 86.0 masl. The higher flow requirements at the WTW during future upgrade phases will increase head losses due to higher flow velocities. As the supply pipeline is gravity fed, this translates into a minimum drawdown level in the dam to provide each flow rate requirement. These minimum drawdown levels in the dam for increasing flow requirements are set out in **Table 2.1**.

The MOL of the dam is above the minimum draw down levels, for flows up to 200 Mℓ/day. This means that these flows can be supplied to the WTW through gravity, without being restricted or reducing the effective/live available storage in the dam. However, it is also evident that for flows exceeding 200 Mℓ/day, the minimum draw down level, and thus the available storage in the dam, are both reduced.

Table 2.1: Minimum draw down levels in Coerney Dam at different flow rates

Flow (Mℓ/day)	Minimum draw down level (masl)
100	76.8
160	81.0
200	84.9
240	89.6

The dam wall will be fenced to restrict unauthorised access. The dam basin will likely not be fenced, however fencing around the wall and associated infrastructure is proposed to extend to approximately 100 m along the basin rim.

2.2.3 New Connecting and Supply Pipework

The proposed connecting pipework for the new dam is shown schematically in **Figure 2.3**. The proposed Coerney Dam will be supplied with water from the Kirkwood primary canal, through a new 1400 mm diameter steel pipeline. The pipeline will abstract water from a new offtake on the canal. The proposed intake is located at the end of the Kirkwood primary canal, in-between the Coerney syphon inlet and the long weir, shortly before the end of the canal at Scheepersvlakte Dam. Locating the intake just upstream of the long weir will provide head from the weir.

A high point exists on the existing supply pipeline to the WTW, located at the cross connection with the Coerney canal syphon (refer to **Figure 2.3**). The elevation of the cross connection is 87.4 masl, which is higher than the outlet level at Coerney Dam, but well below the FSL. However, it can be seen, referring to **Figure 2.3**, that the high point will limit the minimum drawdown level for the high flow requirements at the WTW (above 200 Mℓ/day). Consequently, it is proposed that a branch pipeline be provided between the new Coerney Dam supply pipeline and the Nooitgedagt WTW supply pipeline, to eliminate this high point in the supply pipeline.

The proposed balancing dam works will not have any impact on the operation of the current supply pipeline. Flow to the WTW will still be controlled from the downstream end of the pipeline. However, instead of being supplied from the outlet of the Scheepersvlakte Dam, it is proposed that the pipeline to the WTW be supplied from a combination of the proposed new intake at the long weir and supply from the Coerney Dam.

To enable the above-mentioned operation, the new pipeline from the Kirkwood primary canal to the Coerney Dam will be provided with a cross connection to the existing supply pipeline from the Scheepersvlakte Dam (pipeline from the dam to the Nooitgedagt WTW).

The section of the current supply pipeline between Scheepersvlakte Dam and the new cross connection will be isolated during normal operation, as it will not normally be used to supply the WTW. However, Scheepersvlakte Dam will still be able to supply water to the Nooitgedagt WTW in case of emergencies.

The new supply pipeline to the Coerney Dam and the bypass pipeline will need to cross the Middle Addo canal. A pipe bridge over the canal is proposed to achieve this, to not impact the canal.

Once the Coerney Dam is in operation, the Scheepersvlakte Dam will function as a balancing dam for irrigation only. Its connecting pipework to the WTW will be isolated, but will remain in place with a connection to the Coerney Dam for emergencies.

Two changes to the Nooitgedagt WTW supply pipeline further downstream are proposed. Firstly, an additional syphon under the Sundays River is proposed. This will reduce the risk of supply failure in the event of damage to the existing syphon. An additional syphon is proposed to mitigate the risk of the new balancing storage being located on the opposite side of the river, relative to the WTW.

Secondly, the replacement of the concrete slab in the inlet structure at the Nooitgedagt WTW with a slab having larger holes is planned. This change will reduce energy losses and improve the maximum flow to the WTW.

At the dam wall the new pipeline from the Kirkwood canal will bifurcate to an inlet and outlet pipe. This arrangement will allow the inflow to be stopped while still allowing outflow in line with the WTW water requirement. It will thus allow for automatic augmentation of the supply to meet the requirement of the WTW, in the event that there is reduced/no supply from the long weir and the water level in the Coerney Dam is above the MOL. This negates the need for separate inlet and outlet pipelines, from the canal to the dam, and from the dam to the WTW supply pipeline, respectively. Doubling of the pipeline would have a considerable cost implication, which has been avoided.

3 Topographical Survey

A topographical survey was completed by Survey Services: Southern Operations (National Water Resource Infrastructure) of DWS for the Coerney Dam site (**Figure 3.1**) in May 2018. The results are reported in the relevant survey report of this study, namely *Coerney Dam Contour Survey*, May 2018 (Report No P WMA 15/N40/00/2517/1 and EC 003/2018 EC).

The contours of the existing 1 m contour plans from 1977 and 1984, which were compiled from aerial photography for the design of the LSRGWS, were digitised. Nine test sections were surveyed for the Lower Coerney site, to compare and verify the digitised data to the actual ground data, which resulted in a good match.

In August 2018, the survey was updated and expanded to include the immediate surrounding infrastructure. This is indicated on the map showing the detail survey of the area below Scheepersvlakte Dam, included in **Appendix B-1** and the total plan detail survey of the area, as shown in **Appendix B-2** and **Appendix B-3**.

The available survey data is considered sufficient to undertake the feasibility, preliminary and detailed designs of the proposed infrastructure. It is, however, recommended that a ground centreline survey be done along the finally chosen pipeline centreline, prior to construction commencing. This will serve as a final check on the pipelines' vertical alignment and soil cover depths.

The available survey data and mapping of the project area will be provided in electronic format as part of the deliverables for this feasibility study, and will thus be available to the design team for detailed design of the proposed project.

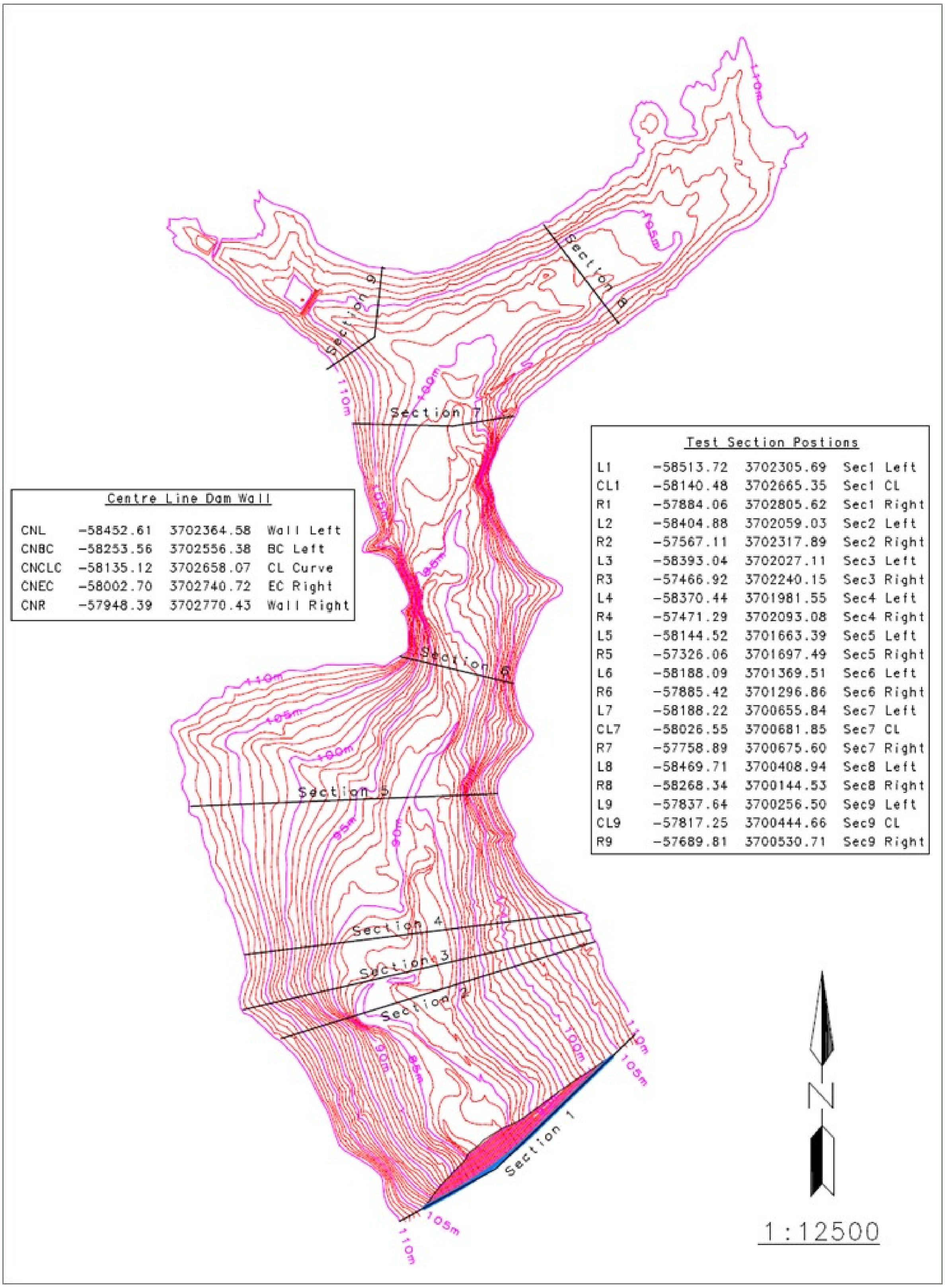

Figure 3.1: Coerney Dam contours

4 Geotechnical Investigation

4.1 Introduction

Geotechnical investigations for the options analysis comparison between the two most favourable dam sites, namely the Upper Scheepersvlakte Dam and Lower Coerney Dam sites were conducted in 2018. These investigations were conducted to inform the selection of the preferred dam site. The findings of this investigation, relevant to the Coerney Dam site, are reported in the *Lower Coerney Dam Geotechnical Survey* Report (Report no P WMA 07/N40/00/2619/2).

The geotechnical investigations included the following elements:

- Geophysical (resistivity) surveys;
- Test pitting including the additional test pitting for the supplementary investigation using a tractor-loader-backhoe (TLB);
- Rotary core drilling;
- In-situ field testing including standard penetration tests (SPTs);
- Packer water loss (Lugeon) testing;
- Sampling and laboratory testing; and
- Interpretation, analysis and reporting.

Following the selection of the preferred site (Lower Coerney) a more detailed test pit investigation was conducted at the site using a tracked excavator, with the aim of collecting supplementary and supportive data required for the feasibility design of the Coerney Dam. Findings of this investigation are reported in the relevant geotechnical report of this study, namely *Lower Coerney Dam Supplementary Geotechnical Survey* Report, September 2019 (Report no P WMA 07/N40/00/2619/3). This report is a stand-alone geotechnical report as it incorporates all data and findings from the first geotechnical report and is therefore the source of geotechnical information for the detailed design team.

4.2 Geology

The underlying geology comprises alluvium, colluvium, reworked terrace gravels (mixed origin), thin grey sandstones, siltstones and mudrocks of the Sundays River Formation of the Uitenhage Group; part of a collection of sedimentary strata within the structurally controlled Algoa Basin.

The seismic hazard of the area is considered to be very low and the Peak Ground Acceleration (PGA) values are less than 0.02g, with a 10% probability of being exceeded in a 50-year period.

The dam site is characterised by gentle, almost flat slopes; as is the greater basin. For the most part, the site is covered by very dense bush.

The geological profile is characterised by soil strata with thickness up to 7 m to 8 m on the left flank, but 3 m to 4 m on the right flank and river section. Various horizons are recognised, including topsoil, colluvium as well as colluvium with evidence of pedocrete development, and a horizon of gravel-sands, considered to represent reworked terrace gravels, that blankets the bedrock across the entire dam footprint, as well as within the basin. Bedrock comprises an alternating succession of sandstones and mudrocks, including silty sandstones. The lateral continuity of these strata is uncertain. The bedrock is characterised by extensive, pervasive weathering, and these rocks are generally considered weak rocks.

The transported soils essentially comprise mixtures of sand, clay and silt; either clayey silt, sandy silt or silty sand. The recent investigation indicates a clay content of 4% to 35%, with the highest content indicated on the mudrocks. A coarser fraction is present within the 'reworked terrace gravels' but is not uniformly distributed. In places a concentrated coarse fraction occurs that might represent former drainage channels, and in other areas the coarse fraction is a minor component. The permeability of the respective soil strata varies between 1.84×10^{-5} cm/s and 7.08×10^{-7} cm/s. The suite of dispersivity tests indicate that the soils are at least non-dispersive to intermediate dispersivity.

The geological profile, as well as other factors such as the topography, indicates that only an embankment dam is possible at this site. There are no suitable sources of rock in the immediate vicinity, and an earthfill embankment is the only viable option. A cut-off (under the embankment) would generally have to extend to the base of the gravel soils in order to ensure that the potential seepage path is effectively cut off.

The side channel spillway on the left flank would be underlain by soils and bedrock; full concrete lining of the chute will be required and provision for energy dissipation must be included at the downstream end. Bedrock was encountered between 3.4 m and 4.9 m in test pits TP126 and TP125 respectively, at the end of the spillway.

Packer tests within the bedrock yielded variable results, and included some significant losses ascribed to wash-out of weathered, soft rock interbeds.

4.3 Recommendations

In assessing various material types within the basin, no clear distinction can be made on the suitability of the various material types for either impervious core material or for semi-pervious shell material. In other words, the properties of the various material groupings do not permit clear definition of their suitability, and therefore clear delineation into different borrow areas for the respective material uses cannot sensibly be made. In view of this, and also considering the almost total compliance of these basin materials with typical homogeneous embankment specifications, it is recommended that the proposed Coerney Dam be constructed as a homogeneous earthfill embankment rather than a zoned embankment.

Involvement of a geotechnical specialist during construction is essential. Activities would include regular inspection of all excavated faces and cut slopes from a stability point of view, oversight of any further geotechnical exploration and quality assurance testing, confirmation of bedrock depth at the spillway end, engineering geological mapping of the cut-off trench, recording of the as-built details, etc.

The geotechnical investigation was limited to the dam sites, with no test pits undertaken along the pipeline route. The following is worth noting from the results for the dam site to inform the feasibility design of the pipeline:

- The test pits indicated that the likelihood of encountering shallow bedrock in the trench excavations is low.
- The laboratory tests indicated that the material excavated from site might not be suitable as pipe bedding material, due to the Plasticity Index (PI) not conforming to the SANS 1200 LB specifications. Some of the excavated material does however comply with the DWS 1110 Type A requirements. This means that part of the bedding material might need to be imported from commercial sources.
- Groundwater was encountered at depths of 8 m and deeper.

It is recommended that a geotechnical investigation be undertaken along the proposed pipeline routes and at the proposed chamber locations to inform the detailed design. The geotechnical investigation should also include a soil resistivity survey and testing for sulphate reducing bacteria, to inform the coating selection and the need for cathodic protection.

Table 4.1 presents the summarised findings of the ground investigations conducted at the Coerney Dam site.

Table 4.1: Summarised geological factors for Coerney Dam site

Geological factors	Description
General geology	Underlain by strata of the Sundays River Formation, Uitenhage Group, comprising thin grey sandstones, siltstones and mudrocks.
Geological profile; dam footprint	***Left flank;*** (upper), soils to 7.2 m (including horizon of gravelly soils 4 m to 7,2 m); very soft rock mudstone, subordinate sandstone from 7.2 m. ***Central section*** (conduit – intake and outlet) *Intake*; sandy soil to 2.65 m; gravelly soils to 7.7 m; soft to very soft rock (occasionally to clay) mudstone from 7.7 m; medium hard to hard rock interbedded mudstone / sandstone from 9.8 m. *Outlet*; sandy soil to 1.3 m; gravel-sand horizon to 4 m; very soft to soft rock sandstone from 4 m; soft to medium hard rock sandstone interbedded mudstone from 4.6 m; hard rock sandstone from 12 m. *Central section*; sandy soils to 2 m; gravelly horizon to 3.25 m; soft to very soft rock sandstone from 3.25 m; medium hard rock sandstone from 5.5 m; hard rock sandstone from 7.5 m; mudstones more prominent from 11 m. ***Right flank***; topsoil to 0.8 m; gravelly horizon to 2.7 m; highly weathered, medium hard to soft rock from 2.7 m. Interbedded sandstones, mudstones. The **upper right flank** comprises upper soils to 3.3 m and 4.2 m where bedrock is encountered
Founding considerations	A gravelly horizon (1.2 m to 5 m thick) is recognised which occurs across the footprint; considered to represent reworked terrace gravels. Note however the horizon is variable. Mostly the matrix was not recovered in the boreholes, but this stratum represents a potential preferred seepage path (a buried channel). Cut-off design is to consider this feature.
Excavation depths	For the **cut-off**, on the extreme / uppermost left flank, the principle of excavating to base of alluvial gravels implies a depth up to 7.2 m, maybe some relaxation allowed on extreme upper flank.; in central section assume minimum depth of 5.5 m but note some variability; on mid right flank consider minimum depth of 3.5 m (below gravel layer).
Foundation treatment	Mudrocks are susceptible to slaking; provision must be made for immediate protection after exposure. As above, remember presence of potential 'buried channel'; must ensure cut-off intersects this stratum. Permeability of rock mass is generally very low / tight, but instances of wash-out of softer strata are recorded. The 'groutability' of these weathered rocks is however uncertain.

Geological factors	Description
	At face value the outlet conduit could likely be founded on the gravel-sand stratum, but this does not consider required founding levels.
Spillway; geological profile	Upper spillway (near ogee / sill); soils to 4 m; gravelly soil horizon to 7.2 m; very soft / soft rock (mainly mudstone, subordinate sandstone) from 7.2 m. Lower spillway (actually midway); soils to 5.45 m; gravelly soils to 6.7 m; very soft rock sandstone (sand in places) from 6.7 m; interbedded sandstone / mudstone from 8 m. End of spillway: bedrock as slightly weathered hard rock sandstone is encountered between 3.4 m and 4.9 m..
Spillway considerations	Soils underlain by weak bedrock that would be susceptible to erosion. Assume full concrete lining is required. The appropriate energy dissipation must be incorporated at the end of the spillway lining, and measures must be incorporated to prevent undercutting of the concrete. The end of spillway should then be founded on the bedrock which should be encountered beyond 2.9 m depth, with all the upper horizons removed prior to placement of concrete
Reservoir slopes	Natural slopes are essentially flat / gently sloping; no slope stability issues foreseen.
Construction materials	No clear distinction can be made between the various material types within the basin in terms of their suitability for either impervious core material, or for semi-pervious shell material. Clear delineation into different borrow areas for the respective material uses therefore cannot sensibly be made. However, these materials do exhibit almost total compliance with specifications for use in a homogeneous earthfill embankment, and it is therefore recommended that the Coerney Dam be constructed as a homogeneous earthfill embankment rather than a zoned embankment. Other materials like coarse aggregate for concrete and filter sands / fine aggregate will have to be imported.

5 Coerney Dam Design

5.1 Salient features of the proposed dam design

The salient features of the proposed Coerney Dam, as determined during the feasibility design, are presented in **Table 5.1** below.

Table 5.1: Main details of the Coerney Dam

Parameter	Value
Classification	
Size	Medium
Hazard potential	High
Classification	Category 3
Dam Site	
Location (coordinates)	33° 26' 54" S 25° 37' 33" E
River	Tributary to Coerney River (in turn a tributary to the Sundays River)
Closest town	Kirkwood
Distance	18 km
Property description	Scheepersvlakte 98 Portion Number 7
Catchment and flood parameters	
Catchment area	33.6 km^2
Recommended Design Flood (RDF) magnitude	Incoming 143 m^3/s Outgoing 110 m^3/s
Water surface elevation at RDF discharge	99.3 masl
Safety Evaluation Flood (SEF) magnitude	Incoming 835 m^3/s Outgoing 753 m^3/s
Water surface elevation at SEF discharge	101.84 masl

Parameter	Value
Probable Maximum Flood (PMF)	835 m^3/s
Dam statistics	
Dam type	Homogeneous earthfill embankment with filter zones
Total crest length	600 m
Maximum height above river bed level	20.5 m
Embankment Non-overspill crest (NOC)	102.0 masl
Full supply level (FSL)	98.2 masl
Gross storage capacity at FSL	4.69 million m^3
Surface area of water at FSL	72 ha
Minimum Outlet Level (MOL)	86.0 masl
Base width of dam at maximum cross section	107 m
Crest width	5 m
Upstream slope	1V:3H
Downstream slope	1V:2H
River bed level at downstream toe	81.5 masl
Spillway	
Spillway type	Uncontrolled ogee overflow crest discharging into a side channel spillway on the left abutment
Ogee crest level	98.2 masl
Crest length	50 m
Freeboard	3.8 m
Energy dissipation	Stilling basin at the end of the discharge channel
Outlet details	
Tower	At the intake in the dam basin the outlet pipes are provided with a wet well tower with two intake levels, viz. 92.0 masl and 86.0 masl. The tower will be accessed via a pedestrian space frame bridge from the embankment NOC.
Inlet and Outlet pipes	The dam will have two pipes of 1000 mm dia each which serve as the inlet and outlet pipes. The pipes will be encased in reinforced concrete through the embankment. The pipes are situated on the left flank.

Parameter	Value
	At the downstream end, each of the pipes will have an arrangement to control the inlet and outlet flows. The inlet branch will have a shutoff valve. The outlet branch will have a shutoff valve as well as a non-return valve. Both pipes will connect to a wet well outlet tower in the dam basin.
Environmental Water Requirements (EWR) outlet description	No allowance for environmental releases is currently included in the design. DWS to determine whether further studies should be conducted to determine the EWR and arrangement required to provide this.

5.1.1 Storage requirement

The design water requirement and storage capacity are discussed in the *Options Analysis Report*. The salient points are reiterated here.

A balancing storage of 21 days average daily demand (ADD) is recommended to limit the risk of shortfall in supply to the NMBM. Thus, the design water requirement for NMBM of 76.6 million m^3/a, or 210 Mℓ/day, equates to a balancing storage of 4.41 million m^3.

Further, considering treatment losses of about 3% this equates to a storage requirement of 4.54 million m^3. A further storage volume of 150 000 m^3 should be included for the Scheepersvlakte Farms irrigator as replacement of their proposed new farm dam, which would have been located just downstream of the proposed Coerney Dam.

Sedimentation has also been discussed in the above-mentioned report. When considering the history of the Scheepersvlakte Dam basin; it has lost 51 000 m^3 storage to sedimentation, which represents a loss of capacity of about 2 320 m^3/annum. This equates to a sediment load of 15 m^3/km^2/annum, which amounts to a loss of 25 000 m^3 over a 50-year period in the Coerney Dam.

The inlet canal also contributes to the sediment load entering the dam, estimated at 0.002 % of inflows[1]. Canal sedimentation was estimated at approximately 32 500 m^3 resulting in a total storage loss due to sediment of 57 500 m^3 over 50 years.

[1] 22 250 m^3 sediment per 114 million m^3/annum canal water inflows

The dead storage provided below the minimum outlet level of 86.0 masl is more than sufficient to provide for this siltation, which means that the bottom outlet should remain unblocked for at least 50 years.

5.1.2 Storage capacity

A basin Storage vs Depth curve and Surface Area vs Depth curve were generated from the surveys at the proposed dam wall position. These are presented in **Figure 5.1** and shown in **Appendix C-1**.

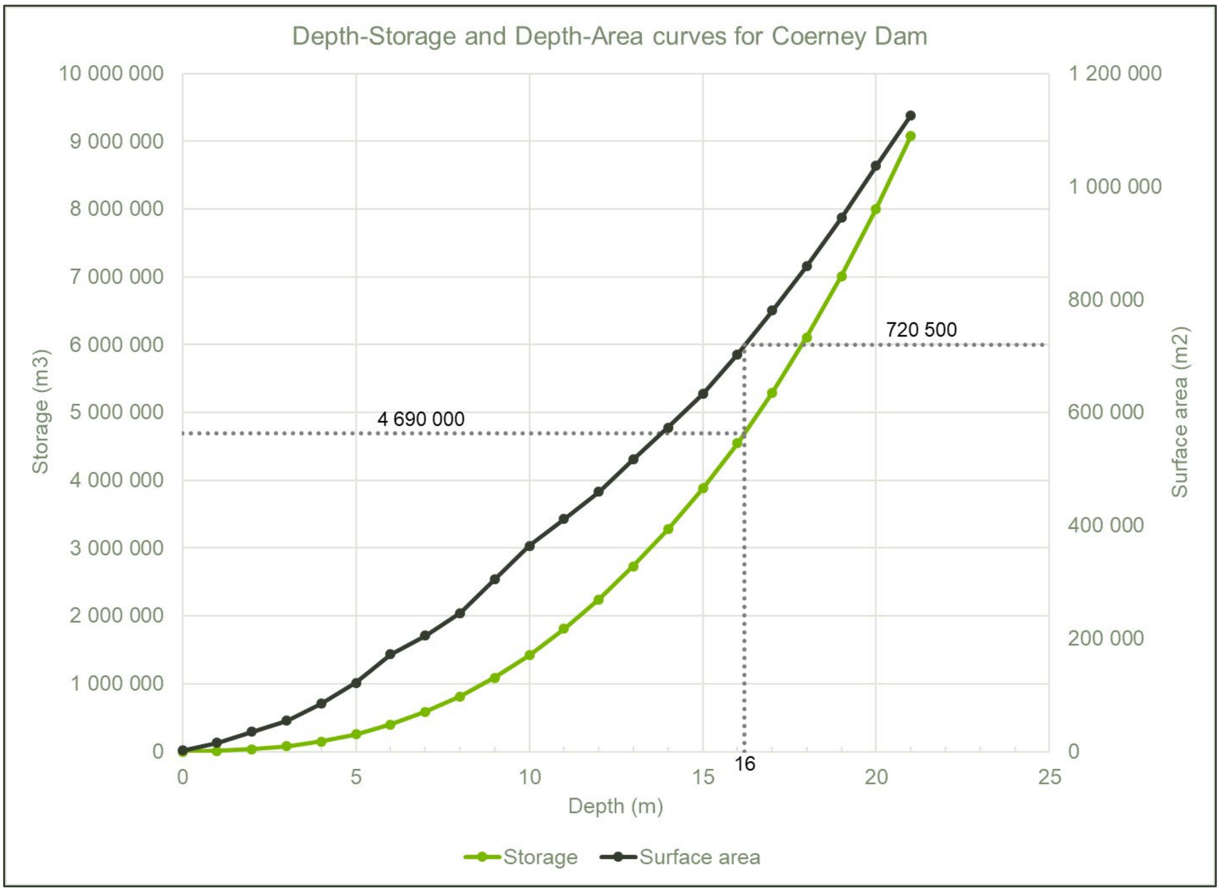

Figure 5.1: Depth-Storage and Depth-Area curves for Coerney Dam

The storage volume excludes the volume of material proposed to be excavated from the basin for use as the main fill material for the embankment.

Topographically there is potential for raising of the dam and there appears to be no developments above the full supply level other than the planned orchard of Scheepersvlakte Farms. A raising of the full supply level by 3 m for instance would increase the storage by approximately 2.3 million m^3. However, the currently proposed dam has not been designed with any raising in mind.

5.2 Dam safety classification

According to the Regulations Regarding the Safety of Dams as published under Government Notice R139 in Government Gazette 35062 of 24 February 2012 (in terms of Section 123(1) of the National Water Act, 1998) a dam with a wall height of more than 5 m and storage capacity of more than 50 000 m^3 must be registered as a dam with a safety risk.

Registered dams are classified into one of three classes (Category 1, 2 or 3) according to a combination of their *Size* and *Hazard Rating* as defined in **Table 5.2**, as reproduced from the regulations.

Table 5.2: Classification of dams with a safety risk

Size class	Hazard potential rating		
	Low	Significant	High
Small	Category I	Category II	Category II
Medium	Category II	Category II	Category III
Large	Category II	Category III	Category III

The first step of the classification considers the Size, or maximum wall height of the dam, according to the table in the regulations. The proposed dam has a wall height of 20.5 m and is thus in the *Medium* size class.

Secondly the dam's Hazard Rating is defined based on three factors in the case of a failure of the dam, namely potential loss of life, potential economic loss and potential adverse impact on resource quality. The Hazard Rating is considered in light of these three variables and is deemed to be *High*.

Consulting **Table 5.2**, the dam is classified as a Category 3 dam.

This classification is further used in the determination of the freeboard requirements, as well as for the recurrence intervals of the design floods.

5.3 Overview of the geology and construction materials

5.3.1 Introduction and background

Geotechnical investigations, as described in Chapter 4, were conducted in 2018. The following sub-sections summarise the relevant findings of the geotechnical investigation from the dam design perspective.

It should be noted that the investigations initially focused on placement of the spillway on the left abutment, with little targeted investigation on the right abutment. In light of the spillway design and the deep foundations found on the left abutment, the placement of the spillway on the right abutment is also considered, as discussed in Section 5.6. The investigations on the right abutment are thus limited to some test pits, and no core drilling was done there. Should the detail design re-consider the spillway on the right flank, additional geotechnical data, in the form of rotary core drilling, should be obtained to define the foundation conditions for the spillway and its discharge channel.

5.3.2 Regional geology

Generally, the underlying geology of the site comprises alluvium, colluvium, reworked terrace gravels (mixed origin), thin grey sandstones, siltstones and mudrocks of the Sundays River Formation of the Uitenhage Group.

Although there are several prominent faults recognised in the region, the seismic hazard of the area is considered to be very low and the PGA values are less than 0.02g, with a 10% probability of being exceeded in a 50-year period.

5.3.3 Dam wall foundation

The foundation of the embankment comprises two main components, namely the core cut-off trench and the shell zone. Considering the cut-off trench foundations, the subsurface geological profile along the centreline is characterised by soil strata with thicknesses ranging from 7 m to 8 m on the left flank, and 3 m to 4 m on the right flank and river section. Various horizons are recognised, including topsoil, colluvium, as well as colluvium with evidence of pedocrete development, and a horizon of gravel-sands.

These gravel-sands are considered to represent reworked terrace gravels and blanket the bedrock across the entire dam footprint as well as within the basin. This horizon (1.2 m to 5 m thick) represents a potential preferred seepage path (a buried channel). The design of the cut-off trench is to consider founding at the base of or below this layer so as to intercept this potential seepage path. Thus, for the cut-off, on the uppermost left flank, the principle of excavating to the base of the alluvial gravels implies a depth up to 7.2 m, with some potential for relaxation permissible on the extreme upper flank. In the central section a minimum depth of 5.5 m is assumed. On the mid right flank, a minimum depth of 3.5 m is considered.

This excavation profile may incorporate partial excavation into bedrock. The bedrock comprises an alternating succession of sandstones and mudrocks, including silty sandstones. It is characterised by extensive, pervasive weathering, and these rocks are generally considered weak rocks. The removal of this rock is assumed to be limited to the excavation of very soft, highly weathered sandstone and mudstone.

For founding of the embankment shell zones, it is assumed that foundation excavations will comprise removal of the topmost 0.3 m to 0.5 m, to remove the potentially organic-rich, and potentially compressible topsoil stratum. The latter value of stripping of 0.5 m is used further to determine embankment volumes, etc.

5.3.4 Foundation of spillway

The geology in the vicinity of the spillway and its discharge channel comprise soils underlain by weak bedrock that would be susceptible to erosion. At the left abutment spillway position these founding rock depths start from 7.2 m. On the right abutment the founding depths appear to be shallower, in the order of 3.0 m.

The upper horizons of the bedrock were shown to comprise completely weathered to highly weathered sandstone and mudstone, and is expected to offer very little long-term protection against erosion. An unlined spillway is thus not feasible. Appropriate protection and energy dissipation must thus be incorporated into the design. The spillway termination structure must also be suitable to prevent erosion and undercutting of the concrete as the spillway chute transitions to the channel downstream.

It should be noted that no targeted investigations were done along the right spillway discharge channel or the termination structure. The conditions are expected to be similar to those found on the left flank where suitable founding conditions were encountered at depths of 3.5 m to 5.0 m.

5.3.5 Foundation treatment

Water pressure (lugeon) testing of the foundation rock determined that the permeability of the rock mass is generally very low / tight, but instances of wash-out of softer strata were recorded during the testing. The ‘groutability’ of these weathered rocks is however uncertain. No allowance has therefore been made for grouting of the foundations.

Special mention should be made of the mudrocks, which are susceptible to slaking or rapid disintegration when exposed during excavations. Provision must therefore be made for immediate protection after exposure to prevent deterioration before construction/covering will commence.

5.3.6 Materials

5.3.6.1 Embankment fill materials

The following comments, extracted from the geotechnical report, summarise broad observations in respect of the suitability of the local materials for either impervious or semi-pervious classification (criteria based on those of Badenhorst, 1988);

- In terms of the material grading, the clay content largely complies with impervious materials with only a few scattered values falling either side of the target range between 10% and 30% clay content. This applies to all the material types encountered. The percentages passing the 0.425 mm sieve are routinely greater than 60%, and therefore show general compliance. Clay content is generally considered too high for semi-permeable materials.
- Considering the Atterberg limits i.e. Liquid Limits, Plasticity Index, and Linear Shrinkage, the results show scatter, reflecting some results falling outside the requirements. Specifically, the results are on the low side for impervious materials and on the high side for semi-pervious materials. Nonetheless, most samples meet the criteria for impervious materials and only a limited number fall outside that for semi-pervious materials.
- The standard Proctor compaction results show general compliance. The gravel horizon material does however record some anomalous values. The samples for impervious materials occasionally yielded dry density values that were too high, while the optimum moisture contents were too low. On the other hand, most of the materials generally fall within the acceptable range for semi-pervious material maximum dry density, i.e. between 1750 kg/m^3 and 2100 kg/m^3.
- The shear strength data shows that the materials all exhibit greater shear strengths than required, while the friction angles largely comply with the requirements (between 18° and 30°) for impervious materials and some values within the range for semi-pervious materials (28° and 38°).
- The measured permeabilities all show relatively impervious materials, well within the range required (less than 10^{-4} cm/sec) and below the value for semi-pervious materials

(greater than 10^{-4} cm/sec). Recorded values varied between 10^{-5} and 10^{-7} cm/sec, which relates to the clay contents for the various materials (typically varied between 10% and 25%), although some anomalous values were also recorded. The permeability of the respective soil strata varies between 1.84 x 10^{-5} cm/s and 7.08 x 10^{-7} cm/s.

- The suite of dispersivity tests indicates that the soils are at least non-dispersive to intermediate dispersivity.

Considering the above evaluation of the various material types available in the basin, it is evident that the materials show wide scatter in their properties and adherence to either impervious or semi-pervious classification. No clear distinction can therefore be made of the suitability between the various material types for their use in an impervious core zone or a semi-pervious shell zone. Clear delineation into different borrow areas for the respective material uses cannot sensibly be made.

On the other hand, if the properties of the various material types are evaluated in terms of the specifications for the homogeneous embankment constructed for Scheepersvlakte Dam (see **Table 5.3**) then the general compliance of the soils within the Coerney Dam basin is evident. Only limited values fall outside these specifications, specifically some Atterberg limits in the form of an occasional Liquid Limit, or some Plasticity Index values, which are less than 12% and therefore slightly on the low side.

Table 5.3: Homogeneous earthfill specifications for Scheepersvlakte Dam (DWA, 1988)

Grading analyses			
Sieve size	% passing		
	Maximum	Minimum	Mean
4.75	100	45.7	89.8
2.00	100	37.0	86.7
0.425	99.2	29.2	80.9
0.150	93.9	220	71.0
0.050	70.0	10.8	46.3
0.005	48.6	00	19.3
0.002	40.7	0.0	16.9
Atterberg limits			
	Maximum	Minimum	Mean
Liquid limit (%)	43.0	20.0	34.2
Plastic limit (%)	29.1	11.9	18.4
Plasticity Index	25.0	4.0	15.8
Linear shrinkage (%)	10.7	1.3	7.6
Compaction (Std. Proctor)			
	Maximum	Minimum	Mean
Maximum dry density (kg/m^3)	1884	1542	1736

Optimum moisture content (%)	24.2	10.8	16.3
Direct shear			
	Maximum	Minimum	Mean
Angle of internal friction (°)	45.0	19.4	35.4
Cohesion (kPa)	153.3	9.29	18.8
Triaxial shear			
	Maximum	Minimum	Mean
Angle of internal friction (°)	44.8	23.6	31.7
Cohesion (kPa)	40.0	0.0	15.5
Coefficient of permeability (cm/sec)			
	Maximum	Minimum	Mean
	4.1×10^{-5}	1.6×10^{-8}	1.1×10^{-6}
Relative density			
	Maximum	Minimum	Mean
	2.75	2.50	2.65

5.3.6.2 Sand filter and concrete aggregates

Other materials, such as coarse and fine aggregate for concrete and sands for filters were not found in the basin or near the site, and will have to be imported from commercial sources. A number of possible commercial sources for sand and coarse aggregates have been identified, but all are located some distance away from the Coerney site. The closest identified possible commercial sources are located in the Uitenhage and Coega areas, which is more than 60 km away from site. The identified sources, which need to be confirmed during detailed design, are as follows:

- Potgieter Quarries, a sand quarry located in the Paterson area is an option. However, attempts to contact the quarry to identify the quantities and type of materials they produce did not yield any results at the time of study.
- Harbron Quarries is located in the Uitenhage area, approximately 50 km from site. This quarry manufactures all types of sand and stone products.
- Denver Afrimat Aggregates quarry is located about 70 km from the Coerney site, also in the Uitenhage area; and produces both sand and coarse aggregates.
- Glendore Sand and Stone produces sand and coarse aggregates from the Sonop sand quarry and Coega Kop quarry respectively. Sonop quarry is located about 75 km from site and Coega Kop Quarry is about 65 km from site.

5.3.7 Cut slopes

The gravel–sand stratum of reworked terrace gravels is a concern in terms of the stability of cut slopes. Where the cut slopes intersect this horizon, there is a likelihood that ravelling and spalling will occur within these gravel soils. This can result in undercutting of the overlying strata, and an associated risk of slope failure. The stability of these horizons will be further compromised when wet. Excavation within these gravels also carries the risk that removal of the coarser fraction can result in further disturbance of the stratum, and due care is called for in these instances.

Generally, the design excavations consider slopes of 1V:1H, for the founding of the outlet tower and outlet pipe encasement, spillway channel excavations and embankment core trench excavations.

5.4 Embankment

5.4.1 Dam type selection

The flat topography, limited materials availability and absence of rock foundations at the site dictate that the only dam type considered suitable is an embankment dam. Embankment dam sub-options were considered, namely rock- and earthfill embankments as well as the possibility of zoning of the embankment materials. Rockfill embankments were not considered viable due to the lack of rock of suitable quality available on site.

During the options assessment stage a zoned embankment was considered, which contained an impermeable central core zone. However, further geotechnical investigations (see Section 4), notably the test pitting and soil testing in June 2019, has shown that there is insufficient differentiation between the various materials (e.g. impervious clay core vs semi-pervious general fill) throughout the dam basin to make the construction of a zoned embankment practical. Therefore, a homogeneous earthfill embankment has been selected as the most suitable dam type for the site.

As can be seen in Drawing No. 112546-0000-DRG-CC-003-A in **Appendix C-2**, the homogenous earthfill embankment still displays some zoning other than the homogenous fill zone. These zones include upstream rip-rap protection, sand chimney and finger drains, gravel filter and rock toe drains.

It is also pertinent to note lessons from construction of nearby Scheepersvlakte Dam, notably in terms of the required moisture content (DWAF, 1992) for the further design of the embankment. As a result of the relatively high moisture requirements (for the homogeneous fill), coupled with the high clay content, construction difficulties were experienced. The high optimum moisture contents also resulted in compaction problems.

5.4.2 Embankment layout

The proposed embankment alignment is largely straight across the valley, but has a slight curve to allow the dam to intercept the valley contour lines perpendicularly and so limit embankment quantities. Refer to Drawing No. 112546-0000-DRG-CC-001 in **Appendix A-1** for the layout of the dam.

The embankment cross-section has typical slopes of 1V:3H on its upstream side and 1V:2H on its downstream side. The crest is approximately 600 m long and 5 m wide with a 2% cross-fall toward the upstream side for surface drainage.

A cross section of the embankment is illustrated in **Figure 5.2** showing the zones and various elements, which are discussed below. This is shown in more detail in Drawing No. 112546-0000-DRG-CC-003-A in **Appendix C-2**.

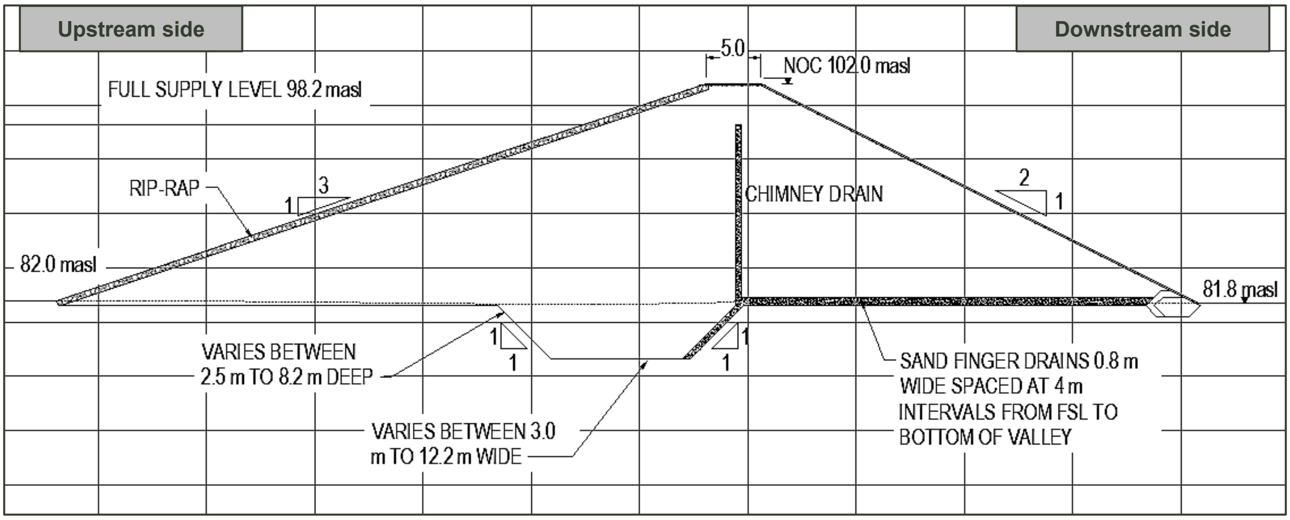

Figure 5.2: Illustrative cross section through the proposed embankment

The lowest level at the valley bottom is 81.5 masl, which with a required NOC level of 102.0 masl, results in a maximum wall height of 20.5 m.

The upstream face is protected by a rip-rap layer 600 mm thick (perpendicular thickness). The downstream face is protected by a 200 mm thick layer of crushed stone.

The internal zoning consists of a chimney drain, 0.5 m thick, which extends from the FSL down to the embankment foundation. It is connected to a number of finger drains 0.8 m x 0.8 m wide spaced at 4 m centre to centre. Finger drains are proposed rather than a blanket drain to reduce the volume of imported sand material required for its construction. The finger drains connect to a gravel and rock toe 3 m wide and 2 m thick, half under-ground.

The core trench depth varies, as discussed in Section 5.3.3, from approximately 8 m on the left abutment to 5 m in the river section and 3 m to 4 m on the right abutment. The core trench bottom width is set to half of the height from the embankment crest to the depth of foundation at that particular position along the embankment crest. Using this method, the core trench bottom width varies from 5 m to 7 m on the left abutment, up to a maximum of 12.2 m in the river section where the embankment is highest, and approximately 6 m to 10 m on the right abutment.

The sides of the core trench will be sloped at 1V:1H in accordance with the slope stability concerns noted in Section 5.3.7, but also to limit the effect of arching of placed fill, which could occur if the slopes were steeper.

5.5 Flood hydrology

5.5.1 Flood hydrology

The investigation into the flood hydrology for both the Upper Scheepersvlakte and (Lower) Coerney sites was performed for the Options Analysis and are detailed in the *Options Analysis Report* (P WMA 15/N40/00/2517/3). A copy of the methodology and calculations is provided in **Appendix C-3** (Design Flood Analysis Report), **Appendix C-4** (Rational Method calculations) and **Appendix C-5** (Soil Conservation Service (SCS) Method calculations).

Based on the size of the study catchments (**Figure 5.3**) and the lack of streamflow records in the study catchments, it was decided to follow only a deterministic approach for the estimation of the design floods. Two deterministic methods were employed for design flood determination; the SCS and Rational Method-approaches.

The catchment characteristics used are given in **Table 5.4**.

Table 5.4: Catchment parameters

Characteristic	Value
Area	33.6 km^2
Length of longest watercourse	9.83 km
Slope of longest watercourse (Equal-Area)	0.0148 m/m
Average catchment slope	6.55 %

The design rainfall used in design flood peak determination must be the 24-hour rainfall for a given recurrence interval. However, the most generally available rainfall data in South Africa and many other countries represent daily measurements by human observers according to a fixed daily cycle of, say, 8 am to 8 am (which is a wholly artificial time resolution). Intense rainstorms might however have durations that straddle the artificial 8 am cut-off for a "daily" measurement. Consequently, the "daily" values in the rainfall record, representing such "straddling" storms on either side of the artificial 8 am cut-offs, cannot reflect the maximum 24-hour values. The 24-hour values are extracted (through a moving 24-hour "window") from records of continuous rainfall measurements by automatic data-loggers. Such installations are quite rare relative to the above 8 am to 8 am type of "daily" rainfall station. In South Africa the ratio of the number of automatically logging stations to "daily" rainfall stations is less than 1:10.

Various studies have shown that the ratio of annual maximum 24-hour rainfall to "daily rainfall" is about 1.10 to 1.15, regardless of location. In South Africa the ratio of 1.11 is most commonly used.

The one-day and 24-hour rainfall values for various recurrence intervals are given in **Table 5.5**.

Table 5.5: 1-Day and 24-Hour point design rainfalls

Recurrence Interval (y)	Point Design Rainfall (mm)	24-Hour Point Design Rainfall (mm)
1:2	51	56.6
1:5	76	84.3
1:10	95	105.5
1:20	116	128.8
1:50	146	162.1
1:100	172	190.9
1:200	202	224.2
PMP		666

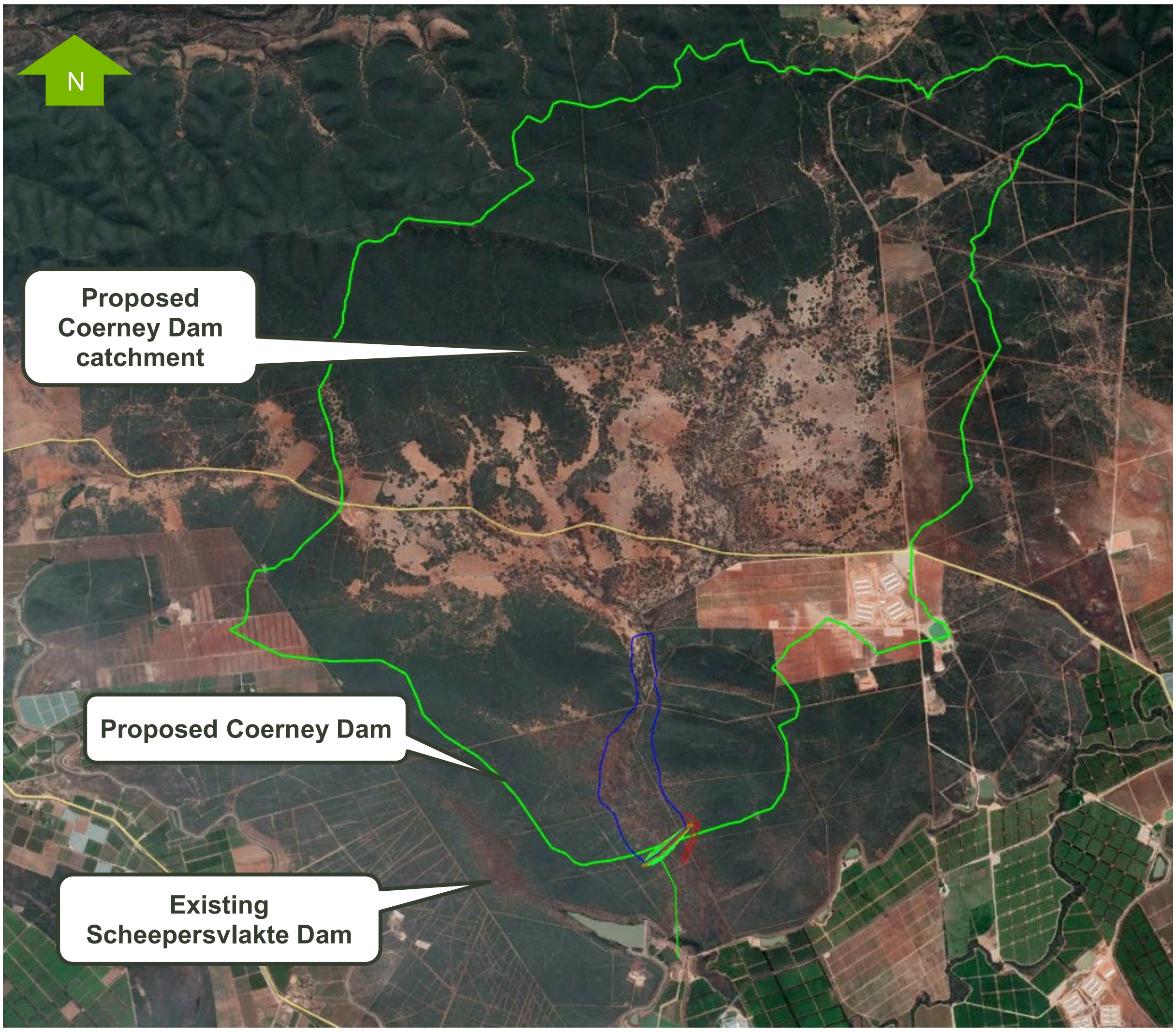

Figure 5.3: Coerney Dam catchment

The resulting flood peaks for the range of recurrence intervals for the two calculation methods employed, as well as the recommended values, are presented in **Table 5.6**.

Table 5.6: Flood Peaks (inflow discharge) at Coerney Dam

Recurrence Interval (year)	Rational method (m^3/s)	SCS method (m^3/s)	Recommended value (m^3/s)
1:2	16	10	**13**
1:5	26	27	**27**
1:10	35	43	**39**
1:20	48	63	**56**
1:50	74	95	**85**
1:100	105	125	**115**
1:200 (RDF)	124	161	**143**
PMF (SEF)	869	801	**835**

5.5.2 Design floods

The expected classification for the dam is Category 3, based on the height, storage capacity and expected hazard potential downstream of the dam, as discussed in Section 5.2.

The SANCOLD Guidelines in Relation to Floods (SANCOLD, 1991) recommend for this Category 3 dam that:

- The Recommended Design Flood (RDF) equals the 1:200 year recurrence interval (0.5% exceedance probability) flow peak of 143 m^3/s.
- The Safety Evaluation Flood (SEF) equals the Probable Maximum Flood (PMF) of 835 m^3/s.

It is noted that these flood hydrology estimates are compiled on a preliminary basis and it is recommended that they are reviewed and explored in greater detail prior to detail design. It is known that extreme flood estimates in this region around Port Elizabeth are notoriously difficult to predict and specialist input is needed to reflect on their applicability to dam design. It is therefore recommended that a site-specific SEF be determined for detailed design of the dam.

5.6 Spillway

5.6.1 Spillway location options

The proposed spillway type, which is suitable for this site, can be positioned on either the left or the right abutment of the dam. Each site holds its own advantages and disadvantages, and cost implications. The same spillway overflow configuration was considered for both options. The main differences are the discharge channel length as well as depth to suitable foundations for the overflow structure. The two spillway location options are discussed below.

The two spillway layouts are shown in **Figure 5.4**. Further details can be found in **Appendix A-1** Drawing No. 112546-0000-DRG-CC-001.

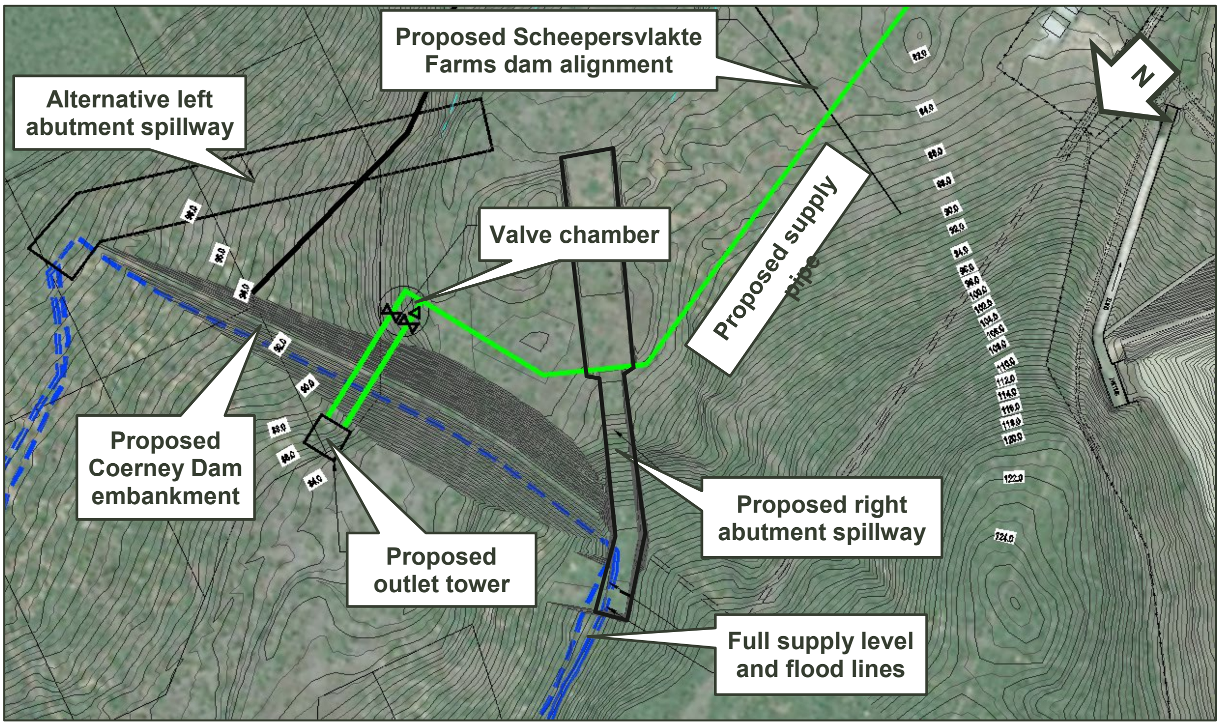

Figure 5.4: Plan view of the embankment showing two spillway location options

5.6.1.1 Right abutment option

During the geotechnical investigations in 2018 the rotary core drilled holes indicated that rock foundations for the mass concrete overflow on the left flank would be deep. During the follow up geotechnical investigations in June 2019 some test pits were excavated with a tracked excavator. These showed that the rock foundations on the right abutment appear to be shallower than on the left abutment. However, detailed targeted investigations or core drilling were not done at the right abutment site.

The shallower foundations, which seem to be apparent at the right abutment site, can have a cost saving implication due to both the volume of excavation required to expose the foundations, as well as the volume of concrete which will be required for the construction of the mass gravity overflow weir structure and for lining of the discharge channel.

Topographically, the right abutment is steeper than the left abutment, meaning the discharge channel may be shorter than the left abutment spillway discharge channel. The right abutment therefore results in significantly reduced excavation volumes and consequently less construction materials.

However, there exist some drawbacks to siting the spillway on the right abutment. The low point in the river is situated on the left side of the valley, which means that there is a relatively long and flat portion from the right abutment to return the flow to the valley low point on the left. A possible solution to this is to guide the flow using an approximately 1 m deep 40 m wide, Armorflex or gabion lined, trapezoidal channel. This adds an additional quantity of excavation, as well as additional lining material to be imported (rock or Armorflex) and has a larger surface area for clearing.

An additional drawback of siting the spillway on the right abutment (western side of the valley) is that the proposed supply pipeline and access road come from the south-west and would thus need to cross over (or under) the spillway discharge channel en-route to the dam wall. Alternatively, the infrastructure may be relocated to approach the dam from the south after crossing the stream to the left bank.

Lastly, the steeper discharge channel results in higher flow velocities which, when compared to the left flank option, requires a larger stilling basin to dissipate the energy before releasing flow into the return channel.

5.6.1.2 Left abutment option

The original position earmarked for the spillway was on the left abutment. The main advantage of the left abutment spillway option is the elimination of the need for the spillway discharge channel to cross both the access road to the dam crest as well as crossing the supply pipeline to the dam. This simplifies construction, and operation and maintenance of the infrastructure in these areas.

The main disadvantage of the left abutment spillway option is the significant excavation volumes (and by implication other material quantities such as concrete) to the rock foundations for the inlet channel, and discharge channel. Further, the discharge channel is also longer than the right abutment option, but it does not need the extensive river return channel and related erosion protection measures.

The slope on the left abutment is flatter than the right resulting in slower and deeper flow depths than the right abutment spillway channel. However, the depths to foundations are far in excess of the required flow depth and this is thus not a significant factor to consider in siting the spillway.

5.6.1.3 Recommended spillway location

There are advantages and disadvantages of the two spillway sites identified. The spillway on the left has deeper foundations and a longer discharge channel, which needs to be lined. The spillway discharge channel on the right is shorter and shallower than the one on the left, with less material quantities as a result.

The left abutment option is approximately R5.42 million (15%) more expensive than the right abutment option, which has a comparative cost of R36 million. This cost comparison hinges on the geotechnical conditions on the right abutment, the rates for the construction items, as well as the percentage of material from the spillway excavations that can be used in the embankment construction. To confirm the cost comparison, the geotechnical conditions would need to be further investigated on the right abutment, as well as the lining method and material costs for the river return channel.

In conclusion, however, the spillway on the right has a distinct drawback of requiring a crossing of both the supply pipeline as well as the access road. This complicates the design and construction of the pipeline, as well as increasing its safety risk. The operation and maintenance of this portion of the pipeline will also be more difficult due to limited access. It is particularly due to this last reason that the left abutment option is favoured by the LSRWUA and the NMBM, who would be the operator and beneficiary of the scheme respectively.

The comparative cost difference is thus considered small enough to justify the preference of the left abutment spillway option, which is recommended for feasibility design and implementation.

5.6.2 Spillway type and arrangement

Considering the general lack of bedrock at or near natural ground level and the gently sloping topography, a spillway, which would take advantage of the deep excavations to foundation rock, should be the most optimal for the site. Two spillway arrangements were compared, namely a straight in-line spillway and a side channel spillway.

A side channel spillway configuration requires little lateral space, as the long spillway overflow can be placed parallel to the contours/river course rather than perpendicular to them, which would require a widening of the already deep excavations. The side channel arrangement also means the discharge channel can be narrower than the overflow length of the weir, resulting in reduced discharge channel excavations and concrete volume for lining.

The side channel dimensions (depth and bottom width) required to convey the site-specific design floods were found to be considerable. However, it was still found to be less than an in-line structure, which requires significantly wider excavation into the abutment and for the discharge channel. It is possible to taper the width of the discharge channel along its length, but the reduction was not found to be significant enough to make it more attractive than the side channel arrangement. It is noted that there is potential for narrowing the discharge channel from the side channel, but this is not considered further in the current level of study.

The side channel spillway arrangement thus appears to make better use of the site conditions and foundation excavation requirements without increasing them beyond what is already necessary, and it was selected as the optimal spillway arrangement.

Non-linear spillways were not considered during this study. It is noted that such innovative and novel spillways could be investigated further during the detail design process (duckbill, labyrinth, piano-key weir etc.) as these may allow for a reduction in the excavation quantities. However, these types of spillways generally need good founding conditions, which may or may not be present at the Coerney Dam site.

5.6.3 Overflow structure

The spillway crest will consist of a 50 m long mass gravity, ogee-shaped, overflow weir. Flow discharges into a 20 m wide, and 6.35 m deep, side channel which directs flow to the head of the discharge channel. It then flows down the abutment slope terminating in the downstream stilling basin, from where it returns to the river channel.

A cross-section through the spillway overflow structure and side channel is shown below in **Figure 5.5**. Further details can be found on Drawing No. 112546-0000-DRG-CC-004 in **Appendix C-6**.

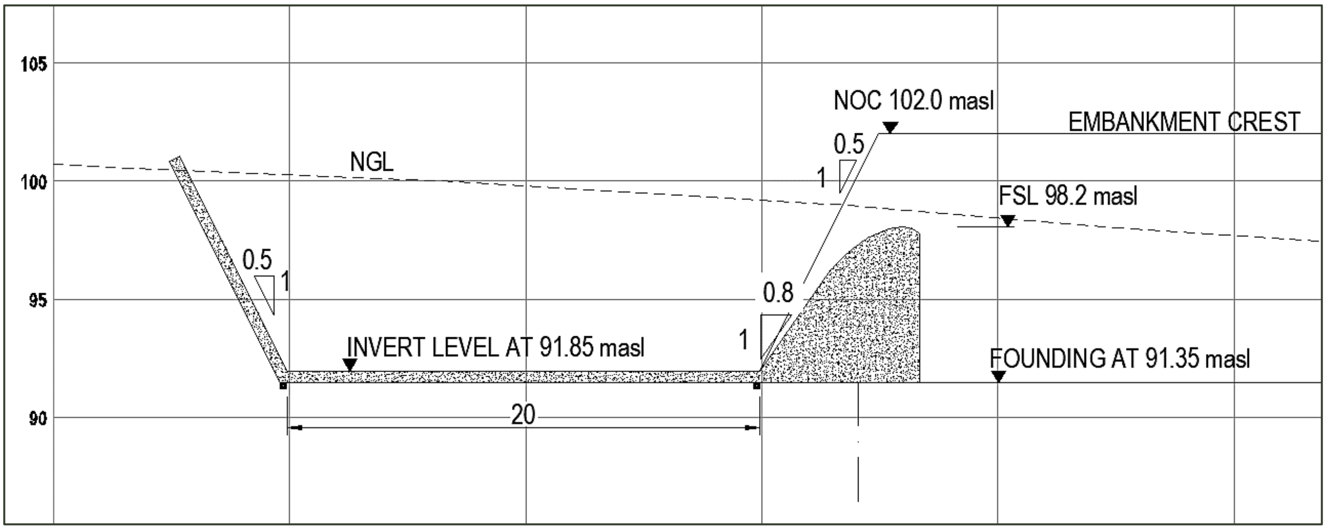

Figure 5.5: Cross-section through spillway overflow structure and side channel

The ogee crest of the spillway was designed in accordance with the standard USBR spillway shapes (USBR, 1987). The flood hydrology has shown that there is a large difference between the design flood (the RDF) and the maximum flood (the SEF). This means the RDF should not be used at the design head as negative pressures could develop at the maximum head (during the SEF) potentially leading to cavitation. The design head for the ogee shape is thus chosen above the RDF, to reduce the cavitation potential, but below the SEF (maximum head) for a more efficient spillway design. The ratio of the design head (H_d) to the maximum head at SEF (H_{max}) ratio should be kept above 0.75. The chosen design head (Hd) is 2.9 m whereas the RDF level is only 1.1 m (see **Table 5.7** in Section 5.6.7).

Contraction losses at the abutments are reduced by designing for rounded abutments, which tend to result in smooth flow lines. There are no piers or flow splitters in the proposed design, which would have further contraction implications on the spillway overflow length and associated discharge.

The weir overflow structure is designed with a founding level of 91.35 masl (expected bedrock level). The weir will thus have a height of 6.85 m. To eliminate the impact on the spillway discharge capacity the upstream pool depth should be two to three times the design head (USBR, 1987). It is thus proposed that the founding excavation level of the weir be extended for the full inlet channel to negate the limiting effects of a shallower weir upstream pool depth.

The spillway discharge calculations are presented in **Appendix C-1** and the discharge rating curve is shown in **Figure 5.6**. The discharge head and maximum stages for the routed and un-routed RDF and SEF are given in **Table 5.7** in Section 5.6.7.

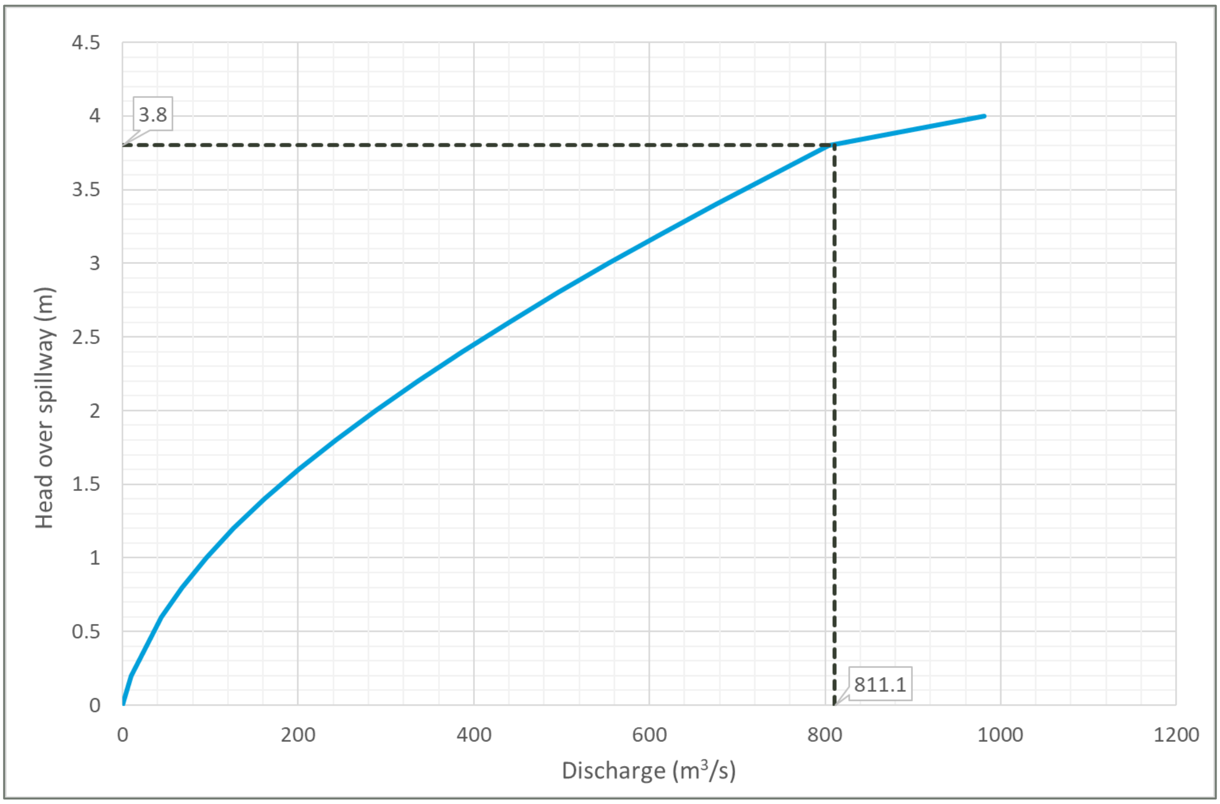

Figure 5.6: Spillway rating curve (showing discharge at NOC)

5.6.4 Side channel

The ogee overflow structure discharges into a side channel where the flow changes direction to be transported down the abutment. The side channel should thus be designed to have subcritical flow, which results in smooth flow, and reduces cross waves and turbulence (USBR, 1987). Subcritical flow can be induced in the side channel by introducing a weir or contraction at the end of the side channel before transitioning to the discharge channel. No specific measure was designed at this stage. Numerical and/or physical modelling would be required to determine the efficacy of any such measure.

Further to this, the channel should be deep enough so as not to drown out the ogee and reduce its discharge via submergence effects. This depth requirement is achieved by keeping the water level in the channel below two thirds of the head over the ogee during the extreme flood, the SEF (USBR, 1987). A backwater calculation was performed to determine the required depth of the side channel with the control section at the transition to the discharge channel.

The resulting channel dimensions are as follows: side slopes of 1V:1H, bottom width of 20 m, invert level of 92.1 masl and longitudinal slope of 0.5%.

It should be noted that the spillway dimensions, especially that of the side channel still need to be optimised to balance hydraulic, cost and constructability efficiencies.

5.6.5 Discharge channel

The proposed discharge channel is a trapezoidal channel with a base width of 20 m and side slopes of 1V:1H. The cross-section of the channel is shown in **Figure 5.7**. Further details can be found in **Appendix C-6** Drawing No. 112546-0000-DRG-CC-004.

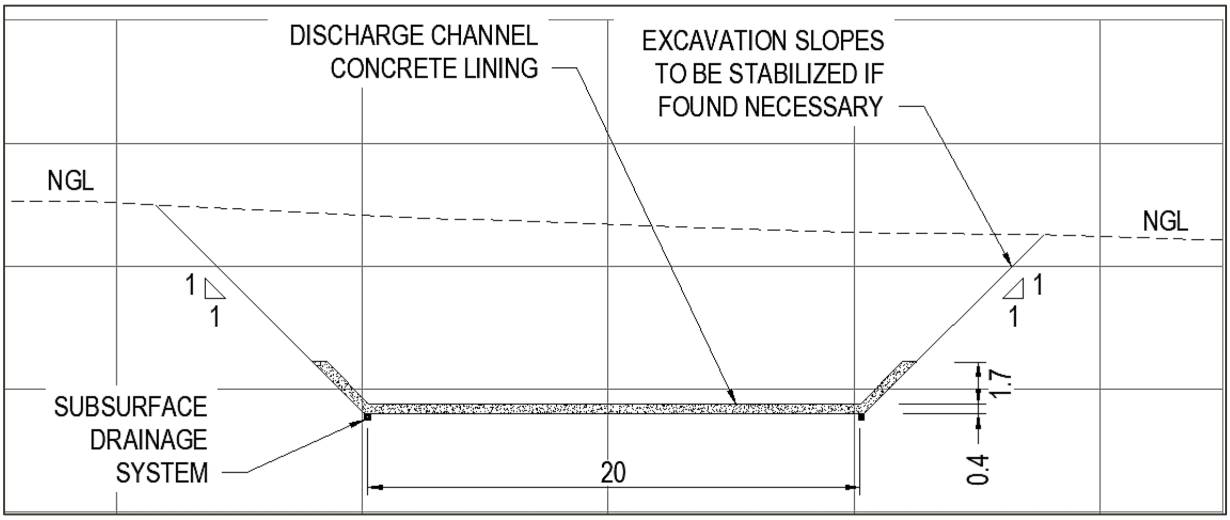

Figure 5.7: Cross-section through the spillway discharge channel

Based on the required depths of the side channel, the depth to foundation of the discharge channel and the energy dissipation structure at the end of the spillway, a longitudinal slope of 0.063 m/m is obtained. Using Manning's equation and assuming uniform flow, the attenuated out-going RDF flows at a depth of 0.51 m and the attenuated out-going SEF at a depth of 1.63 m in this discharge channel.

Due to the soft erodible nature of the underlying soil horizons, the channel should be lined with reinforced concrete. The proposed design allows for a lining depth of 1.7 m, i.e. up to the SEF flow depth plus freeboard. The lining is assumed to be 0.4 m thick. Furthermore, the soft foundations require special care to be taken during detail design of the floor joints of the spillway channel, as well as the appurtenant drainage features. The provision of aeration to limit the onset of cavitation should also be investigated.

The channel founding depth is an estimated 7.5 m, on average, and thus the portion above the spillway lining should be cut back at a flat slope, such as 1V:1H or even flatter, depending on the site conditions encountered. A bench should be provided in the channel excavation at the top level of the concrete lining to facilitate access for construction and future inspections.

The discharge channel could be further optimized by narrowing it from the transition from the side channel and also tapering the bottom width along its length.

5.6.6 Termination / energy dissipation structure

At riverbed level of the left abutment slope the steep discharge channel will terminate in a stilling basin. The end of the stilling basin will step up to discharge back into the low point of the valley, where the river would flow during flood conditions. The configuration and dimensions of the stilling basin needs to be determined by a physical hydraulic model during detailed design.

5.6.7 Flood routing

The outcomes from the flood determination, embankment design and spillway design were used in a level pool flood routing exercise. The hydrographs from the SCS flood determination method were used for the flood routing. The in-coming flood peaks were attenuated by between 23% and 10% for the RDF and SEF respectively. The results are summarised in **Table 5.7** with more details provided in **Appendix C-1**.

Table 5.7: Results of flood routing through the dam

Flood	Recommended Design Flood (RDF)	Safety Evaluation Flood (SEF)
Recurrence interval [Annual exceedance probability]	1:200 year [0.5%]	N/A
Flow peak, In-coming	143 m^3/s	835 m^3/s
Flow peak, Out-going	110 m^3/s	753 m^3/s
Attenuation	±23%	±10%
Maximum water level	99.3 masl	101.84 masl
Height above over FSL, 98.2 masl	1.1 m	3.64 m
Height above NOC, 102.0 masl	-2.7 m	-0.16 m

5.7 Freeboard

Freeboard for the embankment (height between FSL and NOC) is calculated for combinations involving the RDF and SEF flood surcharge levels, as the calculation of freeboard includes a number of other load cases, such as waves and earthquakes. The freeboard combinations for the proposed Coerney Dam were calculated using the current SANCOLD (2011) guidelines for a category III embankment dam. The freeboard calculations are based on the spillway configuration, basin characteristics and routed floods as described in previous chapters. The results are summarised in **Table 5.8**, with detailed calculations and input parameters in **Appendix C-1**.

Table 5.8: Freeboard combinations

Aspect	Value (m)
Full supply elevation	98.20
Non-overspill crest elevation	102.00
RDF elevation (attenuated outflow)	**99.30**
RDF water level above FSL	1.10
SEF elevation (attenuated outflow)	**101.84**
SEF water level above FSL	3.64
Wave height, $H_{2\%}$ (100 yr)	1.89
Design wave run-up, $R_{2\%}$	2.26
Wind setup	0.04
Surges and seiches	0.00

Aspect	Value (m)
Freeboard combinations:	
1. RDF + wave run-up	3.36
2. RDF + wave run-up + set-up & surges	3.41
3. Earthquake	0.08
4. RDF + landslides	1.10
5. RDF + wave run-up + set-up + surges & gates	3.41
6. SEF	3.64
Minimum freeboard required as per guidelines	2.60
Freeboard required	**3.64**
Freeboard provided	**3.80**

5.8 Outlet works

5.8.1 Inlet/Outlet works configuration

The proposed Coerney Dam will be connected to the existing water supply scheme via a 1 400 mm diameter steel pipeline. This proposed pipeline will convey water from the new offtake, located on the Kirkwood primary canal, to the dam. A branch line will connect this new pipeline to the existing 1 400 mm diameter steel pipeline to the Nooitgedagt WTW, downstream of the high point in the existing line.

The offtake from the Kirkwood primary canal will be located downstream of the Coerney syphon intake, and just upstream of the long weir, which will provide head to the new intake. The proposed new intake is a gated weir structure to control the inflow. It is described in further detail in the report *Feasibility level engineering design – Conveyance infrastructure*.

The outlet works of the dam are located on the left abutment (eastern bank) of the valley in which the proposed dam is located. The pipe for supplying water to and from the dam will reduce from 1400 mm diameter to 1200 mm, and then 1000 mm diameter after it bifurcates into a separate inlet and outlet branch at the outlet chamber, which is located at the downstream toe of the embankment. It is proposed that the encased pipework through the embankment be made of stainless steel.

The layout of the existing and new pipelines is shown in **Figure 5.8**. Further details can be found in **Appendix A-1** Drawing No. 112546-0000-DRG-CC-001.

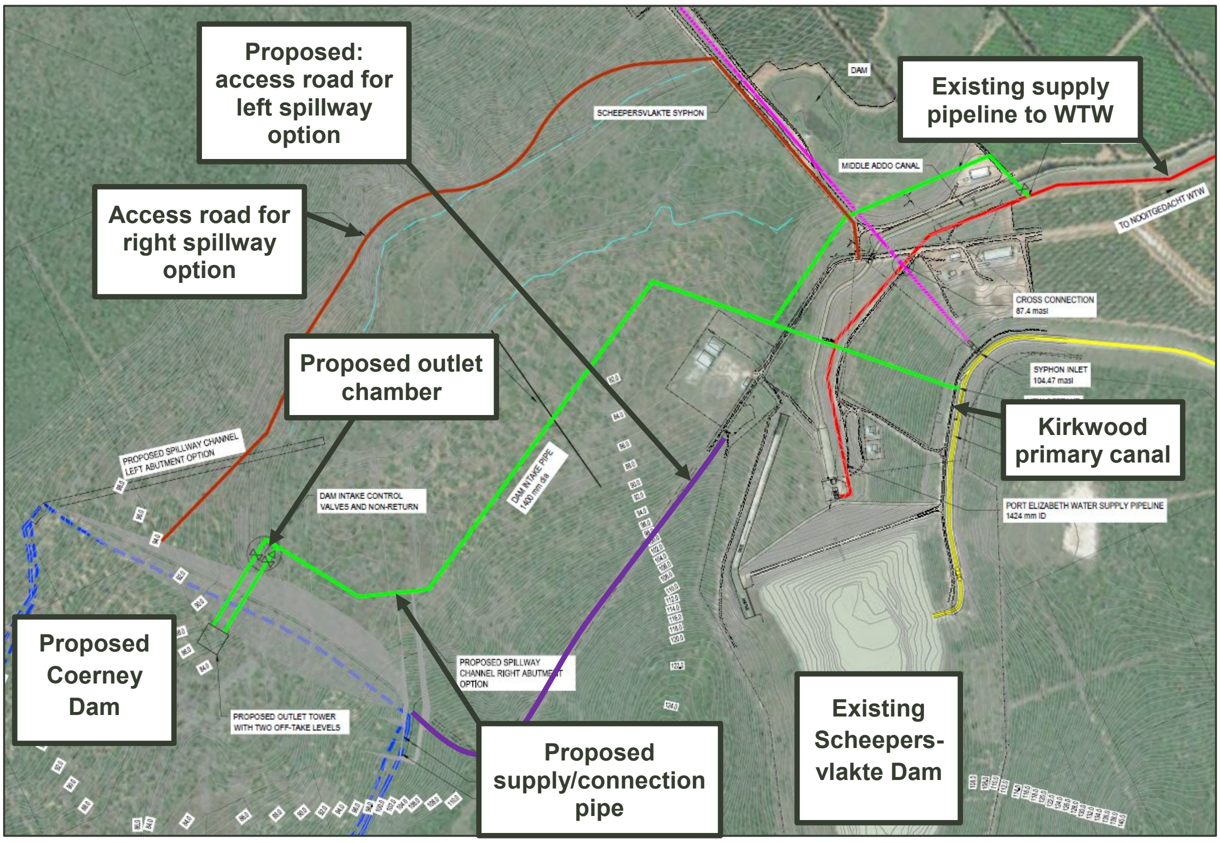

Figure 5.8: Dam and supply/connection pipe layout

The inlet branch, after bifurcation of the pipeline to the dam, will have an isolation valve for shutting off supply when the dam is full, to prevent spilling canal water. The outlet branch will be fitted with an isolation valve and, just downstream of this, a non-return valve. The non-return valve will ensure that water can be 'automatically' supplied from the dam in the event where the inlet has been shut to avoid spilling of the dam when it is full.

The two 1000 mm diameter inlet/outlet pipes will be encased in reinforced concrete through the embankment. This arrangement of the inlet and outlet pipes is shown in **Figure 5.9** and **Figure 5.10**.

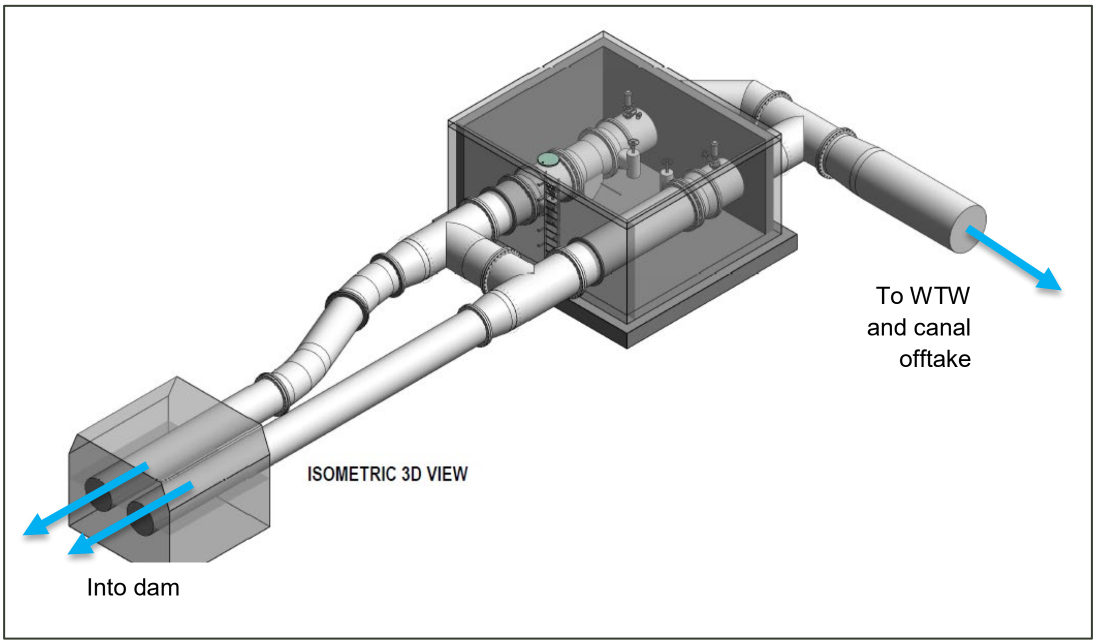

Figure 5.9: Isometric view of the dam's downstream outlet pipe chamber arrangement

The concrete pipe encasement will have battered slopes to improve the compaction and contact between backfill and the encasement, and mitigate the risk of preferential seepage and piping along the outlet pipe.

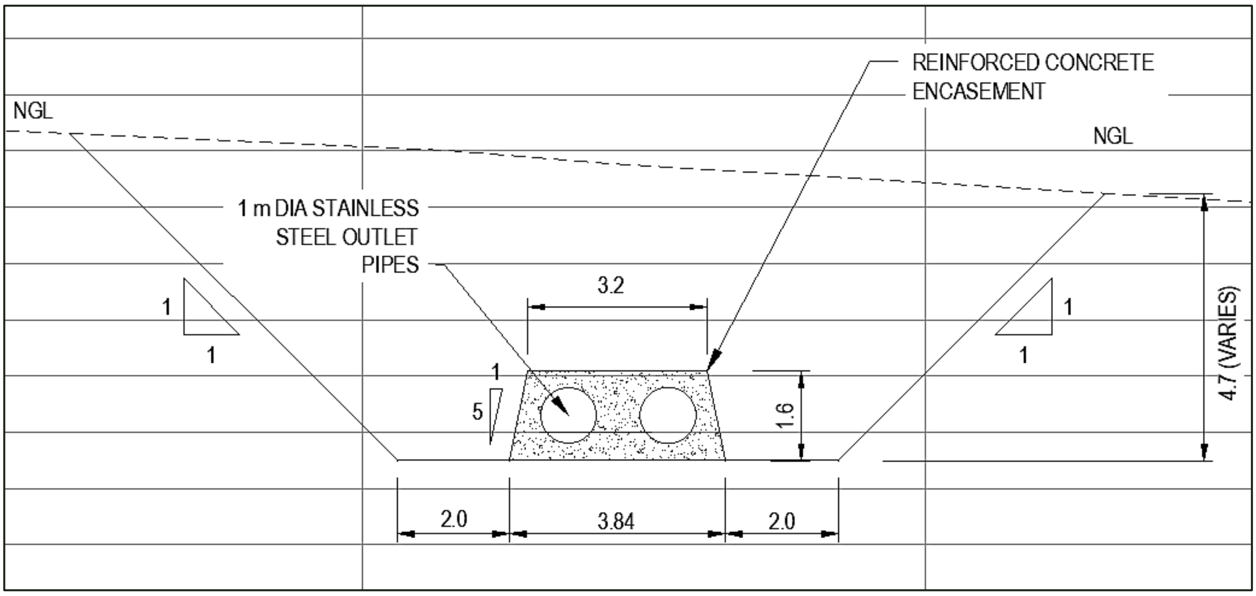

Figure 5.10: Cross-section of the outlet pipe encasement through the embankment

It is proposed that a scour outlet be located such that it will discharge into the lined spillway channel or river return channel. It is noted that the dam should be able to be drained in 30 days to comply with dam safety regulations.

The Ecological Water Requirement (EWR) has not been determined and no allowance is currently made in the design for accommodating this.

5.8.2 Outlet tower

The dam will be provided with a wet well outlet tower connected to the two inlet/outlet pipes. Two intake levels to the tower are proposed, the minimum level at 86.0 masl and another at 92.0 masl. This will allow multiple level draw-off from the dam for selecting the best quality water (if required). Vertical sluice gates at the two inlets on the tower face will allow upstream isolation of the outlet pipes and tower.

This arrangement of the outlet tower is shown in **Figure 5.11**. Further details can be found in **Appendix C-7** Drawing No. 112546-0000-DRG-CC-005.

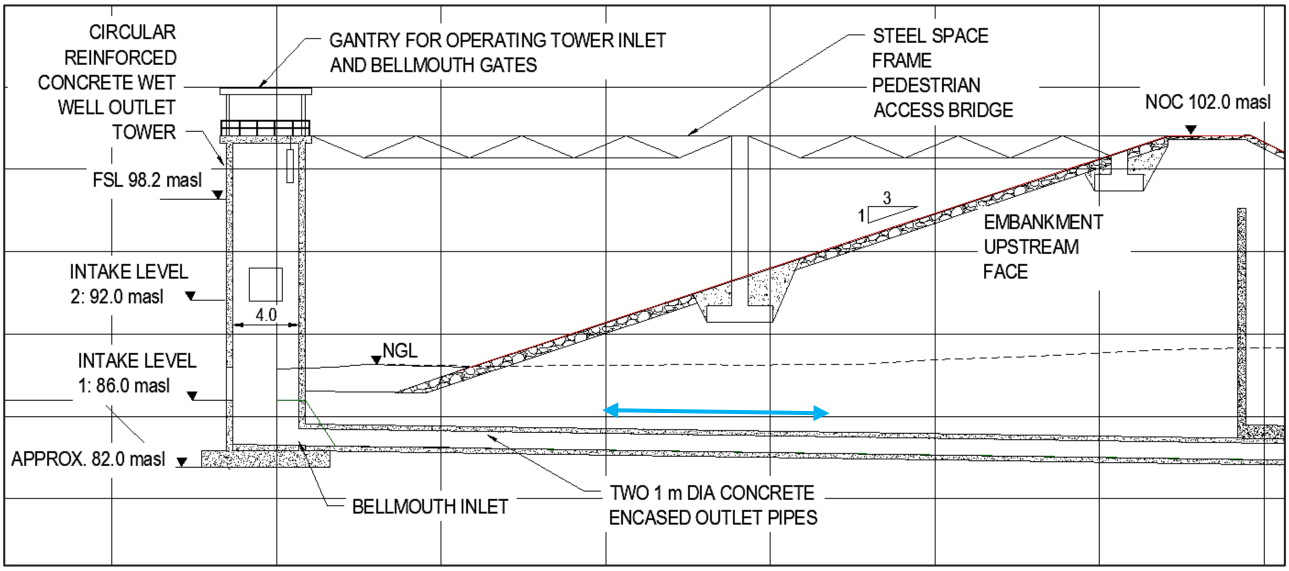

Figure 5.11: View of the outlet tower and access bridge

The proposed circular tower has an internal diameter of 4.0 m and estimated average wall thickness of 0.6 m. The estimated tower footing extends 1.5 m beyond the perimeter of the footprint of the tower. The structural dimensions and stability of the tower will be confirmed during the detail design stage.

Geotechnical investigations indicate that good rock foundations are not present at the site. It is proposed that the outlet pipe encasement is to be founded at minimum on the gravel-sand stratum of reworked terrace gravels at a maximum depth of approximately 3 m.

The outlet tower foundations are assumed to be considerably deeper due to the higher bearing capacity requirements, as well as providing space around the pipes and submergence depth to prevent the intake of air into the pipeline. The proposed design makes allowance for foundations up to 6 m deep for the tower, with cut slopes of 1V:1H.

The founding depth and exact location of the tower to suitable rock must be confirmed with further geotechnical investigations.

The tower is provided with a steel space-frame bridge, for pedestrian access, accessible from the embankment crest, with a mid-way column. The bridge is proposed to have two spans, from the embankment crest to a mid-way column and then to the tower.

5.9 Associated infrastructure

5.9.1 Electrical supply requirements

The proposed balancing dam does not require pumping for filling or supply. Nonetheless, an electrical supply must be provided to the dam to power the associated infrastructure, such as lighting, control and monitoring equipment, valves and actuators, etc.

There is currently no design for the electrical supply requirements to the proposed dam location. A lump sum allowance will be made in the cost and implementation analysis. It is presumed that the supply to the Scheepersvlakte Dam can be extended to the Coerney Dam site.

5.9.2 Access

The proposed dam is located in the valley adjacent (east) of Scheepersvlakte Dam. There are some gravel tracks on either side of the 'valley' in which the dam will be located.

The proposed design of the balancing dam has a spillway situated on the left abutment. Hence, approaching the embankment and outlet chamber from the right abutment is preferred as this does not require a crossing of the spillway channel. It is proposed that the track on the right abutment, leading from Scheepersvlakte Dam, will need to be upgraded. No design of the access road was done as part of the feasibility design, but a lump sum is included in the cost and implementation analysis.

5.9.3 Instrumentation

It is proposed that a number of simple, yet fundamental monitoring instruments should be included in the final design.

The most simple and fundamental is the monitoring of the embankment settlement, which is invaluable for safe operation of the dam. Further, embankment settlement monitoring during the formal dam safety inspections of a category III dam is a requirement of the dam safety legislation. Settlement beacons will greatly improve the accuracy of such monitoring and is therefore highly recommended. A row of settlement bacons on the downstream edge of the crest of the embankment along with reference beacons will be included.

To ease the operation of the water supply as a system, an electronic water depth gauge (e.g. vibrating wire piezometer) is proposed to enable remote water level monitoring of the proposed Coerney Dam. Potentially to compliment this, flow meters on the in- and outlet pipe branches to the dam are proposed. This will allow improved operation and monitoring of the dam's performance and losses (water fluctuation, water losses, filling period, storage, etc.). In any event it is likely that such monitoring instrumentation will form part of the requirements in the water use licence for this dam.

A flow meter on the Scheepersvlakte Farms irrigation offtake should also be installed to monitor their use.

The proposed dam design makes allowance for a sand chimney and finger drain system. The finger drains collect to the toe drain with intermittent manholes, which will enable the visual and volumetric monitoring of the seepage through the embankment at various points along the downstream toe. The finger drains also allow the drainage system to be divided up into compartments, which can be individually monitored for seepage. Should seepage then occur the problem compartment may be identified, and corrective action taken, if needed.

5.9.4 Maintenance

All the components of the dam have been sized with a focus on human-centred design, which aims to ensure that ample working space is available to allow easier maintenance. The design also provides all the barriers and relevant safety features for a safe working environment.

The provision of hoists to remove and maintain the valves, gates or trash racks has not been expressly included in the preliminary design of the proposed works. They have however been accounted for as a lump sum estimate in the costing.

5.10 River diversion

The river diversion strategy for the construction of Coerney Dam should be greatly simplified due to the apparent absence of regular flow in the river channel. It is expected that no regular river flows will need to be diverted during construction. Nonetheless, provision should be made for protection of the embankment during its construction, especially in the early stages when work is focused below ground level during the core trench construction.

The low recurrence interval floods typically used for selecting the river diversion capacity (between 1:2 year and 1:20 year) are presented in **Table 5.6**. These floods exceed the capacity through the outlet pipes if they were to be used as a diversion. There must thus be a coffer dam to help attenuate the incoming flow sufficiently to pass it through the pipes or a diversion canal. Preliminary diversion flood peaks and volumes along with expected coffer dam height and storage are shown in **Table 5.9** as an indication of the order of magnitude of the river diversion.

Table 5.9: Indication of diversion floods and potential coffer dam sizes

Aspect	1:10 year flood	1:20 year flood
Flood peak (m^3/s)	39	56
Flood volume (million m^3)	1.04	1.61
Coffer dam crest elevation (and height)	89 masl (7 m)	90.5 masl (8.5 m)
Coffer dam storage to crest (m^3)	590 000	960 000

It is noted that a coffer dam of this height, particularly at the Coerney site, which is a wide flat valley, requires a relatively large embankment, which must be built within one dry season. The revision of the flood hydrology, as discussed in Section 5.5, as well as the acceptable risk and size of the diversion flood to be accommodated, must be given further thought during the detail design.

Considering the potential size of the coffer dam, and the homogeneous embankment design, it is proposed that the coffer dam form part of the upstream fill of the main embankment.

Ground water was encountered during the geotechnical investigations indicating that subsurface water will need to be dealt with during construction. An allowance for river diversion and coffer dams is made in the cost analysis, but are not specifically included in the feasibility design of the dam (e.g. using coffer dam as shell zones).

6 Conveyance Infrastructure Design

6.1 Pipeline Design

6.1.1 Description of pipelines

The proposed scheme comprises two gravity pipelines, namely:

- A pipeline supplying water from the Kirkwood Primary canal to the proposed Coerney Dam; and
- A pipeline supplying water from the proposed Coerney Dam to a tie-in point on the existing Nooitgedagt pipeline that feeds the Nooitgedagt WTW.

The main advantages of the proposed scheme are that:

- The proposed Coerney Dam would substantially increase the raw water balancing storage capacity for water supply to NMBM; and
- The proposed pipeline route would bypass the high point on the existing gravity main feeding the Nooitgedagt WTW that limits the flow to the WTW. This will increase the hydraulic capacity during periods with low water levels in the dam.

The proposed horizontal pipeline alignments are shown on the layout plan; Drawing No. 112546-0000-DRG-CC-001 in **Appendix A-1**.

6.1.2 Pipeline horizontal alignments

The following factors were considered in determining the horizontal pipeline alignments:

- The pipelines should be easily accessible for future maintenance;
- The pipeline should tie into the existing Nooitgedagt pipeline downstream of the high point;
- The pipeline should be located outside the floodplain immediately downstream of the dam; and
- The pipeline should cross the proposed spillway and existing canals at optimum positions.

During the detailed design process, the pipeline route will need to be confirmed after discussions with affected land owners and authorities. Refinements may be required, depending on developments subsequent to the feasibility design.

6.1.3 Hydraulic input parameters

The following input parameters were used for the hydraulic calculations:

Pipeline from Kirkwood Primary Canal to proposed Coerney Dam

- START : Kirkwood Primary Canal
- Kirkwood Water Levels : Long weir overflow = 105.8 masl
- END : Coerney Dam
- Coerney Dam Water Levels : MOL = 86 masl, FSL = 98.2 masl
- Design capacity : 280 Mℓ/d (3.24 m^3/s)
- Length of section : 1 373 m

Pipeline from Coerney Dam to Tie-In on Nooitgedagt WTW pipeline

- START : Coerney Dam
- Coerney Dam Water Levels : MOL = 86 masl, FSL = 98.2 masl
- END : Nooitgedagt WTW Supply pipeline
- Design capacity : 280 Mℓ/d (3.24 m^3/s)
- Length of section : 1 573 m

The Hazen-Williams equation was used, which can be expressed as:

$$S = 6{,}84 * (v/c)^{1.85}/D^{1.167}$$

Where; "S" is the friction loss measured in m/m, "v" the velocity (m/s), "c" the Hazen-Williams friction coefficient and "D" the internal pipeline diameter (m). Typical Hazen-Williams friction factors for new and aged pipes are 140 and 115, respectively.

To determine the optimum pipeline diameter, the following calculations were performed:

- Using the MOL of 86.0 masl, the maximum flows that could be discharged for different pipe diameters were determined, as shown in **Table 6.1**;
- The minimum water levels required in the Coerney Dam to discharge the maximum flow of 280 Mℓ/d (3.24 m^3/s) were also calculated for various pipeline diameters, as shown in **Table 6.2**; and

- The results shown in **Table 6.2** were then compared against the Depth-Storage Curve for the dam (refer to **Figure 6.1**) to compare the percentage storage versus the minimum water level required to discharge the maximum flow of 280 Mℓ/d.

Table 6.1: Maximum flow with water level at MOL

HW=130; MOL = 86.0 masl	
Nominal Diameter (mm)	**Flow (Mℓ/d)**
1200	56.0
1300	71.7
1400	106.6
1500	138.9
1600	166.8

Table 6.2: Minimum water level required for flow of 280 Mℓ/d

HW=130; Q = 280 (Mℓ/d)		
Nominal Diameter (mm)	**Minimum water level (masl)**	**Storage capacity in dam (%)**
1200	94.4	54
1300	91.6	26
1400	89.8	17
1500	88.7	12
1600	87.9	8

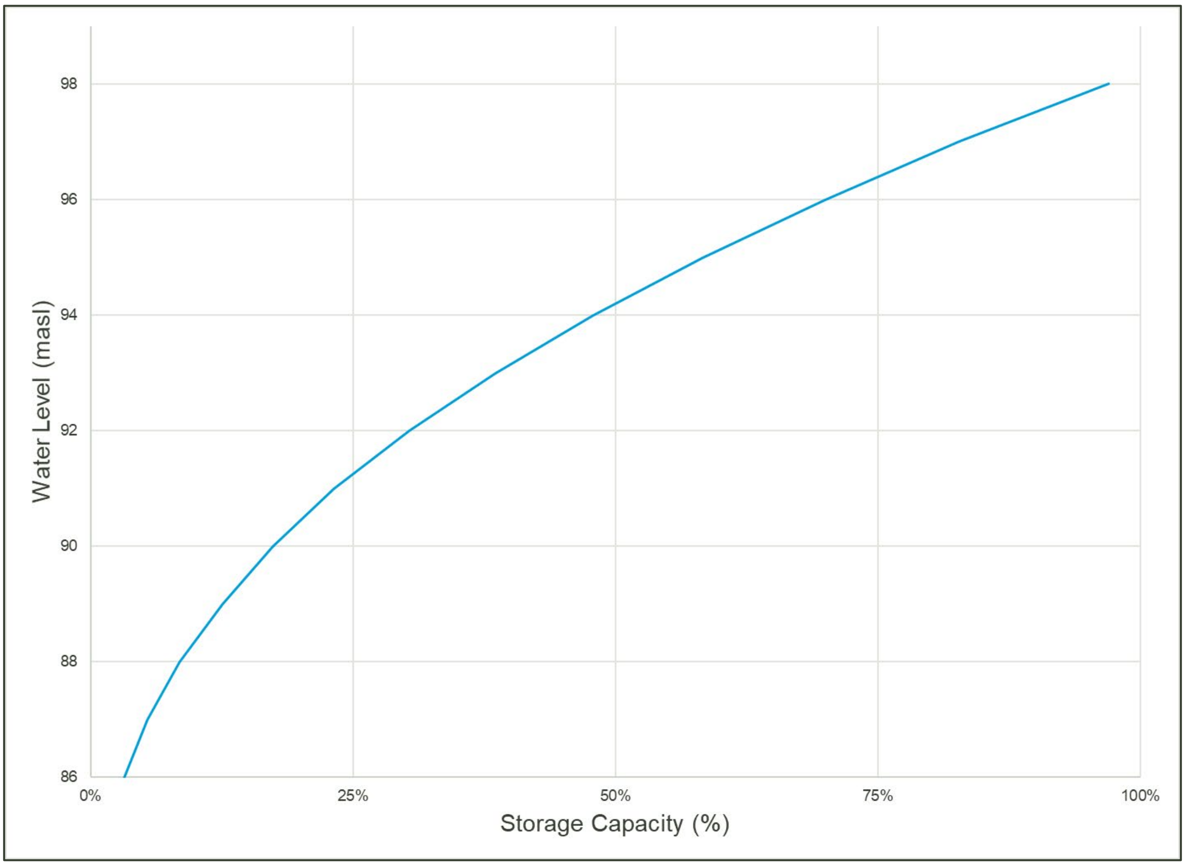

Figure 6.1: Depth-Storage Curve for Coerney Dam

It is evident from **Table 6.2** that a DN 1200 pipeline would require a minimum storage of 54% to discharge the maximum flow of 280 Mℓ/d, whereas a storage capacity of only 17% would be required for a DN 1400 pipeline. Given that the existing Nooitgedagt pipeline is a DN 1400 pipeline, and the fact that a storage capacity of only 17% would be required to discharge the design flow of 280 Mℓ/d, it is recommended that the proposed pipelines also be designed as DN 1400 pipelines. It is also evident from **Table 6.1** that a flow of 106.6 Mℓ/d can be discharged with the dam level at MOL, i.e. almost 40% of the maximum flow rate.

Table 6.3 shows the flow rates that would be achieved in a DN 1400 pipeline for different Hazen-Williams friction coefficients (i.e. 115 being an aged pipeline and 140 being a newly installed pipeline).

Table 6.3: Flow rates for Hazen-Williams (HW) friction coefficients in DN 1400 pipeline

Water level in dam (masl)	HW = 115	HW = 130	HW = 140
	Q (Mℓ/d)	Q (Mℓ/d)	Q (Mℓ/d)
86.0	94	106	115
87.0	207	234	252
88.0	222	251	270
89.0	236	267	288
89.8	247	280	301

It is evident from **Table 6.3** that the flows calculated with a Hazen-Williams friction coefficient of 130 would increase by approximately 8% in a newly installed pipeline and would reduce by approximately 12% in an aged pipeline.

Figure 6.2 shows the hydraulic gradient lines of the Kirkwood Primary Canal to the Coerney Dam pipeline for a flow of 280 Mℓ/d and with the Coerney Dam water levels at MOL (i.e. 86 masl) and FSL (i.e. 98.2 masl).

It is evident from **Figure 6.2** that the hydraulic gradient line at the Kirkwood Canal would be 102 masl when Coerney Dam is at its FSL of 98.2 masl. The floor level of the Kirkwood Canal at the offtake point is approximately 103.9 masl with the long weir having a height of 105.8 masl. It would therefore be possible to discharge 280 Mℓ/d from the Kirkwood Canal to the Coerney Dam, even when the dam is at FSL.

Figure 6.3 shows the hydraulic gradient line from the Coerney Dam to the tie-in point on the Nooitgedagt pipeline with the dam at a level of 90 masl and discharging a flow of 280 Mℓ/d.

It is evident from **Figure 6.3** that a residual pressure of approximately 3 m would be available at the tie-in point.

Figure 6.4 shows the hydraulic gradient line from the Coerney Dam to the Nooitgedagt WTW with the dam level at 90 masl and a discharge of 280 Mℓ/d.

It is evident from **Figure 6.4** that the high point located at chainage 4 500 m would create a hydraulic control, meaning that a short section of the existing pipeline would flow partially full immediately downstream of this high point. The proposed pipeline from Coerney Dam is therefore connected to the existing pipeline downstream of this high point.

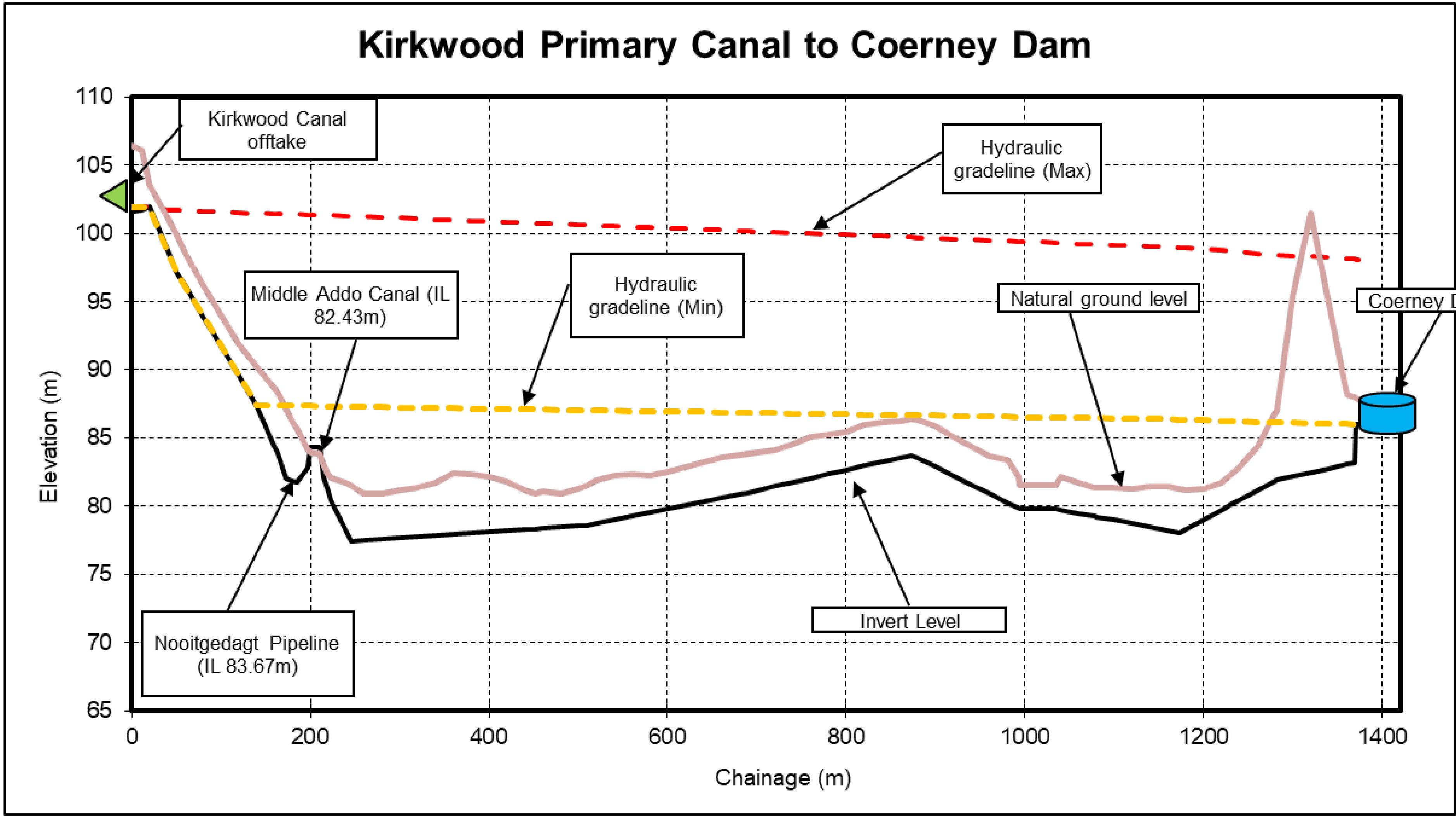

Figure 6.2: Kirkwood Canal to Coerney Dam: HGL for 280 Mℓ/d in aged DN 1400 pipeline

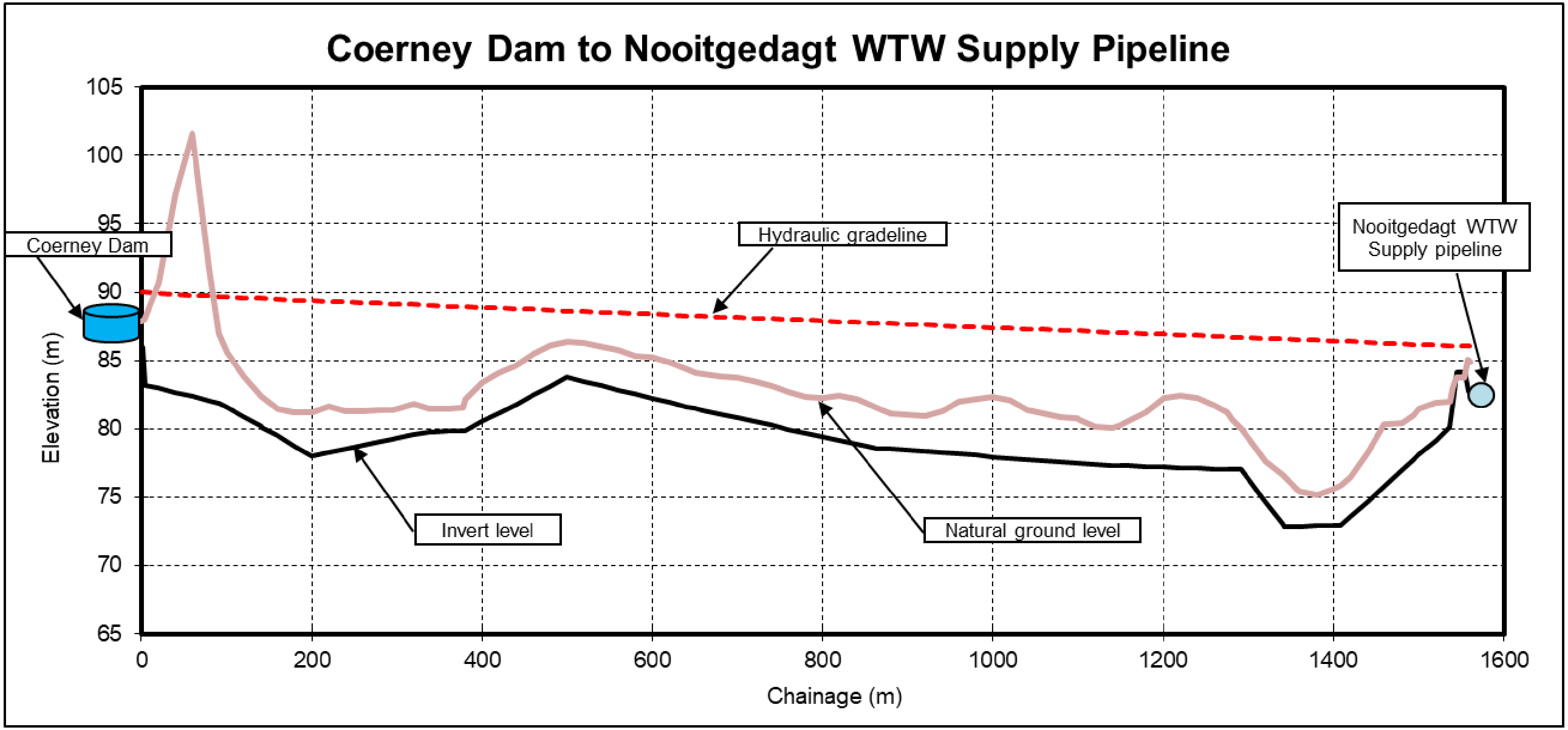

Figure 6.3: Coerney Dam to Tie-In Point: HGL for 280 Mℓ/d in aged DN 1400 pipeline

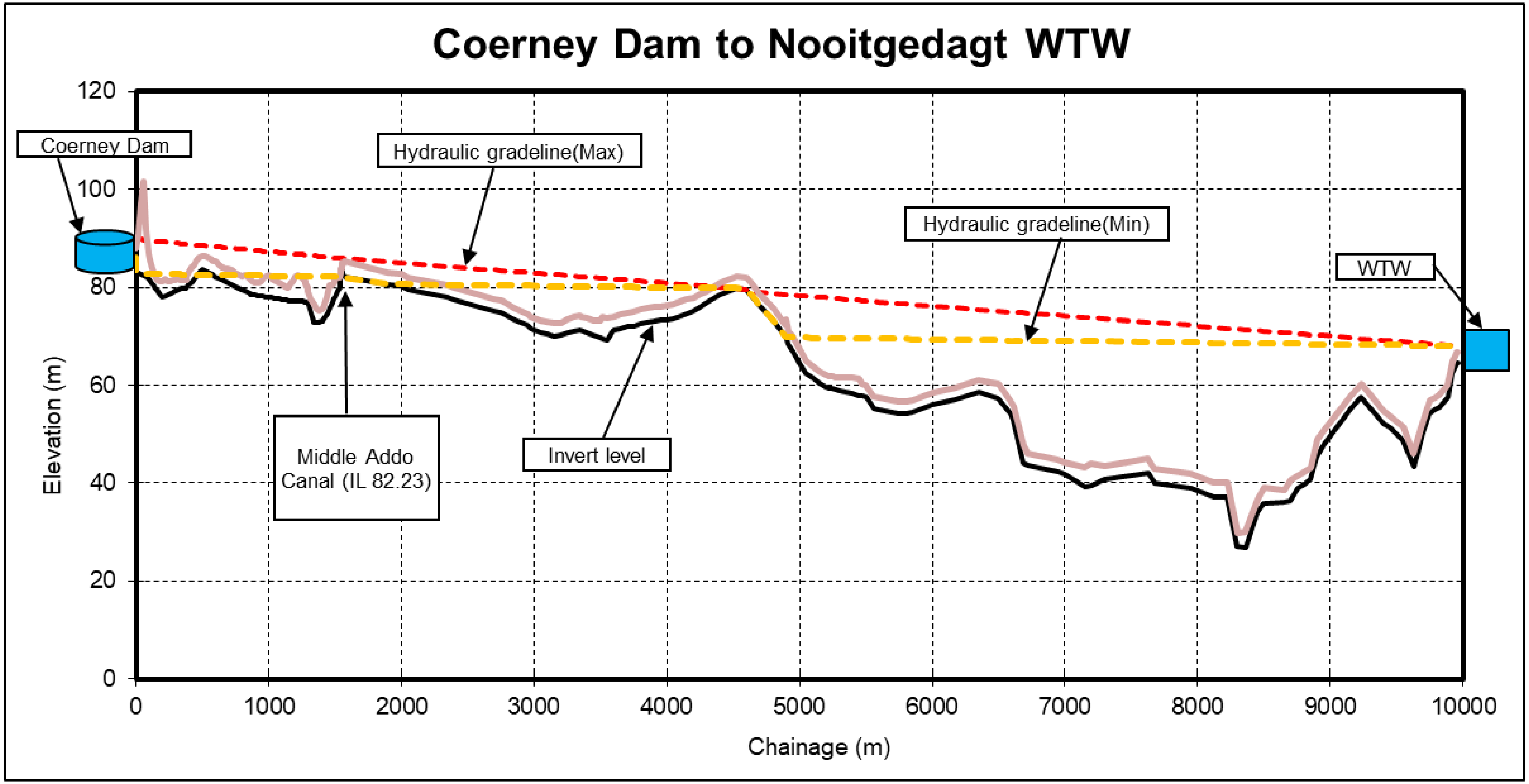

Figure 6.4: Coerney Dam to WTW: HGL for 280 Mℓ/d in aged DN 1400 pipeline

6.1.4 Pipe material selection

It is evident from **Figure 6.2** and

that maximum working pressures would not exceed 30 m in the new pipelines. Given the pipeline diameter and expected working pressures, it would be possible to consider the following as suitable pipe materials:

- Glass Reinforced Polyester (GRP);
- Ductile Iron; and
- Steel.

Each of these materials has advantages and disadvantages with respect to hydraulic capacity, price, ease of installation, need for corrosion protection, ease of repairs, etc.

GRP pipes would most likely be the preferred pipe material based on cost, no need for cathodic protection, and lowest internal roughness. The GRP pipes are, however, prone to damage during the construction phase of the project. They require utmost care when laying, both in the quality of the bedding material and the compaction. Special care must also be taken when connecting GRP pipes to other pipe materials or at structures, as differential settlement can result in the cracking of the pipes. Large anchor/thrust blocks would also be required at bends.

Steel pipes are typically more economical than ductile iron pipes when used in low pressure applications due to the thinner wall thicknesses. Steel pipes also have the benefit that no anchor/thrust blocks would be required. Steel pipes are also preferred due to the ease with which connections to existing pipelines can be made and the flexibility to make site-specific changes by mitring of bends. The potential negative aspect of the steel pipes is that cathodic protection would most likely be required.

Given the advantages of steel pipes, they are recommended as the preferred pipe material for the proposed pipelines.

Consideration should be given during the detailed design to the following aspects:

- Internal lining - whether the pipelines should be cement-mortar or epoxy lined.
- External coating - a choice must be made between polyurethane, medium density polyethylene (i.e. Sintakote), 3LPE and polymer modified bitumen.
- Quality inspection at the factory of the pipes, including lining and coating, is essential.
- Further quality inspection of the pipes on site, before and after installation is required.
- An external inspector, independent of the pipe manufacturer, is required.

6.1.5 Preliminary wall thickness calculation

Based on the limited geotechnical information available, an assumption was made that the minimum E-value of the native material would be in the order of 6 MPa. Imported bedding material would be required along the majority of the pipeline route and would therefore have an E-value of at least 12 MPa. This will provide a combined Modules of Soil Reaction of 7.20 MPa in accordance with AWWA M11's guidelines.

The recommended pipeline diameters and wall thicknesses are based on the hydraulic analyses and external loads. The following assumptions were also made in the calculations:

- A live wheel load of 80 kN.
- Maximum working pressure of 30 m.
- Maximum surge pressure of 40 m (assumed)
- Maximum water table at 100 mm below natural ground level.
- Minimum soil cover on the pipe of 1.0 m.
- Maximum soil cover on the pipe of 3.4 m.
- Grade X52 steel will be used.
- Factor of safety for working pressures of 1.67 (based on DWS guidelines).
- Factor of safety for surge pressures of 1.67 (based on DWS guidelines).
- Factor of safety for combined stresses of 1.65 (based on DWS guidelines).
- Maximum deflection due to cement-mortar lining of 3%.

The existing gravity supply pipeline to Nooitgedagt WTW is a DN 1400, 11 mm Grade B steel pipe. Based on the assumptions and calculations above, the proposed pipelines will be DN 1400, Grade X52 steel, with a yield strength of 358 MPa and a recommended wall thickness of 10 mm. The maximum soil cover of 3.4 m will have to be adhered to during the detailed design of the vertical alignment of the pipelines.

Thicker wall thickness might be required if the E-value of the native soil is worse than expected or if the E-value of the bedding material is lower than anticipated.

It is proposed that further geotechnical investigations be undertaken to verify the E-value of the native soil. The wall thickness provided above is therefore only a preliminary wall thickness, which should be verified once the additional geotechnical information becomes available.

6.2 Connections and associated impacted infrastructure

6.2.1 Control narrative of scheme

The proposed connecting pipework for the new dam to the existing infrastructure is shown schematically in **Figure 6.5**.

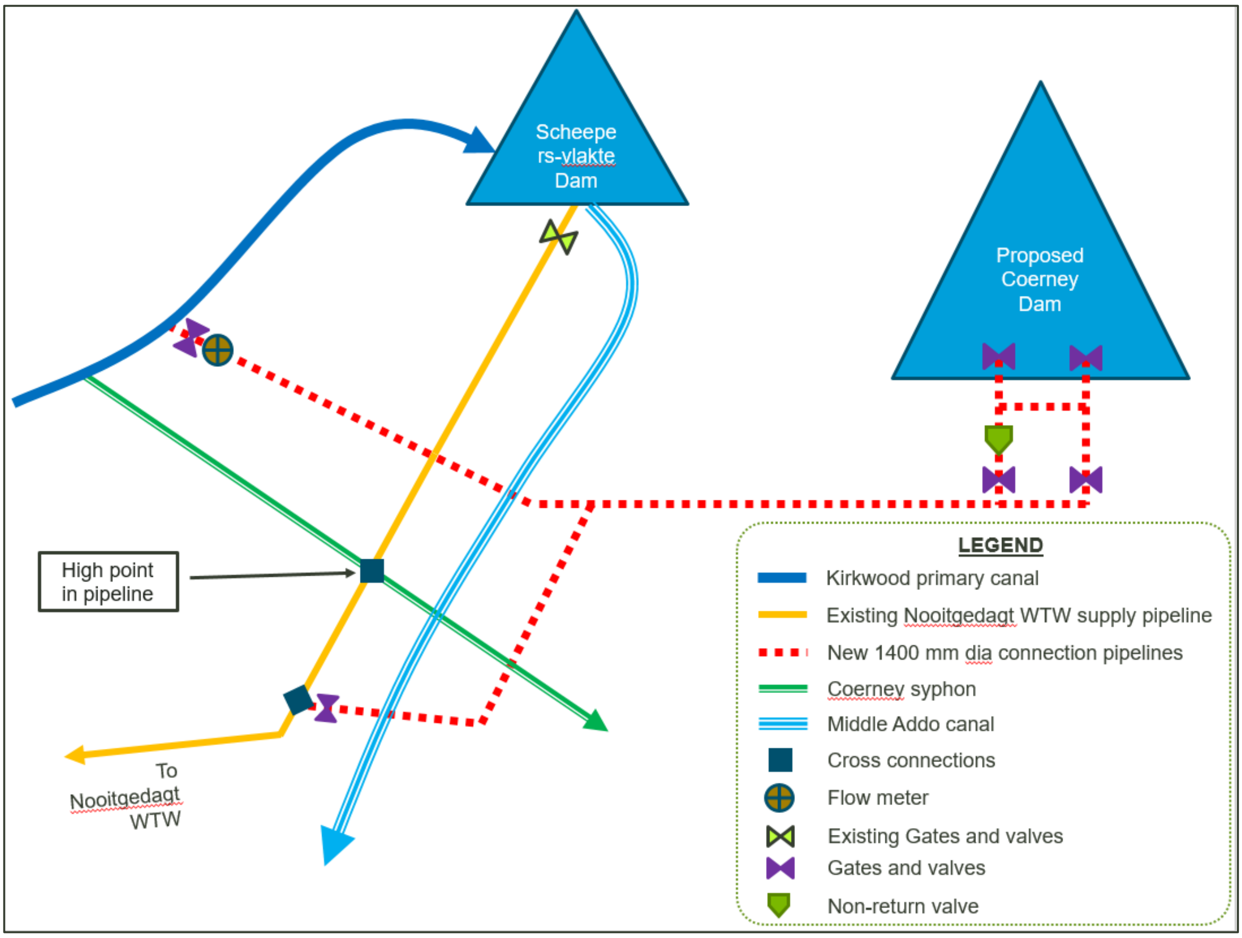

Figure 6.5: Schematic layout of the proposed Coerney Dam and connecting pipelines

The proposed Coerney Dam will be supplied with water from the Kirkwood primary canal, through a new 1400 mm diameter steel pipeline. The pipeline will abstract water from a new offtake on the canal. The proposed intake is located at the end of the Kirkwood primary canal, between the Coerney syphon inlet and the long weir, just upstream of the Scheepersvlakte Dam. Locating the intake just upstream of the long weir will provide head from the weir.

Under normal operating conditions, the existing valve on the pipeline immediately downstream of the Scheepersvlakte Dam will be closed with all the other valves shown in **Figure 6.5** being open. Water will be abstracted from the Kirkwood primary canal and fed directly into the Nooitgedagt pipeline, except that surplus water will flow into the Coerney Dam. In the event that the demand of the Nooitgedagt WTW exceeds the supply from the Kirkwood primary canal, the shortfall will be supplied automatically from the Coerney Dam.

Should the Coerney Dam reach its full supply level, the inlet valve on the supply pipeline to the dam will be closed. The outlet pipeline, which is fitted with a non-return valve to prevent inflow into the dam, will remain open to supply the WTW with water as required.

Given that the system is downstream controlled (i.e. at the inlet to the WTW), the flow into and from Coerney Dam would automatically adjust, based on the contribution from the Kirkwood primary canal and demand from the WTW.

Once the Coerney Dam is in operation, the Scheepersvlakte Dam will function as a balancing dam for irrigation only. Its connecting pipework will be isolated, but will remain in place with a connection to the Coerney Dam for emergency supply to the WTW.

6.2.2 New offtake at Kirkwood primary canal

The proposed offtake from the Kirkwood primary canal will be located downstream of the Coerney syphon intake, and just upstream of the long weir, which will provide head to the new intake.

It is proposed that the new offtake comprises an adjustable weir that would allow for regulating of the flow that could be discharged from the canal to the WTW or to the Coerney Dam. At the offtake location, the canal has a floor level of 103.9 masl with the top of the long weir at a level of 105.8 masl. The length of the adjustable weir (or sluice gate) must therefore be such that the head required would be less than the overflow level of the long weir.

Table 6.4 shows the head required for different weir widths to discharge different flows, e.g. for a weir width/length of 1.0 m, a discharge head of 1.426 m would be required for a flow of 250 Mℓ/d.

Table 6.4: Kirkwood canal offtake: Flow depths for different flows and weir widths

Width of weir (m)	Flow (Mℓ/d)					
	50	100	150	200	250	280
0.5	0.774	1.229	1.610	1.950	2.263	2.441
1.0	0.488	0.774	1.014	1.229	1.426	1.537
1.5	0.372	0.591	0.774	0.938	1.088	1.173
2.0	0.307	0.488	0.639	0.774	0.898	0.969
2.5	0.265	0.420	0.551	0.667	0.774	0.835
3.0	0.234	0.372	0.488	0.591	0.685	0.739

It is proposed that the floor level be raised at the off-take to a level of 104.2 masl to mitigate the risk of sediment being transported from the bottom of the canal to the pipeline. This leaves 1.6 m as the maximum head available to discharge a flow of 280 Mℓ/d, meaning that a 1.5 m weir length would still allow just over 400 mm of freeboard.

The water from the off-take will discharge into a wet well that will be piped through a magnetic flow meter. The display from the flow meter will be positioned next to the adjustable sluice gate, which will allow the weir to be adjusted to discharge a certain flow.

The proposed offtake configuration is shown on Drawing No. 112546-0000-DRG-CC-101 in **Appendix D-1** and in **Figure 6.6**.

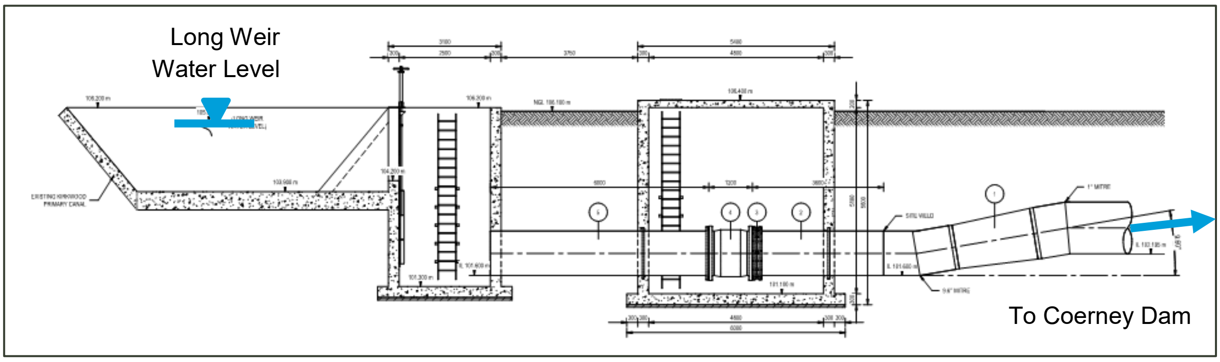

Figure 6.6: Section view of Offtake at Kirkwood Canal

6.2.3 Coerney Dam Inlet/Outlet Chamber

The proposed dam will be supplied from the Kirkwood primary canal with a DN 1400 pipeline, which will also be used to transfer water to the tie-in point on the existing Nooitgedagt WTW pipeline.

The pipe for supplying water to and from the dam will bifurcate into an inlet and outlet branch at the outlet chamber at the downstream toe of the embankment. The inlet branch will have an isolation valve for shutting off supply when the dam is full; this is to prevent spilling canal water. The outlet branch will be fitted with a non-return valve and an isolation valve upstream and downstream. The non-return valve will ensure that water can be 'automatically' supplied from the dam in the event where the inlet has been shut to avoid spilling of the dam when it is full. The isolation valves will ensure that the non-return valve can be serviced while the inlet pipe remains in operation.

The inlet and outlet pipe branches will reduce from DN 1400 to DN 1200 at the bifurcation and reduce from DN 1200 to DN 1000 after the cross connection, before passing through the embankment in a concrete encasement. Both pipes will connect to a wet well outlet tower in the dam basin.

The Inlet/Outlet chamber is shown on Drawing No. 112546-0000-DRG-CC-100 in **Appendix D-2** and in **Figure 6.7**.

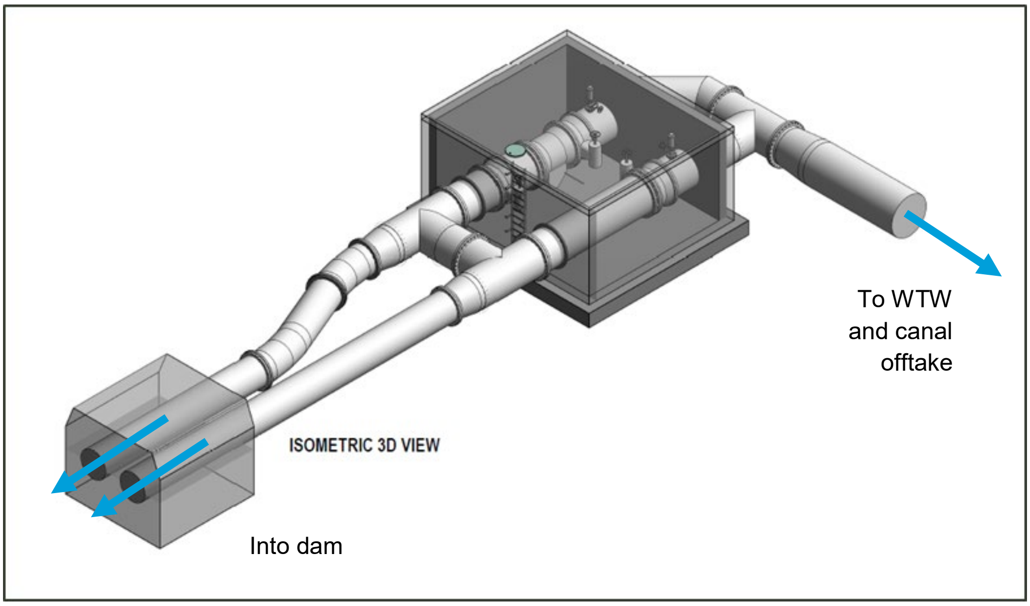

Figure 6.7: Isometric view of Coerney Dam Inlet/Outlet Chamber

6.2.4 Tie-in to existing Nooitgedagt WTW supply pipeline

A connection needs to be made into the existing 1400 mm diameter Nooitgedagt WTW supply pipeline. The existing pipeline is manufactured from Grade B steel with a cement-mortar lining and bitumen fibreglass coating, and has an 11 mm wall thickness at the connection point. The tie-in will be located downstream of the cross connection with the Scheepersvlakte syphon and downstream of the existing high point in the existing supply line.

The tie-in will comprise a 1400 mm x 1400 mm equal tee that will be cut into the existing pipeline. The branch of the tee will be fitted with an isolation valve to close the Coerney Dam's supply should maintenance be required on this pipeline. The tie-in detail is shown on Drawing No. 112546-0000-DRG-CC-102 in **Appendix D-3** and in **Figure 6.8**.

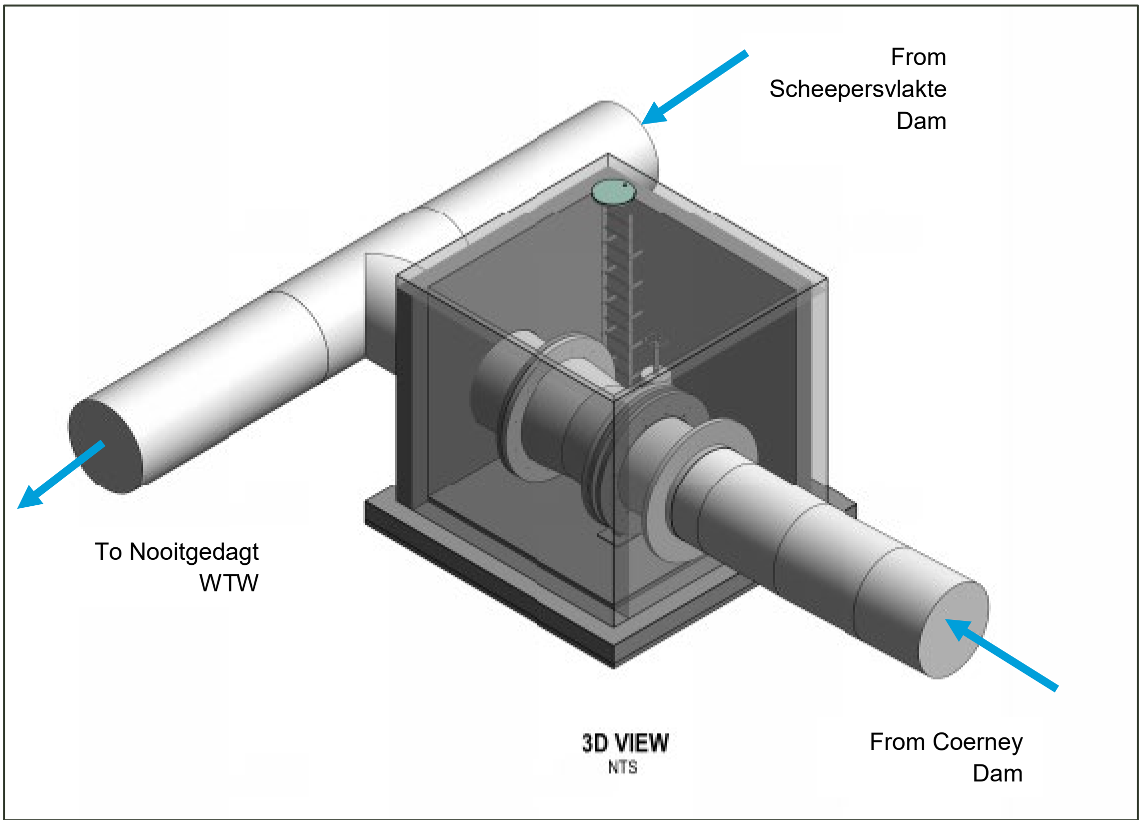

Figure 6.8: Isometric view of Tie-in to Existing Nooitgedagt WTW Supply Pipeline

6.2.5 Middle Addo Canal Crossings

The new supply pipeline to the Coerney Dam and the bypass pipeline will need to cross the Middle Addo canal. The approximate elevations and width of the canal at the points of crossing are indicated in **Table 6.5**.

Table 6.5: Middle Addo Canal details

Canal details	New Supply Pipeline to Coerney Dam	Bypass Pipeline to WTW
Canal width	5.35 m	5.35 m
Centre line canal	82.43 masl[1]	82.23 masl[1]
Left Bank canal	83.91 masl[1]	83.73 masl[1]

[1] Levels are approximate

The exact positions of the Middle Addo canal crossing must be verified during the detailed design of the pipelines.

It is proposed that the pipeline be installed over the canal (above ground) not to impact the operation or integrity of the canal, and to facilitate easier maintenance, if required. The 1400 mm diameter steel pipe will serve as the pipe bridge with concrete supports on either side of the canal. An air valve will have to be installed at the high point created by the canal crossing. The air valve will also serve as an access point into the pipeline for maintenance purposes. Additional protection of the exposed pipe may be required.

A typical detail of the pipe bridge is shown in **Figure 6.9**.

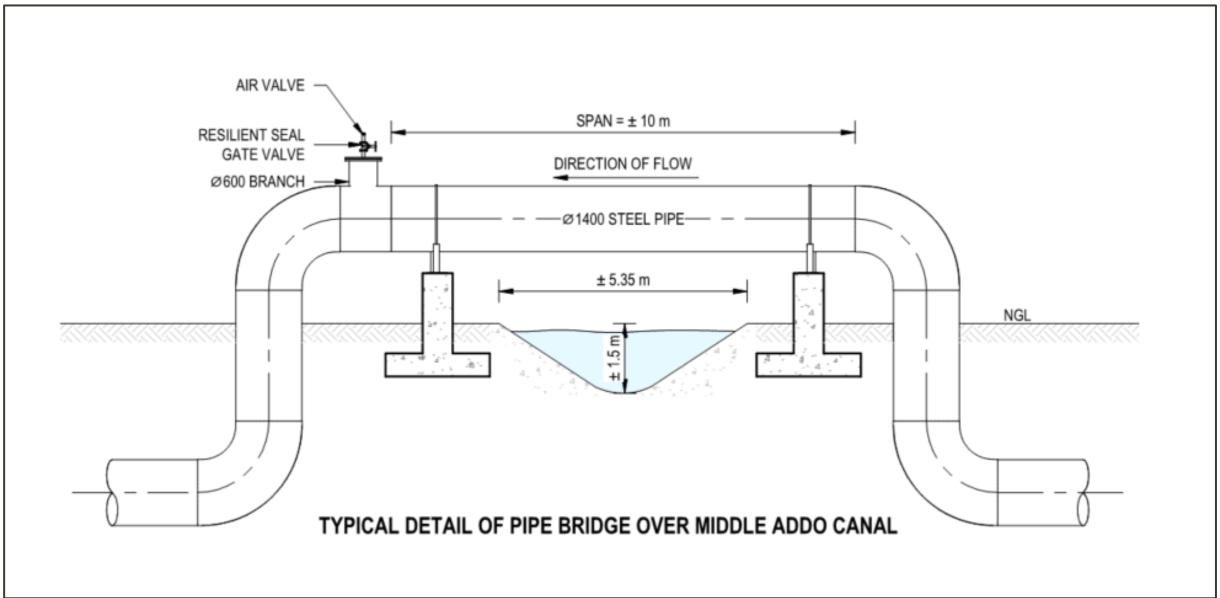

Figure 6.9: Typical detail of pipe bridge over Middle Addo Canal

6.2.6 Proposed Coerney Dam spillway crossing (right abutment spillway)

The Coerney Dam spillway is to be constructed on the left abutment, which will have no impact on the proposed supply pipeline to the dam. However, if the Coerney Dam spillway is positioned on the right abutment, the proposed DN 1400 pipeline will need to cross the spillway. It is proposed for this option that the pipeline crosses under the spillway just downstream of the stilling basin as shown in **Figure 6.10**. The pipeline will most likely be encased as part of the stilling basin's end sill.

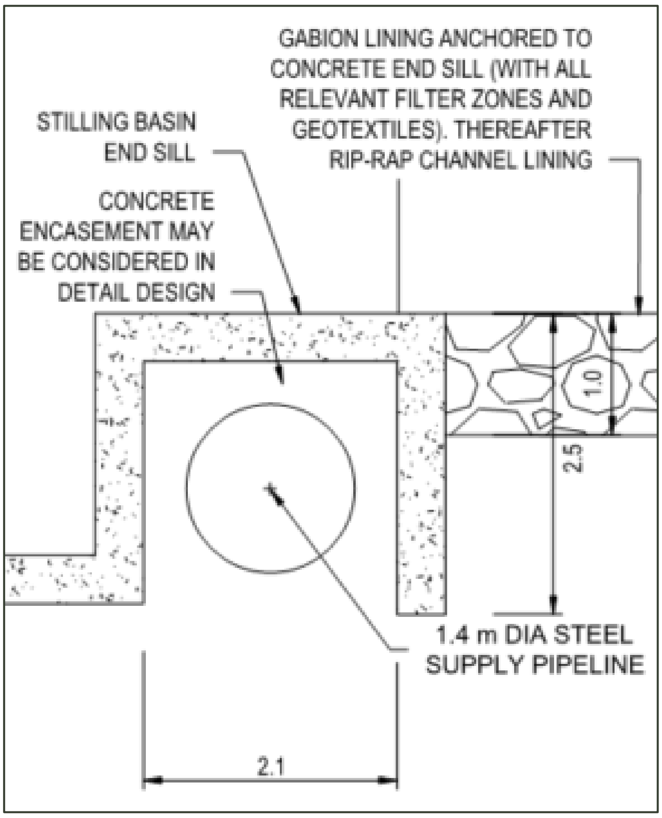

Figure 6.10: Typical detail of pipe underneath spillway

6.2.7 Existing Nooitgedagt WTW supply pipeline crossing

The proposed pipeline from the Kirkwood primary canal to the Coerney Dam will have to cross the existing Nooitgedagt WTW supply pipeline.

The proposed pipeline would need to cross under the existing Nooitgedagt pipeline due to the limited soil cover on the existing pipeline at the point of crossing. The invert level at the proposed crossing is approximately 83.7 masl. The new pipeline will have a 300 mm clearance between the invert of the existing pipeline and the crown of the new pipeline. The existing pipeline will have to be excavated by hand to confirm the exact levels during the construction phase of the project and to ensure that no damage is done to the existing pipeline.

6.2.8 Syphon under Sundays River

An additional syphon under the Sundays River on the existing Nooitgedagt WTW supply pipeline is proposed to:

- Reduce the risk of supply failure in the event of damage to the existing syphon; and
- Mitigate the risk due to the new balancing storage being located on the opposite side of the river, relative to the WTW.

The additional syphon under the Sundays River will be concrete encased. The top of the reinforced pipe encasement should be below riverbed level. The length of the encasement is assumed to be approximately 105 m (the same as the existing pipeline).

It is proposed that the new syphon be located upstream of the existing syphon at a suitable point to cross the river. The new syphon should be separate from the existing syphon at a suitable distance upstream. The additional syphon will potentially also be on private property and landowner discussions will need to be initiated.

An air valve chamber and a scour valve chamber will have to be installed, and tie-ins made into the existing pipeline. The air valve will also serve as an access point into the new pipeline for maintenance purposes. The tie-ins will comprise 1400 mm x 1400 mm equal tees that will be cut into the existing pipeline and installed on the new syphon pipeline. Isolating valves will be provided so that the new syphon can be isolated, as it will only be used if the existing syphon is damaged or when maintenance is required.

As-built drawings and/or information will have to be obtained on the existing syphon during the detailed design phase of the project.

Apart from doubling the syphon it is also recommended that an adequate stockpile of replacement pipes be kept, to be able to quickly repair the pipeline in case of failure.

7 Cost Estimate

This chapter discusses the dimensions, assumptions and other factors affecting the costing of the various components of the scheme and presents capital, operational and other estimated costs.

7.1 Coerney Dam

7.1.1 Embankment materials

The proposed embankment dam design has typical embankment slopes of 1V:3H upstream and 1V:2H downstream. The dam has a crest length of 600 m and a crest width of 5 m. The lowest level at the valley bottom is 81.5 masl with a non-overspill crest (NOC) level of 102.0 masl, which results in a maximum wall height of 20.5 m.

As part of the site preparations, the topmost 0.3 m - 0.5 m of soil will need to be removed from the embankment footprint (4.3 ha) before placement of the embankment fill material. This topsoil should be stockpiled for use on disturbed areas.

The total embankment volumes are summarised in **Table 7.1**. The material for the embankment homogeneous fill zone will be excavated from within the dam basin, which is within normal free-haul distance of 1 km, and no allowance is thus made for overhaul. It has been assumed that the essential excavations from the spillway and discharge channel (approximately 94 000 m^3) will be used in the embankment construction, with no double handling of these materials.

Table 7.1: Embankment material quantities and sources

Zone	Volume	Source
Homogeneous fill	350 000	Dam basin
Upstream rip-rap	12 650	Imported
Sand and gravel filters	5 100	Imported
Rock toe drain	2 370	Imported

Materials, such as fine and coarse aggregate for concrete and sand for filters, were not found in the basin or near the site and will have to be imported from commercial sources. No project-specific borrow areas were identified or investigated. The current costing makes allowance for their importation from commercial sources.

A few possible commercial sources for sand and coarse aggregates have been identified, but all are located some distance away from the Coerney Dam site. These are discussed in **Section 5.3.6.2** of this report.

7.1.2 Foundation treatment

The 'groutability' of the weathered rock encountered for the foundations of the dam core trench, outlet pipe encasement and outlet tower is uncertain.

No allowance has been made for consolidation or curtain grouting under the embankment. Some allowance is made for curtain grouting along the spillway overflow structure centreline.

Special mention should be made of the mudrock, which is susceptible to slaking. This rock will require immediate protection after exposure to prevent deterioration before construction/covering will commence. No allowance has been made for this.

7.1.3 Spillway

The spillway is excavated into the left abutment. The spillway overflow sill is an ogee-shaped mass gravity concrete overflow weir that is 6.35 m high and 50 m long. In general, the upper soils are underlain by weak bedrock that would be susceptible to erosion. It is thus required that the spillway side channel and discharge channel are lined with reinforced concrete. The side channel liner extends to the NOC, as it will also act as a retaining wall where the channel abuts to the embankment. The thickness of the liner is 0.4 m and the discharge channel is lined to the safety evaluation flood (SEF) flow depth of 1.7 m.

The discharge channel ends in a rectangular stilling basin. The basin is 20 m long with 6 m high side walls, which have a slight batter (1H:5V).

7.1.4 Inlet/Outlet works configuration

The outlet works of the dam will be located on the left abutment (eastern bank) of the valley. The pipe for supplying water to and from the dam will bifurcate into an inlet and outlet branch at the downstream outlet chamber, which is situated at the downstream toe of the embankment. The costing for this pipe, up to the start of the encasement, is described in Section 6, which addresses the conveyance infrastructure. The inlet/outlet pipes through the embankment have been included in the dam costing. The two stainless steel DN 1000 mm inlet/outlet pipes pass through the embankment, where they are encased in reinforced concrete.

7.1.5 Outlet tower

The inlet and outlet pipes will be connected to a circular (4 m internal diameter) reinforced concrete wet-well outlet tower. The tower is founded on a concrete base slab 1.0 m thick, extending 1.5 m past the wall perimeter. The walls are an average of 0.6 m thick. The tower is provided with two intake levels, which can be controlled with gates operated from the top of the tower. The tower will be accessed via a steel pedestrian space frame bridge with a mid-length column dividing the bridge into two spans.

The access bridge, intake gates, gantry and other mechanical equipment in the tower are accounted for with lump sum values.

7.1.6 Electrical supply requirements

There is currently no design for the electrical supply to the proposed dam location. It is presumed that the supply to the Scheepersvlakte Dam can be extended to the Coerney Dam site. A lump sum allowance has been made in the costing for this extension.

7.1.7 Access

There are some gravel tracks on either side of the valley of the proposed dam site. It is proposed that the track on the right abutment leading from Scheepersvlakte Dam be upgraded. An overall cost for access roads has been made on a per-kilometre rate and provision of 1.5 km of roads.

7.1.8 Instrumentation

A number of simple monitoring instruments were discussed and proposed in the feasibility design of the dam. A percentage allowance in the costing has been included to allow for these items.

7.1.9 River diversion

The river diversion strategy for the construction of Coerney Dam is relatively rudimentary due to the apparent absence of regular flow in the river channel. A lump sum provision has been made in the cost estimate to account for the provisions and risks associated with the river diversion to deal with water and floods. The estimate is a lump sum based on the size of the diversion embankment required and an all-in rate.

7.1.10 Construction cost estimate

A detailed cost estimate for the balancing dam is provided in **Appendix E-1**.

Table 7.2 shows a summary of the costs for the various components and types of work discussed above.

Table 7.2: Cost estimate summary for Coerney Dam (excluding VAT)

No	Description	Amount (R million)
1	Clearing	0.195
2	River diversion	10.000
3	Excavations	20.262
4	Drilling and grouting	0.677
5	Embankment fill materials	41.717
6	Concrete works	52.570
7	Mechanical Items	22.525
8	Miscellaneous	8.877
	Sub-total	**156.823**
9	Preliminary and General Items (50%)	78.412
10	Access and electrical supply	2.750
11	Contingencies (25%)	59.496
	Total	**297.481**

7.1.11 Operation and maintenance costs

The operation and maintenance costs of the various dam components have been included as an annual cost, based on a percentage of the construction value.

The components are divided into three sections:

- Civil works, which includes the concrete spillway, intake tower and access bridge, the access roads and pipelines.
- The mechanical works, which includes hoisting equipment, valves and gates.
- The dam, which includes the embankment and dam basin.

A simple annual cost estimate for the three categories has been determined, based on a percentage of the construction value. The construction value used includes 25% contingencies and excludes the cost of the river diversion and the 50% preliminary and general charge items.

The operation and maintenance costs for the dam, based on the above, are shown in **Table 7.3**.

Table 7.3: Operation and maintenance costs for Coerney Dam (excluding VAT)

No	Description	Percentage	Construction value (R million)	Annual Amount (R million)
1	Civil works	0.5%	89.65	0.448
2	Mechanical works	4.0%	18.750	0.750
3	Dam (embankment)	0.25%	78.564	0.196
Total			**186.967**	**1.394**

7.2 Conveyance infrastructure

7.2.1 New offtake at Kirkwood primary canal

The new offtake from the Kirkwood primary canal comprises a 1.5 m wide adjustable weir, which will allow for regulating of the flow that can be discharged from the canal to the WTW or to the Coerney Dam. The weir level will be adjusted by means of a manually operated sluice gate, which will allow water to discharge from the canal into a wet well. Water will be piped from the wet well through a magnetic flow meter. The display from the flow meter will be positioned next to the adjustable sluice gate, which will allow the weir to be adjusted to discharge a certain flow.

7.2.2 Inlet/outlet to proposed Coerney Dam

The proposed dam will be supplied from the Kirkwood primary canal through a DN 1400 pipeline, which will also be used to transfer water to the tie-in point on the existing Nooitgedagt WTW pipeline.

The pipeline for supplying water to and from the dam will bifurcate into an inlet and outlet branch at the outlet chamber, which is situated at the downstream toe of the embankment. The inlet branch will have an isolation valve for shutting off supply when the dam is full. The outlet branch will be fitted with a non-return valve and an isolation valve, both upstream and downstream. The non-return valve will ensure that water can be 'automatically' supplied from the dam in the event where the inlet has been shut to avoid spilling of the dam when it is full. The isolation valves will ensure that the non-return valve can be serviced while the inlet pipe remains in operation.

The inlet and outlet pipe branches will reduce from DN 1400 to DN 1200 at the bifurcation and reduce from DN 1200 to DN 1000 after the cross connection, which is before passing through the embankment in a concrete encasement.

7.2.3 New connecting pipework

A DN 1400 steel pipeline is required between the Kirkwood primary canal offtake and the proposed Coerney Dam, as well as from the proposed dam to a tie-in point on the existing DN 1400 pipeline supplying the Nooitgedagt WTW.

The total length of the two steel pipelines is approximately 2 950 m. Based on the limited geotechnical information available, and assuming that the pipes would be manufactured from Grade X52 steel, a 10 mm wall thickness would be required.

7.2.4 Tie-in to existing Nooitgedagt supply pipeline

A connection is required to the existing 1400 mm diameter Nooitgedagt WTW supply pipeline. This connection needs to be located downstream of the existing cross connection on the Scheepersvlakte pipeline, as well as downstream of the high point in the existing supply line.

The tie-in will comprise a 1400 mm x 1400 mm equal tee that will be cut into the existing pipeline. The branch of the tee will be fitted with an isolation valve to close the Coerney Dam's supply should maintenance be required on this pipeline.

7.2.5 Crossing of the Middle Addo Canal

The proposed pipeline to Coerney Dam will need to cross the Middle Addo canal twice. It is recommended that the pipeline be installed over the canal in order not to impact the integrity or operation of the canal, and to facilitate easier maintenance, if required. The 1400 mm diameter steel pipe will serve as the pipe bridge with concrete supports on either side of the canal. An air valve will have to be installed at the high point created by the canal crossing. The air valve will also serve as an access point into the pipeline for maintenance purposes.

7.2.6 Syphon under the Sundays River

An additional syphon under the Sundays River on the existing Nooitgedagt WTW supply pipeline is recommended to:

- Reduce the risk of supply failure in the event of damage to the existing syphon; and
- Mitigate the risk of the new balancing storage being located on the opposite side of the river, relative to the WTW.

The additional syphon under the Sundays River will be concrete encased. The top of the reinforced pipe encasement should be below riverbed level. The length of the encasement is assumed to be approximately 105 m (the same as the existing pipeline).

An air valve chamber and a scour valve chamber will have to be installed, and tie-ins made into the existing pipeline. The air valve will also serve as an access point into the new pipeline for maintenance purposes. The tie-ins will comprise 1400 mm x 1400 mm equal tees that will be cut into the existing pipeline and installed on the new syphon pipeline. Isolating valves will be provided so that the new syphon can be isolated, as it will only be used if the existing syphon is damaged or when maintenance is required.

7.2.7 Construction cost estimate

A detailed cost estimate of the required conveyance infrastructure is provided in **Appendix E-2**. A summary of the costs for the various components and types of work discussed above is provided in**Table 7.4**.

Table 7.4: Cost estimate summary for the conveyance infrastructure (excluding VAT)

No	Description	Amount (R million)
1	Kirkwood Canal off-take	1.873
2	Inlet/outlet to Coerney Dam	6.831
3	Main connecting pipeline	32.760
4	Tie-in to Nooitgedagt pipeline	1.277
5	Crossing of Middle Addo canal	0.812
6	Syphon under Sundays River	11.205
	Sub-total (a)	**54.759**
7	Preliminary and General Items (50%)	27.379
	Sub-total (b)	**82.138**
8	Contingencies (15%)	12.321
	Total	**94.458**

7.2.8 Operation and maintenance costs

The operation and maintenance costs (**Table 7.5**) of the various conveyance infrastructure components have been included as an annual cost based on a percentage of the construction value.

The components are divided into two sections.

- Civil works, which includes the pipelines and all concrete work at tie-ins and chambers; and
- The mechanical works, which includes valves and gates.

A simple annual cost estimate for the two categories has been determined, based on a percentage of the construction value. The construction value used includes 15% contingencies and excludes the 50% preliminary and general charge items.

Table 7.5: Operation and maintenance costs for the conveyance infrastructure (excluding VAT)

No	Description	Percentage	Construction value (R million)	Annual Amount (R million)
1	Civil works	0.5%	56.809	0.284
2	Mechanical works	4.0%	6.164	0.247
	Total		**62.972**	**0.531**

7.3 Other miscellaneous project costs

7.3.1 Land acquisition

The surface area required for the dam wall and spillway is 4.4 ha. The surface area covered by the basin when at the recommended design flood (RDF) water level is 80.1 ha. This equates to a total area of 84.5 ha. Assuming that it is undeveloped irrigable land, at a cost of R 250 000 per hectare, the cost of land acquisition amounts to R 21 250 000.

The surface area for the 1 860 m long pipeline, including a 15 m wide servitude, is 2.8 ha. Thus, the estimated cost of land acquisition for conveyance infrastructure, at a cost of R 250 000 per hectare, amounts to R 700 000.

7.3.2 Professional fees

An allowance of 10% of the construction cost has been included to account for professional fees for engineering services, site supervision, dam safety requirements, environmental compliance and other specialist requirements.

7.3.3 Cost estimate

Other miscellaneous project costs are summarised in **Table 7.6**.

Table 7.6: Other miscellaneous project costs (excluding VAT)

No	Description	Amount (R million)
1	Land acquisition	21.950
2	Professional fees for dam (10% of construction cost)	29.748
3	Professional fees for conveyance infrastructure (10% of construction cost)	9.446
	Total	**61.144**

7.4 Project cost estimate

The cost estimate for the construction of the balancing dam, the conveyance infrastructure, other miscellaneous costs, professional fees and land acquisition costs have been discussed in the preceding sections. **Table 7.7** shows a summary of the total cost estimate.

Table 7.7: Project cost estimate

No	Description	Amount (R million)
1	Balancing dam	297.481
2	Conveyance infrastructure	94.456
3	Professional fees	39.194
	Value Added Tax (15%)	64.670
4	Land acquisition	21.950
	TOTAL (January 2020 prices)	**517.753**

The project completion cost estimate for commencement of construction in 2023, with escalation of 6.5% from the base year of 2020, is shown in **Table 7.8**.

Table 7.8: Project completion cost estimate (including 15% VAT)

Year	Escalation rate	Notes	Present cost (R million)	Future cost (Base year 2020) (R million)
2023	6.5%	Land acquisition + 40% of Professional fees	39.979	48.293
2024	6.5%	33% Construction value + 20% Professional fees	159.257	204.879
2025	6.5%	33% Construction value + 20% Professional fees	159.257	218.196
2026	6.5%	33% Construction value + 20% Professional fees	159.257	232.379
		TOTAL	**517.751**	**703.747**

8 Legislative Compliance

This chapter describes water use licensing and dam safety legislation and need for compliance, the potential need for the determination of an ecological water requirement and the environmental requirements, and processes that are required to make the recommended project implementation ready.

8.1 Water Use Licensing

This chapter describes the legal requirements and processes that are required to make the recommended project implementation ready in terms of water use licensing and dam safety regulations.

8.1.1 Water use licence

A water use licence will need to be obtained for storing water and affecting and altering the banks of a river (Section 21 (b), (c) and (i), National Water Act, 1998). This licence application is included in the scope of work for the EIA study Chapter 4.8:

> *The proposed dam will require a Water Use Licence Application (WULA) in terms of Section 21(b) of the National Water Act (NWA). As the proposed dam site is also located within minor drainage lines, Section 21 (c) and (i) applications will also be required. The appointed PSP must therefore make provision for an application for a water use licence for the proposed dam. The WULA process and deliverables will comply with GN R267/2017.*

The implementation of the proposed dam and associated conveyance infrastructure will trigger the requirement for a combined water use licence in accordance with Section 21 of the National Water Act (No. 36 of 1998, as amended) (NWA). The volume of water to be stored in the dam exceeds the maximum volume generally authorised under GN 538 (2016 with effect from March 2017) Appendix A. The dam will thus require a Water Use Licence Application in terms of Section 21 (b) of the NWA.

As the dam site is also located within minor drainage lines, Section 21 (c) and (i) applications will also be required. The associated conveyance infrastructure will also cross minor drainage lines (Section 21 (c) and (i)) will therefore be included in the WULA[2]. Water uses that need to be included in the WULA are[3]:

i. Storing water (dam);
ii. Impeding or diverting the flow of water in a watercourse (dam and associated conveyance infrastructure); and
iii. Altering the bed, banks, course or characteristics of a watercourse (dam and associated conveyance infrastructure).

The WULA will be submitted via the DWS online eWULAAS platform. The relevant Water Management Area is *Catchment N - Fish to Tsitsikamma*, which means that the application will be processed by the Port Elizabeth DWS office.

The issuing of a water use licence is based on an evaluation of the proposed activity in terms of the impact on the resource as well as the potential social, economic and environmental impacts of the proposed use. Supporting documentation and studies are required to show the extent to which the proposed water use will impact on the resource, the steps that will be undertaken to mitigate this impact, the extent to which the proposed water use will contribute to the local and national economy, and the social benefits in terms of job creation and income generation in the area. A strong emphasis is given to water use that supports water allocation reform (WAR), the re-dress of previous inequitable allocation of water use licences and the equitable use of the natural resource.

The information required for a Section 21(a), (b), (c) and (i) application is listed in **Table 8.1** below.

Table 8.1: Information requirements for Section 21(a), (b), (c) and (i) in terms of GN R267/2017 Annexure C

Description
Proof of Payment of Licence Application Processing Fee (Compulsory)
Copy of Identity Document of Applicant and Proponent (if applicable) (Compulsory)
Letter of Authority or Power of Attorney to Apply on behalf of Applicant
Letter of Consent if the Applicant is not the Property Owner (Compulsory)

[2] Note that although the conveyance infrastructure in isolation would have qualified for a General Authorisation in terms of GN 509/2016, the need for a full water use licence in terms of Section 21(b) of the NWA means that related Section 21 (c) and (i) water uses are also included in the licence application (Section 3 (c) of GN 509/2016)
[3] Note that the abstraction licence for the greater project/scheme has already been issued i.e. a Section 21 (a).

Description
*Applicant Information Form: Water Service Provider (DW 757 1 770)
*Applicant Information Form: Company, Partnership, Government (DW 7581771)
*Applicant Information Form: Water User Association (DW 759 1 772)
Property Details Form (DW 901)
Property Owner Details (DW 902)
Permission to Occupy (PTO), Title Deed, Lease Agreement, Community Resolution
A description of the location of the activity, including (aa) the 21-digit Surveyor General code of each cadastral land parcel, (bb) where available, the physical address or farm name, (cc) where the required information in sub -regulation (aa) and (bb) is not available, the coordinates of the boundary of the property or properties,
A plan which locates the proposed activity or activities applied for at an appropriate scale, or if it is- (aa) a linear activity, a description and coordinates of the corridor in which the proposed activity or activities is proposed; or (bb) on land where the property has not been defined, the coordinates of the area within which the activity is proposed
*Taking water from a water resource Form (DW 773)
Section 21(a) application
*Pump Technical Data Form (DW 784)
*Canal Technical Data Form (DW 786)
Irrigation Field and Crop Details (DW 787)
*Supplementary Info: Power Generation, Industrial or Mining (DW 788)
*Supplementary info: Domestic, Urban, Commercial or Industrial (DW 789)
Soil Suitability Report (for irrigation from Dept. Agriculture)
Section 21(b) application
*Storing water form (DW 774)
'Dam and Basin Technical Data Form (DW 789)
*Dam Classification Form (DW 793) (for dams >5m and >50 000m^3)
Dam Location Map
Section 21(c) application
* Impeding or diverting the flow of water in a watercourse form (DW 763)
*Altering the bed, banks, course or characteristics of a watercourse (DW 789)
*Supplementary Information for 21 (c) & (i) form (DW775)

The **disposal of inert waste** is unlikely to require a Section 21 (g) application and is therefore currently excluded from the authorisations/licences/permits required.

8.2 Dam Safety

The requirements in terms of dam safety regulations relating to authorisations and licences are as follows:

In terms of Chapter 12 of the National Water Act, "dams with a safety risk" must be registered and classified, and the design and construction monitoring must be carried out under the supervision of an Approved Professional Person (APP).

The following legal requirements apply to new dams, alterations to existing dams or repair of dams that failed, as issued by the Dam Safety Office:

- Application for classification of the dam with the Dam Safety Office (DSO) (part of the Department of Water and Sanitation).

  The proposed dam is expected to be classified as a Category III dam, being of "medium" size with a "high" hazard rating.

- Application to the DSO for approval of an APP and professional team.

  The APP should preferably be registered with the DSO for the design of 20 m high or higher earthfill dams. The professional team members must have adequate experience in their field of expertise to support the APP. The APP will be responsible for the design of the dam.

- Application to the DSO for a Licence to Construct.

  This comprises an application form, design report, engineering drawings and construction specifications. This licence is required before construction of the dam may commence.

- A Water Use Licence or written authorisation must be obtained from the DWS Regional Director Eastern Cape before a Licence to Construct can be issued.

- During construction the APP is responsible for quality control and must submit quarterly reports to the DSO on the construction progress.

- Application for a Licence to Impound from the DSO.

  This involves the compilation and submission of an operation and maintenance manual and emergency preparedness plan. This licence is required before impoundment may commence.

- After completion of all construction work, the APP must submit a completion report, completion drawings and a completion certificate stating that the work has been completed according to specification.

8.3 Ecological Water Requirement

In accordance with the NWA, any new or raised dam is required to make ecological water requirement (EWR) releases to sustain the downstream riparian environment.

The proposed Coerney Dam will be situated in a small ephemeral tributary of the Coerney River, which joins the lower Sundays River near the Nooitgedagt WTW. This tributary has no defined channel or any evidence of flow and has seemingly not seen flows for over 20 years, according to Mr Boetie Muller, who lives on the farm. The dam site however has a catchment area of 34 km^2.

It is therefore uncertain what provision, if any, needs to be made to route natural flows through the dam. DWS Directorate: Resource Directed Measures should determine whether to undertake an EWR determination study for non-perennial systems for this tributary. This could potentially be included as part of the separate EIA study to be undertaken. Determination of an EWR may be a challenge though, seeing that the tributary does not even seem to be ephemeral, but probably only flows (at surface at least) when there is an extraordinary rainfall event.

The proposed balancing dam would be operated close to FSL. If a flood event in this tributary should occur, almost all the flow will pass through the dam to the downstream river valley.

No specific allowance will be made in the design of the balancing dam for EWR requirements.

8.4 Environmental Impact Assessment

8.4.1 Introduction

The DWS is undertaking an Environmental Impact Assessment (EIA) process for the proposed Coerney Dam, in terms of all applicable environmental legislation. The EIA is expected to start in 2021. A detailed scope of services has been prepared by DWS to invite proposals from professional service providers.

In terms of the National Environmental Management Act (No. 107 of 1998, as amended) (NEMA), an Environmental Authorisation for the proposed project will be required from the Department of Environment, Forestry and Fisheries (DEFF). The procedural requirements for the EIA process are set out in GN R983 of 2014 (as amended). Of greatest importance is the multi-staged approach to public participation and stakeholder engagement stipulated by these regulations.

Impact mitigation measures and environmental management are to be set out in an Environmental Management Programme (EMPr) and must address the life-cycle of the project. The EMPr must be compiled in parallel with the EIA process and informed by the EIA process and submitted as part of the final EIA report to the competent authority.

8.4.2 Specialist studies

Various specialist studies will be required, as part of the EIA process, to quantify and assess social and environmental impacts of the proposed project and identify suitable mitigation measures. Standard specialist studies that are envisaged for this project include:

- A terrestrial ecology and botanical study,
- An aquatic ecology and wetland assessment (to be used for both the EIA and WULA process), and
- A Phase 1 heritage impact and paleontological assessment.

8.4.3 Applicable Legislation

The legislation applicable in terms of authorisation, permits and/or licences for the proposed Coerney Dam includes the following:

8.4.3.1 National Environmental Management Act (No. 107 of 1998, as amended)

The proposed project will require a Scoping-EIA process in terms of the National Environmental Management Act (NEMA), as it will trigger *inter alia* activities 15 and 16 of GN R984 (2014, as amended). The proposed project will also trigger various listed activities under GN R983 and R985 (2014, as amended).

8.4.3.2 National Environmental Management: Biodiversity Act (No. 10 of 2004, as amended)

The proposed project area is located within an 'Endangered' ecosystem associated with the AZa6 Albany Alluvial Vegetation type. The proposed dam site is also in a near natural state with notable species diversity. It is therefore anticipated that a permit might be required for the destruction or relocation (restricted activity as per Section 1 of NEM: BA) of plant species, which are protected under the National Environmental Management: Biodiversity Act (NEM: BA).

8.4.3.3 National Forests Act (No. 84 of 1998, as amended)

The proposed dam site is located within an ‘Endangered’ ecosystem, which is in a near natural state. It is therefore anticipated that a permit might be required for the destruction of tree species that are protected under the National Forests Act (NFA).

8.4.3.4 Nature and Environmental Conservation Ordinance (No. 19 of 1974)

The proposed dam site is located within an ‘Endangered’ ecosystem, which is in a near natural state. It is therefore anticipated that a permit might be required for the relocation, damage or destruction of species that are protected under the Nature and Environmental Conservation Ordinance.

8.4.3.5 National Heritage Resources Act (No. 25 of 1999)

The proposed project requires notification of the Eastern Cape Provincial Heritage Resources Agency (ECPHRA), in terms of Section 38 (1)(a) to (c) of the National Heritage Resources Act (NHRA). In the event of a heritage object and/or site being identified during the Phase 1 Heritage and Paleontological Assessment, an application for a permit for destruction or relocation will be required.

8.4.3.6 Competent Authority

Note that in terms of Section 24C (2)(d)(i) of NEMA and Section 43 (1)(c)(i), the national Department of Environment, Forestry and Fisheries (DEFF) will be the competent authority for all listed activities under GN R983 to R985.

8.4.4 Scheepersvlakte Farms 98 EIA

The farm on which the proposed Coerney Balancing Dam will be located, Scheepersvlakte No. 98, has received an Environmental Authorisation dated 5 August 2019 for 852 ha agricultural development, from the Provincial Department of Economic Development, Environmental Affairs and Tourism (reference EC/06/C/LN2/M/47-2018). The Environmental Authorisation includes approval for an irrigation water storage dam of 140 000 m^3 capacity.

Comment was provided by DWS on the Consultative Scoping Report that the Department plans to construct a larger balancing dam on the same site. During consultation with Scheepersvlakte Farms (Pty) Ltd it was agreed that they would not build their storage dam but would abstract water from the proposed Coerney Dam when it is completed. An interim arrangement for supply from the Coerney canal has been agreed.

The proposed storage capacity of the Coerney Dam includes an allowance for the farm dam, namely one week of storage for the Scheepersvlakte farm, equal to 150 000 m^3 of storage. There will be little operational impact on the dam because of the irrigation offtake.

9 Implementation Arrangements

This chapter describes affected land, land acquisition and wayleaves, operation and maintenance requirements, as well as the identification of any institutional arrangements that will ensure that the project is functional and sustainable.

9.1 Affected Land and Land Acquisition

The portion of land upon which the dam is to be located is known as Portion 7 of Scheepersvlakte No. 98, owned by Scheepersvlakte Farms (Pty) Ltd.

The footprint of the proposed Coerney Dam overlaps with portions of the planned future Scheepersvlakte Farms development. The overlaps occur within the full supply level of the dam, the 1:100 year flood line in the basin, as well as portions of the dam wall and the spillway, which is sited on the left abutment. The overlapping areas can be seen on Drawing No. 112546-0000-DRG-CC-002 in **Appendix A-2**. This is also illustrated in **Figure 9.1**. The total area where the future planned orchards and dam infrastructure and 1:100 year flood line overlap is estimated at 36 ha.

The layout of the new citrus developments would need to be revised. Normally the DWS policy is that no development will be allowed within the 1:100 year flood line of the dam, and that development should also be outside the purchase line. In this case, since the unnamed tributary in which the Coerney Dam will be located seems to have not flowed for a very long time (reportedly at least not in the past 20 years), this policy may be relaxed. Development could be allowed between the flood line and the purchase line, but this would need to be negotiated.

The possibility that runoff and return flow from the new citrus orchards could affect the water quality in the dam, would normally require a buffer zone between the dam and the orchards. This aspect needs further investigation to determine whether a buffer zone or drain would be required.

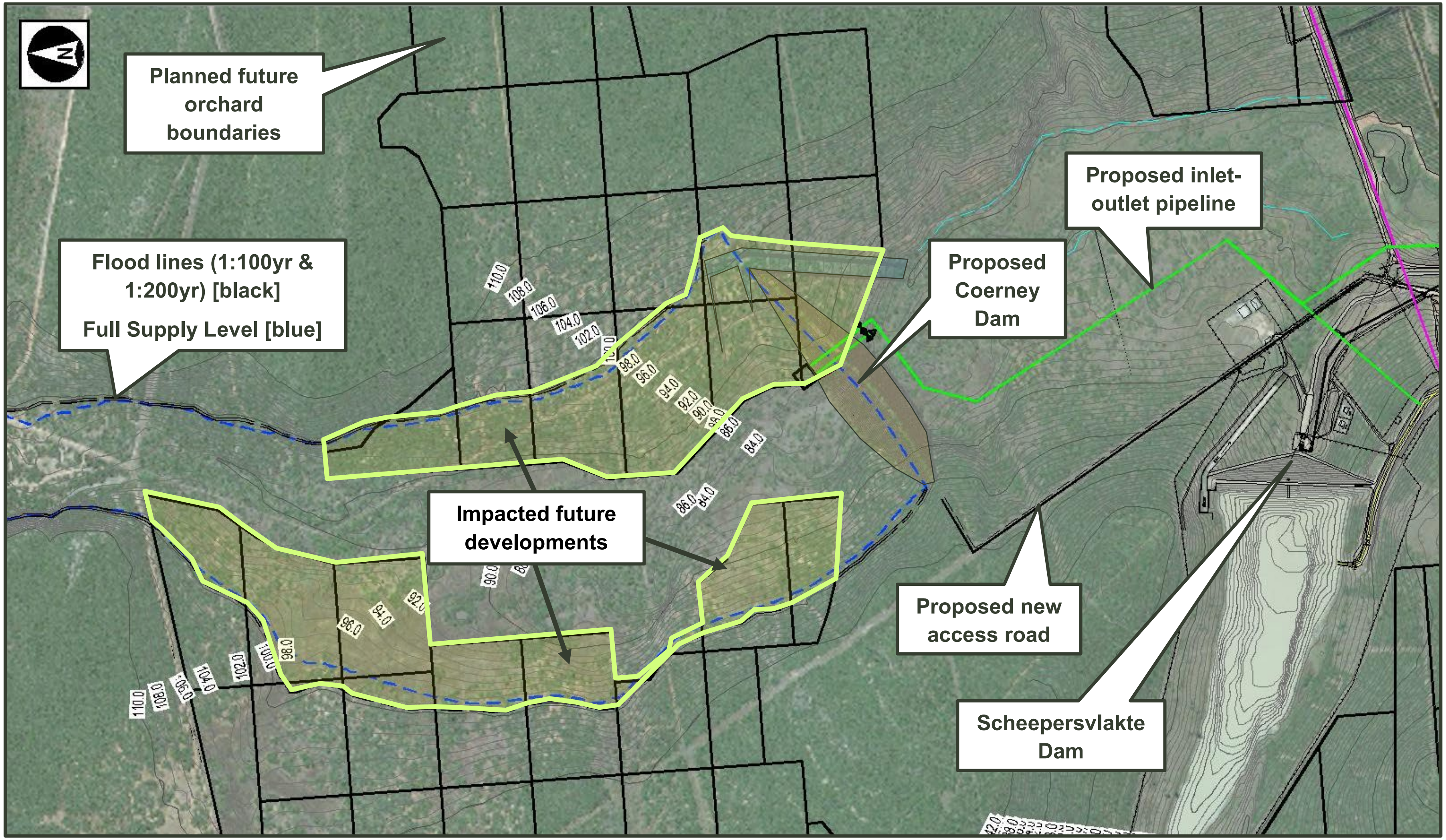

Figure 9.1: **Overlay of planned future farming development and FSL of Coerney Dam**

Scheepersvlakte Farms has obtained an Environmental Authorisation (EA) for expansion of their citrus orchards, as well as for a small storage dam with a backwater footprint, which would overlap with the proposed Coerney Dam embankment. The approved farm dam will have storage of 140 000 m^3, surface area of 7 ha and wall height of 6 m.

The proposed storage capacity of the Coerney Dam includes an allowance for the farm dam, namely one week of storage for Scheepersvlakte Farms, equal to 150 000 m^3 of storage. This water could be abstracted via a floating pump on the Coerney Dam surface, which is the preferred option. Alternatively, an offtake can be provided on the dam's downstream outlet works. The means of abstraction will be confirmed. There will be little operational impact on the dam due to the irrigation offtake.

The route of the proposed new supply pipeline from the long weir to the proposed Coerney Dam does not overlap with any of the proposed future farm developments.

During development of the detailed design route of the conveyance infrastructure the affected land owners should be consulted.

For the purchase of land, the affected areas should be indicated on a map that shows:

- The proposed purchase line;
- Areas (m^2) of different land uses affected;
- Potential borrow areas;
- Affected assets (to be complemented by a list of affected assets); and
- Any land that may no longer be accessible or usable.

The acquisition of land and affected assets is undertaken in terms of the Expropriation Act of 1975 and the Constitution of South Africa. In conjunction with this the new Expropriation Bill has been approved by Cabinet and is expected to come before Parliament later this year (2020).

Servitudes to address issues, such as temporary and permanent access, would need to be prepared during the acquisition process.

9.1.1 Wayleaves

Wayleave applications will be submitted to all the relevant service authorities to (a) obtain information on the location of their existing services, (b) comment on the proposed pipeline alignments, and (c) to obtain their requirements that must be adhered to during construction.

This process should be undertaken during the detailed design phase of the project.

9.2 Operation and maintenance

Once the proposed Coerney Dam is operational and has been filled, the dam will be used as balancing storage for NMBM. This will improve operation and reduce the risk of failure of water supply to NMBM. The Scheepersvlakte Dam will then revert to its original function and will only be used as balancing storage for irrigators. It has been agreed that Coerney Dam will be operated and maintained by LSRWUA as part of the existing scheme.

The proposed balancing dam works will not have any impact on the operation of the current supply pipeline from the Scheepersvlakte Dam to the WTW and supply will still be controlled from the downstream end of the pipeline at the WTW.

The two dams, Scheepersvlakte and Coerney, although filled from the same source, should be operated separately under normal operation. Isolation valves on the existing connections between Scheepersvlakte Dam and the WTW supply pipeline will separate the systems, but enable their linkage in the case of emergencies or maintenance.

The flow rate from the long weir to the WTW/dam will be controlled based on the water requirement from the WTW and the water level in the Coerney Dam. The LSRWUA can reduce the abstraction from the canal to the WTW when Scheepersvlakte Dam needs to be filled. If the supply pipe from the canal to the WTW/dam is closed, the WTW requirement will automatically be met from the Coerney Dam.

The dam should be operated just under its full supply level to leave some buffer storage to balance inflow and outflow. Water will be abstracted from the dam when the requirement cannot be supplied directly from the canal. Water in the dam would thus only be drawn down significantly in an emergency to supply the Nooitgedagt WTW (such as a canal break). This situation can arise during filling of Scheepersvlakte Dam, maintenance or shut down of the canal supply over weekends, and during emergencies.

The dam will be filled, and topped up, over a lengthy filling period through gravity supply. The proposed dam will need to be supplemented with water from time to time to replenish losses resulting from evaporation and infiltration, and minor operational abstractions.

The joint use of water from the dam by NMBM and the Scheepersvlakte 98 Citrus Development Trust irrigation has been agreed by the parties concerned and will need to be formalised. The location of the Scheepersvlakte Farm offtake from the dam needs confirmation, although a floating offtake is preferred.

An earthfill dam does not require much maintenance, but certain components can be designed and/or specified to further reduce maintenance. These interventions during detailed design can also increase the operational life of the dam. These design / specification interventions include the following:

- Properly designed riprap to protect the upstream embankment slope.
- A layer of crushed stone for downstream slope protection instead of grass cover.
- A suitable gravel wearing course on the embankment crest to prevent erosion by rain and forming of ruts by vehicles.
- The use of stainless steel for the pipes under the embankment, from the intake tower to the outlet valve chamber.
- A properly designed gravel access road to ensure adequate drainage.
- Adequate corrosion protection of mechanical items in the intake tower, outlet chamber, etc.

9.3 Institutional Arrangements

The dam will be implemented and owned by DWS. The use of the dam is solely for the NMBM, with the additional storage for use by the Scheepersvlakte Trust.

It has been agreed that the new balancing dam should be operated by the LSRWUA. This arrangement is supported by NMBM. LSRWUA currently operate the supply scheme, and if the proposed dam is included, it will thus enable the operation of the full scheme as a single system, thereby simplifying operation.

10 Further Investigations for Detailed Design

This chapter describes further investigations that are required to successfully undertake the detailed design.

10.1 Topographical Surveys

The accuracy of the contour surveys undertaken is suitable for detailed design of the dam. No further topographical survey of the dam site should be required to complete the detailed design of the dam wall and associated structures.

It is recommended that a centreline survey of the final pipeline routes, which connect the proposed Coerney Dam to the existing scheme, be undertaken prior to construction. This will serve as a final check on the pipeline vertical alignment and soil cover depths.

10.2 Geotechnical Investigations

Further geotechnical investigation of the dam site may be required to provide more information for detailed design, particularly to confirm the choice of spillway location on the left abutment, by comparing this with further investigations into the founding conditions on the right abutment spillway location.

The current level of information should be adequate if a conservative design approach is followed. In other words, if excavation volumes and foundation depths are conservative, the contract price should be adequate for construction of the proposed dam without unnecessary cost overruns.

A geotechnical investigation should be undertaken along the proposed pipeline routes and at the proposed chamber locations to inform the detailed design. The geotechnical investigation should also include a soil resistivity survey and testing for sulphate reducing bacteria to inform the coating selection and the need for cathodic protection.

10.3 Construction Materials

The geotechnical investigation of the dam basin indicates that the material properties are suitable for the construction of a homogeneous embankment. There are also sufficient quantities of material available.

Other materials, such as coarse aggregate for concrete, sand for filters and riprap are not found in the basin or near the site and will have to be imported from commercial sources. Several possible commercial sources for sand and coarse aggregates have been identified, but all are located some distances away from the Coerney Dam site. The closest identified possible commercial sources are in the Uitenhage and Coega areas, which is more than 60 km away from site a list of commercial sources is given in Sub-section 5.3.6.2 of this report. Sources of riprap, concrete aggregates and filter sand should be confirmed during detailed design.

Geotechnical investigations along the pipeline route (as above) should inform an estimate of the volume of suitable pipeline bedding material that is available on site and the volume that will need to be imported from a commercial site. A suitable source of imported bedding material must be confirmed during detailed design.

10.4 Hydraulic Model Study

A hydraulic model study of the spillway configuration (overflow structure, side channel, discharge channel and energy dissipation structure) is required to optimise the detailed design of these components.

10.5 Electrical Power supply

The electrical power requirements of the balancing dam need to be determined, as well as the power source. It has been assumed that a power line can be constructed from Scheepersvlakte Dam.

Power at the proposed Coerney Dam will be required for lighting of the dam wall and valve chambers, operation of valves and hoists, and telemetry. The supply of these requirements from an existing supply point (assumed Scheepersvlakte Dam) should therefore be possible.

10.6 Site-specific flood determination

Site-specific methods should be used to determine the Recommended Design Flood (RDF) and Safety Evaluation Flood (SEF) for the proposed Coerney Dam site.

11 Project Implementation

This chapter provides information on the recommended implementation process, as well as the possible timeframe and milestones dates.

11.1 Implementation process

The recommended steps for the implementation of this project are discussed under the relevant headings below. The various actions are provided in chronological order, although some can be undertaken in parallel.

11.1.1 Declaration as Emergency Works

The implementation of the proposed Coerney Dam is critical to reduce the risk of failure of water supply to NMBM. The current risk to the system is mainly due to the condition of the conveyance infrastructure. There was a failure of the Kirkwood primary canal in 2017 and a failure of the Elandsdrift canal in 2019.

The water supply to NMBM is already under stress due to on-going water shortages from the storage dams to the west of Port Elizabeth. If water supply from Nooitgedagt WTW should be interrupted for more than a week, NMBM would have a crisis as there is no other water supply. It is therefore imperative that the proposed Coerney Dam be implemented as a matter of urgency.

The urgency of the project was discussed at the Study Management Meeting held on 19 February 2020. The committee agreed that the project should be declared as an Emergency Works. This will enable the detailed design to be undertaken by DWS in parallel with the EIA process. A submission was therefore submitted to the Minister of Human Settlements, Water and Sanitation to declare this project an Emergency Works. The Minister approved the submission on 10 July 2020.

11.1.2 Detailed Design of Coerney Dam and associated works

As discussed in the previous sub-section, the project has been declared an Emergency Works. This will enable the detailed design of the project to be undertaken by DWS in parallel with the EIA process.

DWS Engineering Services has the capacity and expertise to successfully undertake the detailed design of the proposed Coerney Dam and associated works. This chief directorate has registered professionals with expertise in the various fields required for detailed design of this project. Their expertise includes dam design, geotechnical engineering, hydrology, hydraulic modelling, pipeline design, structural analysis, engineering drawing, mechanical design, tender documentation and report writing.

11.1.3 Funding Arrangements

The proposed Coerney Dam and associated works will form part of the Lower Sundays River Government Water Scheme. The project will therefore be implemented as a Government Waterworks and funded by National Treasury.

Funding from National Treasury will need to be secured during the detailed design stage. This will enable the project to be implemented as soon as the detailed design and tender documentation are ready, and environmental authorisation has been received. Undertaking these three stages in parallel will substantially reduce the time required for implementation (approximately two years).

11.1.4 Record of Implementation Decisions

The Record of Implementation Decisions (RID) is the official internal DWS document to hand over the project for implementation. The RID describes the components of the project, design aspects, further investigations to be undertaken, institutional and funding arrangements, operational aspects and other pertinent information for implementation of the project.

The RID will be completed as soon as possible and issued to DWS Infrastructure Development to formalise the early implementation of the project as an Emergency Works. An Addendum to the RID will be issued after the Environmental Authorisation has been received.

11.1.5 Environmental Authorisation

DWS is undertaking a comprehensive EIA process in accordance with the National Environment Management Act (No. 107 of 1998, as amended) (NEMA) and the 2014 EIA Regulations (GN R982 – 985, as amended). The EIA process is a legal requirement to obtain

Environmental Authorisation for implementation of the project from the Department of Environment, Forestry and Fisheries.

11.1.6 Water use licences

The water use licence application for storing water and affecting and altering the banks of a river (Section 21 (b), (c) and (i), National Water Act, 1998) is included in the scope of work for the EIA study Chapter 4.8.

11.1.7 Dam safety regulation requirements

Applications for licences for complying with the dam safety regulations will need to be completed before certain tasks may continue.

A licence to construct must be issued by the DSO before any construction may commence. This application includes the relevant application forms, the Detailed Design Report for the dam with engineering drawings, the water use licence, EIA and engineering specifications.

Before the bottom outlets of the dam are closed, thereby commencing the impounding of water, the licence to impound must be obtained from the DSO. This application includes the relevant application forms, the operation and maintenance manual and the emergency preparedness plan.

11.2 Programme and Milestones

The project implementation programme that includes the described tasks and milestones, with rough estimated timeframes has been included in in **Appendix F**.

12 Conclusions

The feasibility-level design of the proposed Coerney Dam has concluded the following:

12.1 Scheme Requirement and Purpose

1) The Nooitgedagt WTW has been designed to cater for peak/back-up supplies from the Nooitgedagt WTW at times when the older infrastructure, from sources to the west of Port Elizabeth, will be requiring maintenance or emergency repairs; in other words, the dam is a balancing dam for emergency water supply to NMBM.
2) The capacity of the existing Scheepersvlakte Balancing Dam is currently too small to adequately supply NMBM and irrigators (769 000 m^3), presenting a risk of shortfall in supply, should there be any interruption to the supply. There is thus a need to investigate the provision of adequate balancing storage for the supply of Orange River Water to NMBM.
3) The Lower Coerney Dam site was identified as the most favourable site for the proposed new balancing dam. The proposed dam is referred to as 'Coerney Dam' in this report and future reports, as there is no Upper Coerney Dam.
4) The main purpose of the proposed new balancing dam, at the Lower Coerney site, is to improve operation and provide balancing storage to reduce the risk of failure of water supply to NMBM.
5) Following the completion of Coerney Dam, the full volume of the existing Scheepersvlakte Dam will be used for irrigation balancing storage, as originally intended. This will provide operational flexibility for the canal system and alleviate the time pressures for canal maintenance.

12.2 Overview of the Scheme

1) The proposed Coerney Dam will be situated in a valley just to the east of the existing Scheepersvlakte Dam. The proposed new dam is a 4.69 million m^3 earthfill embankment dam, with a side channel spillway and intake tower. The dam will be filled from the Kirkwood primary canal, by abstracting water from a new offtake on the canal, between the long weir and syphon intake. A new 1400 mm diameter steel pipeline will

supply the dam with water from the offtake under gravity. This proposed new pipeline will be connected to the existing pipeline, which supplies water to the Nooitgedagt WTW from the Scheepersvlakte Dam. The connecting pipework will bypass an existing high point in the supply pipeline. This arrangement will allow the gravity supply of water to the WTW by a combination of the proposed new intake at the long weir and supply from the Coerney Dam. At the dam the supply pipeline will bifurcate into an inlet and outlet branch through the embankment, ending in an intake tower with two offtake levels.

2) In addition to the alterations around the existing dam, an additional syphon on the supply pipeline to the WTW under the Sundays River is planned, as well as some changes at the inlet structure at the WTW to reduce energy losses and improve the maximum flow.

12.3 Topographical Survey

1) A topographical survey was completed by Survey Services: Southern Operations (National Water Resource Infrastructure) of DWS for the Coerney dam site, inclusive of relevant existing infrastructure near the Scheepersvlakte Dam.
2) The available survey data is considered sufficient to undertake the preliminary and detailed designs of the proposed infrastructure.

12.4 Geotechnical Investigations

The findings of the ground investigations conducted at the Coerney Dam site is as follows:

1) **General geology**: Underlain by strata of the Sundays River Formation, Uitenhage Group, comprising thin grey sandstones, siltstones and mudrocks.
2) **Geological profile, dam footprint**:

   Left flank: (upper), soils to 7.2 m (including horizon of gravelly soils 4 m to 7,2 m); very soft rock mudstone, subordinate sandstone from 7.2 m.

   Central section (conduit – intake and outlet):

   Intake: sandy soil to 2.65 m; gravelly soils to 7.7 m; soft to very soft rock (occasionally to clay) mudstone from 7.7 m; medium hard to hard rock interbedded mudstone / sandstone from 9.8 m.

   Outlet: sandy soil to 1.3 m; gravel-sand horizon to 4 m; very soft to soft rock sandstone from 4 m; soft to medium hard rock sandstone interbedded mudstone from 4.6 m; hard rock sandstone from 12 m.

Central section; sandy soils to 2 m; gravelly horizon to 3.25 m; soft to very soft rock sandstone from 3.25 m; medium hard rock sandstone from 5.5 m; hard rock sandstone from 7.5 m; mudstones more prominent from 11 m.

Right flank: topsoil to 0.8 m; gravelly horizon to 2.7 m; highly weathered, medium hard to soft rock from 2.7 m. Interbedded sandstones, mudstones. The *upper right flank* comprises upper soils to 3.3 m and 4.2 m where bedrock is encountered.

3) **Founding considerations**: A gravelly horizon (1.2 m to 5 m thick) is recognised which occurs across the footprint; considered to represent reworked terrace gravels. Note however that the horizon is variable. Mostly, the matrix was not recovered in the boreholes, but this stratum represents a potential preferred seepage path (a buried channel). The cut-off design should consider this feature.
4) **Excavation depths**: For the *cut-off*, on the extreme / uppermost left flank, the principle of excavating to the base of alluvial gravels implies a depth up to 7.2 m, maybe some relaxation can be allowed on extreme upper flank. In the central section assume minimum depth of 5.5 m but note some variability. On the mid-right flank consider a minimum depth of 3.5 m (below gravel layer).
5) **Foundation treatment**: Mudrocks are susceptible to slaking; provision must be made for immediate protection after exposure.

   As above re presence of a potential 'buried channel'; must ensure cut-off intersects this stratum.

   Permeability of rock mass is generally very low / tight, but instances of wash-out of softer strata are recorded. The 'groutability' of these weathered rocks is however uncertain.

   At face value the outlet conduit could likely be founded on the gravel-sand stratum, but this does not consider required founding levels.
6) **Spillway, geological profile**: Upper spillway (near ogee / sill); soils to 4 m; gravelly soil horizon to 7.2 m; very soft / soft rock (mainly mudstone, subordinate sandstone) from 7.2 m.

   Lower spillway (actually midway); soils to 5.45 m; gravelly soils to 6.7 m; very soft rock sandstone (sand in places) from 6.7 m; interbedded sandstone / mudstone from 8 m.

   End of spillway: bedrock encountered as slightly weathered hard rock sandstone is encountered between 3.4 m and 4.9 m.
7) **Spillway considerations** Soils underlain by weak bedrock that would be susceptible to erosion. Assume full concrete lining is required. The appropriate energy dissipation must be incorporated at the end of the spillway lining, and measures must be incorporated to prevent undercutting of the concrete. The end of spillway should

then be founded on the bedrock which should be encountered beyond 2.9 m depth, with all the upper horizons removed prior to placement of concrete

8) **Reservoir slopes**: Natural slopes are essentially flat / gently sloping; no slope stability issues foreseen.
9) **Construction materials**: No clear distinction can be made between the various materials types within the basin in terms of their suitability for either impervious core material, or for semi-pervious shell material. Clear delineation into different borrow areas for the respective material uses therefore cannot sensibly be made.
   However, these materials do exhibit almost total compliance with specifications for use in a homogeneous earthfill embankment, and it is therefore recommended that the Coerney Dam be constructed as a homogeneous earthfill embankment rather than a zoned embankment.
   Other materials like coarse aggregate for concrete and filter sands / fine aggregate will have to be imported.

12.5 Coerney Dam Design

1) The dam is located upstream of the Coerney Siphon outlet, in a valley east of and adjacent to the existing Scheepersvlakte Dam.
2) The main advantage of this site is that the dam would be operated under gravity.
3) The dam will be filled from the Kirkwood primary canal via a new pipeline. The dam will supply the Nooitgedagt WTW via a new connecting pipeline to the existing 1 400 mm Nooitgedagt WTW pipeline.
4) The proposed dam is a homogeneous earthfill embankment dam.
5) There is some zoning of the embankment for slope protection; rip-rap on the upstream face and crushed stone on the downstream face, as well as internal filter drains.
6) The dam has a crest width of 5.0 m, an upstream slope of 1V:3H and a downstream slope of 1V:2H.
7) The lowest level at the valley bottom is 81.5 masl which, with a NOC level of 102.0 masl, results in a maximum wall height of 20.5 m.
8) The full supply level is 98.2 masl, which gives a maximum water depth of 16.2 m and a storage capacity of 4.69 million m^3.
9) The wall height, together with the expected High hazard rating, results in a Category III dam safety classification.
10) The geotechnical investigations have shown that the material found in the basin does not have enough differentiation for selection of core and general fill zones. For this

reason, a homogeneous embankment design, which makes use of a semi-pervious to impervious material for the embankment fill, has been proposed.

11) Sand, coarse aggregate and rock are not available on or near the site for the filter zones, embankment protection and concrete production, and will need to be imported.
12) The geotechnical investigations at the dam site have identified that the core trench excavation should extend past a potential seepage path layer consisting of sand and gravel. The depth of this layer ranges from 7 m to 8 m on the left abutment, to 4 m in the river section and from 3 m to 5 m deep on the right abutment.
13) The spillway should be founded on the left abutment. Although it was found to have deep foundations to suitable bedrock, the spillway on the left does not cross the access road or supply pipeline.
14) A side channel spillway was found to be the most favourable option for the site conditions.
15) The side channel spillway has an ogee shaped overflow crest with a length of 50 m, a trapezoidal cross-section with base width of 20 m, a depth of 6.35 m, and side slopes of 1V:0.5H.
16) The side channel flows into a trapezoidal discharge channel with a base width of 20 m and slopes of 1V:1H, lined with reinforced concrete to the depth of the safety evaluation flood (SEF) flow of 1.7 m.
17) The spillway terminates in a stilling basin at the foot of the abutment slope.
18) It should be noted that the various spillway dimensions, especially that of the side channel, still need to be optimised to balance hydraulic, cost and constructability efficiencies.
19) The dam, being classified as Category III, should have a recommended design flood (RDF) equal to the 1:200 year flood. This has an incoming flow peak of 143 m^3/s, which is attenuated to 110 m^3/s, after routing through the basin.
20) The SEF is equal to the probable maximum flood with a peak inflow of 835 m^3/s, which is attenuated to 753 m^3/s.
21) The required freeboard was determined for a Category III embankment dam, using the attenuated flood levels, and was found to be 3.64 m. The freeboard provided is 3.8 m.
22) The dam will be connected to a new offtake on the existing Kirkwood primary canal and the Nooitgedagt WTW pipeline via a 1400 mm diameter supply pipeline.
23) At the outlet chamber, which is at the toe of the dam, the supply pipeline will bifurcate into an inlet and outlet branch, and then reduce to 1200 mm diameter and then to 1000 mm diameter before entering the dam. These two pipes through the embankment will be of stainless steel and will be encased in reinforced concrete.

24) The dam will have a wet well outlet tower in the basin.
25) The outlet tower is accessible from the embankment crest via a pedestrian walkway.
26) The outlet tower will have two inlet levels, one at 86.0 masl and another at 92.0 masl.
27) The inlets can be isolated with vertical sluice gates operated from the top of the tower.
28) The dam will be accessed via a road extending from the downstream end of the Scheepersvlakte Dam.
29) The river diversion strategy for the construction of Coerney Dam should make provision for a flood between the 1:5 year and the 1:20 year flood. It is expected that no regular river flows will need to be diverted during construction.
30) The legislative requirements for the implementation of the dam with regards to the environmental authorisation, water use licence and ecological water requirement are discussed in the Implementation Support Report.
31) The dam safety regulation requirements and licence requirements are briefly discussed in this sub-report.

12.6 Conveyance Infrastructure Design

1) The proposed conveyance infrastructure comprises of two gravity pipelines.
2) The main advantages of the proposed scheme are that the proposed Coerney Dam would increase the raw water storage capacity of NMBM and the high point in the existing Nooitgedagt WTW gravity main would be bypassed.
3) The hydraulic calculations of both pipelines are based on a design capacity of 280 Mℓ/d (3.24 m^3/s) and Coerney Dam water levels at MOL of 86 masl and a FSL of 98.2 masl.
4) A storage capacity of only 17% would be required for a DN 1400 pipeline to deliver the design flow rate of 3.24 m^3/s. A flow of 106.6 Mℓ/d can be discharged through a DN 1400 pipeline with the dam level at MOL, i.e. almost 40% of the maximum flow rate.
5) Based on the hydraulic gradient lines it would be possible to discharge 280 Mℓ/d from the Kirkwood Canal to the Coerney Dam.
6) It is recommended that steel pipes be utilised as the preferred pipe material for the proposed pipelines.
7) Based on the preliminary wall thickness calculations, the proposed pipelines will be DN 1400, Grade X52 steel with a yield strength of 358 MPa and a recommended wall thickness of 10 mm.
8) The proposed dam will be supplied from the Kirkwood primary canal through a DN 1400 pipeline, which will also be used to transfer water to the tie-in point on the existing Nooitgedagt WTW pipeline.

9) The offtake from the Kirkwood primary canal will be located downstream of the Coerney syphon intake, and upstream of the long weir. It is proposed that the new offtake comprises an adjustable weir that would allow regulating of the flow that could be discharged from the canal to the WTW or the Coerney Dam.
10) A connection will be made into the existing Nooitgedagt WTW supply pipeline downstream of the cross connection with the Scheepersvlakte syphon, and downstream of the high point in the existing supply pipeline.
11) The Middle Addo canal will have to be crossed at two locations by means of a pipe bridge.
12) The proposed Coerney Dam spillway will need to be crossed by the DN 1400 pipeline, if the spillway is constructed on the right abutment of the dam. There will be no impact on the pipeline if the spillway is constructed on the left abutment as recommended in this report.
13) An additional syphon under the Sundays River on the existing Nooitgedagt WTW supply pipeline is recommended. The purpose is to reduce the risk of supply failure and to mitigate the risk of the new balancing storage being located on the opposite side of the river, relative to the WTW.
14) It is recommended that the new syphon be located upstream and separate from the existing syphon. Apart from doubling the syphon it is also recommended that an adequate stockpile of replacement pipes be kept on site, to enable quick repair of the pipeline in case of failure.

12.7 Legislative Compliance

12.7.1 Water Use and Dam Safety Licences

1) A water use licence application (WULA) will need to be submitted for storing water and affecting and altering the banks of a river (Section 21 (b), (c) and (i), National Water Act, 1998). This licence application is included in the scope of work for the EIA study (Chapter 4.8).
2) The WULA will be submitted via the DWS online eWULAAS platform to the relevant Water Management Area office for *Catchment N - Fish to Tsitsikamma*, which is the Port Elizabeth DWS office.
3) The proposed dam is a “dam with a safety risk” and is expected to be classified as a Category III dam, being of “medium” size with a “high” hazard rating. The dam safety regulations are thus applicable to the design and construction of the dam.

12.7.2 Ecological Water Requirement (EWR)

4) There has been no recorded flow in the tributary on which the proposed dam site is located for more than 20 years and there is no defined river bed. No specific allowance will be made in the design of the balancing dam for EWR requirements.
5) DWS Directorate: Resource Directed Measures should determine whether it is necessary to undertake an EWR determination study for non-perennial systems for this Coerney River tributary in which the proposed dam would be located. If required, the EWR determination could be included in the Environmental Impact Assessment.

12.7.3 Environmental Impact Assessment

6) The DWS will undertake an Environmental Impact Assessment (EIA) process for the proposed Coerney Dam, in terms of all applicable environmental legislation. The EIA is expected to start in 2021.
7) Impact mitigation measures and environmental management are to be described in an Environmental Management Programme (EMPr) and must address the life-cycle of the project. This will be compiled and submitted as part of the EIA process.
8) Specialist studies will be required as part of the EIA.
9) Applicable legislation to consider in the EIA process in terms of the National Environmental Management Act, National Environmental Management: Biodiversity Act, National Forests Act, Nature and Environmental Conservation Ordinance, National Heritage Resources Act, and National Water Act.

12.8 Implementation Arrangements

12.8.1 Affected Land and Land Acquisition

1) The portion of private land on which the dam is to be located is known as Portion 7 of Scheepersvlakte No. 98, owned by Scheepersvlakte Farms (Pty) Ltd. The State would need to acquire this land. The footprint of the proposed Coerney Dam and the 1:100 year flood line overlaps with approximately 36 ha of the planned future Scheepersvlakte Farms irrigation development on this property. Given the extreme ephemeral nature of the tributary river in which the dam will be located, the land owner may be interested in negotiating to farm some land within the 1:100 year flood line.
2) The joint use of water from the dam by NMBM and the private developer has been agreed by the parties concerned.
3) There is a possibility that runoff and return flow from the new citrus orchards into the balancing dam could affect the water quality in the dam, potentially ascribed to the use

of pesticides and fertiliser (even though organic farming is planned). This would require a buffer zone between the dam and the orchards, and/or water quality mitigating measures to be established and implemented.

12.8.2 Scheme Operation and Maintenance

4) Once Coerney Dam is operational and has been filled, the dam will be used as balancing storage for NMBM.
5) An operational agreement with the land owner will need to be drafted, for sharing of the impoundment and use of the additional volume that has been included in the dam volume for irrigation by the land owner.
6) The Scheepersvlakte Dam will then revert to its original function and will only be used as balancing storage for irrigators.
7) The dam should be operated just under its full supply level to leave some buffer storage to balance inflow and outflow. Water will be abstracted from the dam when the requirement cannot be supplied directly from the canal. Water in the dam would thus only be drawn down significantly in an emergency to supply the Nooitgedagt WTW. The dam will be filled over a lengthy filling period.
8) The bifurcation and valve arrangement will allow shutoff of inflow to the dam when it is full, but automatic augmentation of supply to the WTW when the supply from the offtake on the canal is insufficient to meet the requirement.
9) The proposed balancing dam works will not have any impact on the operation of the current supply pipeline from the Scheepersvlakte Dam to the WTW and supply will still be controlled from the downstream end of the pipeline at the WTW.

12.8.3 Institutional Arrangements

10) It has been agreed by the parties concerned, including NMBM, that Coerney Dam will be operated and maintained by LSRWUA as part of the existing LSRGWS. LSRWUA currently operate the supply scheme, and if the proposed dam is included, it will thus enable the operation of the full scheme as a single system, thereby simplifying operation.

12.9 Further Investigations for Detailed Design

12.9.1 Topographical Survey

1) The accuracy of the contour survey undertaken for the feasibility study is suitable for detailed design of the dam.

2) It is recommended that a centreline survey of the final pipeline routes, which connect the proposed Coerney Dam to the existing scheme, be undertaken prior to construction.

12.9.2 Geotechnical and Materials Investigations

3) Further geotechnical investigation of the dam site may be required to provide more information for detailed design. The current level of information should however be adequate if a conservative design approach is followed.
4) A geotechnical investigation should be undertaken along the proposed pipeline routes and at the proposed chamber locations to inform the detailed design. The geotechnical investigation should also include a soil resistivity survey and testing for sulphate reducing bacteria to inform the pipeline coating selection and the need for cathodic protection of the pipelines.
5) Materials such as coarse aggregate for concrete, sand for filters and riprap are not found in the basin or near the site and will have to be imported from commercial sources.
6) Geotechnical investigations along the pipeline route should be done to inform an estimate of the volume of suitable pipeline bedding material available.

12.9.3 Hydraulic Modelling

7) A hydraulic model study of the spillway configuration is required to optimise the detailed design.

12.9.4 Electrical Supply

8) The electrical power requirements of the balancing dam need to be determined, as well as the power source. It has been assumed that a power line can be constructed from Scheepersvlakte Dam.

12.9.5 Design Floods

9) Site-specific methods should be used to determine the magnitude of the design flood and safety evaluation flood for the proposed Coerney Dam site.

12.10 Project Implementation

1) The proposed Coerney Dam and associated works will form part of the LSRGWS. The project will therefore be implemented as a Government Waterworks and should be funded by National Treasury.

2) The use of the dam is solely for the NMBM, with the additional storage for use by the Scheepersvlakte Trust.
3) Coerney Dam should be operated by the LSRWUA.
4) A recommended project implementation programme has been compiled, indicating the described tasks and milestones, with rough estimated timeframes.
5) The Record of Implementation Decisions (RID) is the official internal DWS document to hand over the project for implementation. The RID describes the components of the project, design aspects, further investigations to be undertaken, institutional and funding arrangements, operational aspects and other pertinent information for implementation of the project.

13 Recommendations

The following recommendations are applicable to the detailed design and construction phases of the project:

13.1 Topographical Survey

1) A ground centreline survey should be done along the final chosen pipeline centreline route, prior to construction commencing. This will serve as a final check on the pipeline vertical alignment and soil cover depths.

13.2 Geotechnical Investigations

1) Involvement of a geotechnical specialist during construction is essential. Activities would include regular inspection of all excavated faces and cut slopes from a stability point of view, oversight of any further geotechnical exploration and quality assurance testing, confirmation of bedrock depth at the spillway, engineering geological mapping of the cut-off trench and recording of the as-built details, etc.

2) A geotechnical investigation should be undertaken along the proposed pipeline routes and at the proposed chamber locations to inform the detailed design. The geotechnical investigation should also include a soil resistivity survey and testing for sulphate reducing bacteria to inform the coating selection and the need for cathodic protection.

13.3 Coerney Dam Design

A number of design inputs and elements should be refined in the detailed design phase.

1) The assumptions made in the determination of the desired dam storage volume (e.g. siltation both from the canal and catchment, “normal use” volume and resulting siltation, infiltration losses) should be checked and refined.

2) Filter sand, aggregate and rock sources, other than what has been identified thus far, could be investigated and identified. There was no investigation into site specific borrow pits outside of the dam basin.

3) The embankment zoning and dimensions are based on typical values for embankment dams of this size using similar materials. The zoning dimensions must thus be designed based on the actual material properties and design constraints for the particular zones. These include elements such as: filter zone thicknesses and spacing depending on the target filter sand and permeability, the core trench bottom width depending on the permeability of the target fill material as well as the permissible seepage losses (to be clarified), and the embankment slopes and slope stability.
4) Additional consideration and investigation into the possibility of providing an auxiliary and service spillway arrangement should be done. The service spillway would contain the Recommended Design Flood and an auxiliary spillway, which can be unlined, would accommodate the Safety Evaluation Flood.
5) Determine a site-specific RDF and SEF for detailed design of the dam and spillway.
6) Undertake a hydraulic model study of the spillway configuration.

13.4 Conveyance Infrastructure Design

1) An estimate is required of the volume of suitable pipeline bedding material that will need to be imported, as well as locating suitable sources.
2) The wall thickness calculations of the proposed pipelines should be confirmed once the additional geotechnical information becomes available.
3) During the detailed design, the pipeline route will need to be confirmed after discussions with affected land owners and authorities. Some refinements to the routes may be required due to developments subsequent to the feasibility design.
4) A decision is required on the preferred lining system to be used (i.e. cement mortar or epoxy) as well as the coating system (i.e. polyurethane, medium density polyethylene (i.e. Sintakote), 3LPE and polymer modified bitumen).
5) The exact positions of all connections and impacted associated infrastructure must be verified during the detailed design of the pipelines.
6) Independent quality control inspections of the pipes, at the factory and on site, must be included in the construction tender documents.
7) As-built drawings and/or information of the existing syphon under the Sundays River will need to be obtained during the detailed design phase of the project.

8) The position of the new syphon, upstream of the existing syphon, needs to be determined.

13.5 Legislative Compliance

1) The water use licence application for storing water, and affecting and altering the banks of a river (Section 21 (b), (c) and (i), National Water Act, 1998) is included in the scope of work for the EIA study (Chapter 4.8).
2) Dam safety regulation requirements:
   - Applications for licences for complying with the dam safety regulations will need to be completed before certain tasks may continue.
   - A licence to construct must be issued by the DSO before any construction may commence. This application includes the relevant application forms, the Detailed Design Report for the dam with engineering drawings, the water use licence, EIA and engineering specifications.
   - Before the bottom outlets of the dam are closed, thereby commencing the impounding of water, the licence to impound must be obtained from the DSO. This application includes the relevant application forms, the operation and maintenance manual and the emergency preparedness plan.
3) The DWS should consider the merit of undertaking an EWR determination study for non-perennial systems for the small ephemeral tributary of the Coerney River, in which the Coerney Dam will be situated.
4) DWS should undertake a comprehensive EIA process in accordance with the National Environment Management Act (No. 107 of 1998, as amended) (NEMA) and the 2014 EIA Regulations (GN R982 – 985, as amended). The EIA process is a legal requirement to obtain Environmental Authorisation, for implementation of the project, from the Department of Environment, Forestry and Fisheries.

13.6 Implementation Arrangements

1) The irrigation development on Scheepersvlakte Farm and the implementation of the dam need careful planning and coordination. An agreement for the joint use of water will need to be agreed. The location of the Scheepersvlakte Farm offtake from the dam should be confirmed.

2) Further investigations for detailed design should be undertaken, inclusive of topographical surveys, geotechnical investigations, construction materials, hydraulic model study, electrical power supply and site-specific flood determination.

3) The revision of contractual arrangements with LSRWUA should be considered.

4) The project has been declared as an Emergency Works by the Minister of Human Settlements, Water and Sanitation on 10 July 2020. This will enable the detailed design to be undertaken by DWS, in parallel with the EIA process.

5) The RID needs to be completed as soon as possible and issued to DWS Infrastructure Development to formalise the early implementation of the project as an Emergency Works.

6) Funding needs to be secured from National Treasury to enable construction of the project to commence as soon as the detail design is complete and Environmental Authorisation has been received.

7) An Addendum to the RID needs to be issued after the Environmental Authorisation has been received.

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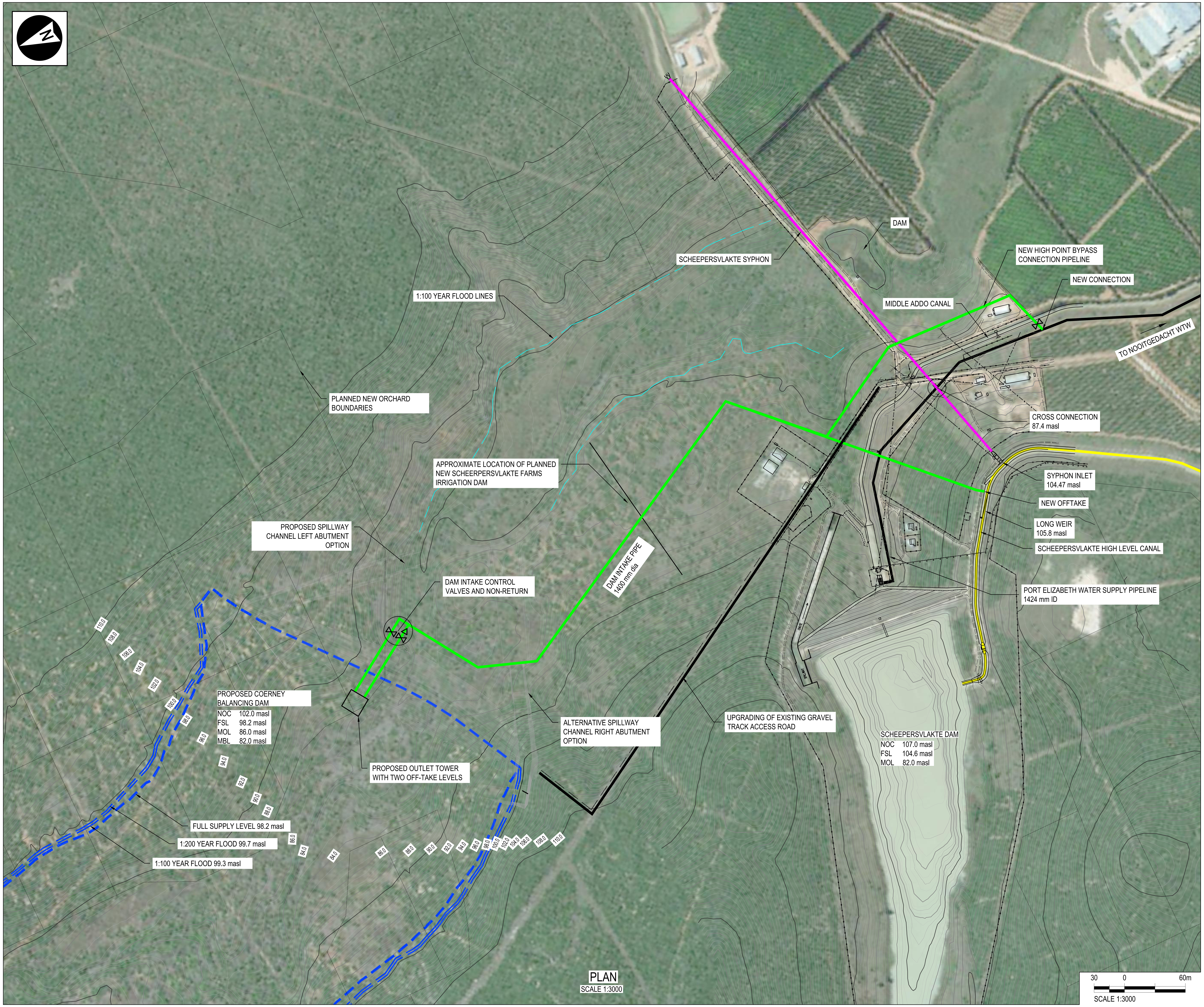
Appendix A: Layout and Locality Plans



REFERENCE DRAWINGS

DRAWING No.	TITLE / DESCRIPTION

LEGEND

- FUTURE ORCHARD BOUNDARIES
- PE WATER SUPPLY PIPELINE
- MIDDLE ADDO CANAL
- SCHEEPERSVLAKTE SYPHON
- SCHEEPERSVLAKTE HIGH LEVEL CANAL
- PROPOSED PIPELINE

NOTES

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CLIENT

REV	DATE	REVISION DETAILS	APPROVED
A	08/2019	FOR INFORMATION	E. V. D. BERG
B	02/2020	FOR INFORMATION	E. V. D. BERG
C	02/2020	FOR INFORMATION	E. V. D. BERG
D	03/2020	FOR INFORMATION	E. V. D. BERG

SCALE	SIZE	
AS SHOWN	A1	FOR INFORMATION

DRAWN: P. NDLOVU
DESIGNED: O. HUMAN
CHECKED: M. SHAND
APPROVED
DATE

PROJECT
SUPPORT OF THE WATER RECONCILIATION STRATEGY OF THE ALGOA WATER SUPPLY SYSTEM

TITLE
COERNEY BALANCING DAM LAYOUT PLAN

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
112546	0000	DRG	CC	001	D

PLAN
SCALE 1:5000

REFERENCE DRAWINGS

DRAWING No.	TITLE / DESCRIPTION

LEGEND

- FUTURE ORCHARD BOUNDARIES
- PE WATER SUPPLY PIPELINE
- MIDDLE ADDO CANAL
- SCHEEPERSVLAKTE SYPHON
- SCHEEPERSVLAKTE HIGH LEVEL CANAL
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NOTES

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CLIENT

REV	DATE	REVISION DETAILS	APPROVED
A	08/2019	FOR INFORMATION	

SCALE	SIZE
AS SHOWN	A1

FOR INFORMATION

DRAWN: P. NDLOVU

DESIGNED: O. HUMAN

CHECKED:

APPROVED

DATE

PROJECT

SUPPORT OF THE WATER RECONCILIATION STRATEGY OF THE ALGOA WATER SUPPLY SYSTEM

TITLE

LOCALITY PLAN

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
112546 -	0000 -	DRG -	CC -	002 -	A

Appendix B: Survey Maps

CO-ORDINATE SYSTEM LO 25° HARTEBEESTHOEK

R.L. FROM T275 91.70 METERS ABOVE M.S.L.

STASIE	Y	X	H.H.	STATION	Y	X	R.L.
CH150	−57 989.90	+3 703 396.17	106.960				
SCP1	−58 074.56	+3 703 429.75	87.100				
SCP2	−58 173.21	+3 703 676.11	106.516				
SCP3	−58 363.70	+3 703 741.78	84.057				
SCP4	−58 374.54	+3 703 595.35	83.703				
SCP5	−58 784.19	+3 703 431.45	87.905				

SHEET LAYOUT

1

0 15 30 60 90 120 150 180 210 m

PLAN SCALE 1:1500

SOUTHERN OPERATIONS (NWRI)
PO BOX 5501
WALMER
6065

SURVEYED: SOUTHERN OPERATIONS (NWRI)	08/18	TRACED: SOUTHERN OPERATIONS (NWRI)	08/18
COMPILED: SOUTHERN OPERATIONS (NWRI)	08/18	TOPOGRAPHY CHECKED:	
PHOTOGRAPHY:		CADASTRAL CHECKED:	

No.	REVISION	DATE

REPUBLIC OF SOUTH AFRICA
DEPARTMENT WATER AND SANITATION

LOWER SUNDAYS RIVER GWS

DETAIL SURVEY OF AREA BELOW SCHEEPERSVLAKTE DAM

DISTRICT PATENSIE	SHEET 1 OF 1	LOC. NO. N400-10	REG. NO. 177014/18
AREA Ha	CODE		

CO-ORDINATE SYSTEM LO 25° HARTEBEESTHOEK

R.L. FROM T275 91.70 METERS ABOVE M.S.L.

STASIE	Y	X	H.H.	STATION	Y	X	R.L.
◉CH150	-57 989.90	+3 703 396.17	106.960				
◉SCP1	-58 074.56	+3 703 429.75	187.100				
◉SCP2	-58 173.21	+3 703 676.11	106.516				
◉SCP3	-58 363.70	+3 703 741.78	84.057				
◉SCP4	-58 374.54	+3 703 595.35	83.703				
◉SCP5	-58 784.19	+3 703 431.45	87.905				

SHEET LAYOUT

2
1

0 50 100 200 300 400 500 600 700 m

PLAN SCALE 1:5000

SOUTHERN OPERATIONS (NWRI)

PO BOX 5501
WALMER
6065

SURVEYED: SOUTHERN OPERATIONS (NWRI)	08/18	TRACED: SOUTHERN OPERATIONS (NWRI)	08/18
COMPILED: SOUTHERN OPERATIONS (NWRI)	08/18	TOPOGRAPHY CHECKED:	
PHOTOGRAPHY:		CADASTRAL CHECKED:	

No.	REVISION	DATE

REPUBLIC OF SOUTH AFRICA

DEPARTMENT WATER AND SANITATION

LOWER SUNDAYS RIVER GWS

DETAIL SURVEY

DISTRICT PATENSIE	SHEET 1 OF 2	LOC. NO. N400-10	REG. NO. 177015/18
AREA Ha	CODE		

CO-ORDINATE SYSTEM LO 25° HARTEBEESTHOEK

R.L. FROM T275 91.70 METERS ABOVE M.S.L.

STASIE	Y	X	H.H.	STATION	Y	X	R.L.

SHEET LAYOUT

2

1

0 50 100 200 300 400 500 600 700 m

PLAN SCALE 1:5000

SOUTHERN OPERATIONS (NWRI)
PO BOX 5501
WALMER
6065

SURVEYED: SOUTHERN OPERATIONS (NWRI)	08/18	TRACED: SOUTHERN OPERATIONS (NWRI)	08/18
COMPILED: SOUTHERN OPERATIONS (NWRI)	08/18	TOPOGRAPHY CHECKED:	
PHOTOGRAPHY:		CADASTRAL CHECKED:	

No.	REVISION	DATE

REPUBLIC OF SOUTH AFRICA

DEPARTMENT WATER AND SANITATION

LOWER SUNDAYS RIVER GWS

DETAIL SURVEY

DISTRICT	PATENSIE	SHEET 2 OF 2	LOC. NO.	REG. NO.
AREA	Ha	CODE	N400-10	177016/18

Appendix C: Dam Design Plans, Drawings and Calculations

Coerney Dam

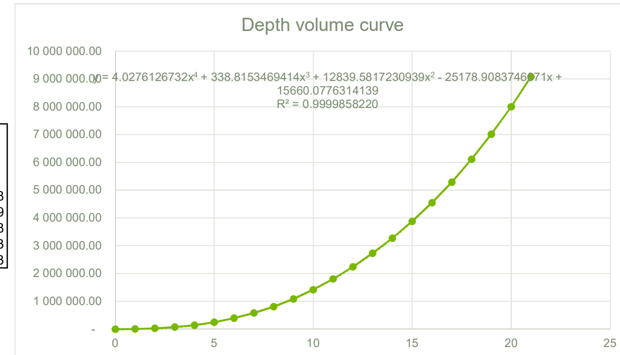
LEVEL POOL ROUTING

PROJECT No:	112546	Input Data	
Title:	Basin curves	Calculated	
Calculated by:	O Human	**Date:**	04 March 2020

Elevation	Depth	Area	Incremental volume	**Cumulative volume**	Incremental volume	Cumulative volume	Cumulative volume (formula)		Area (curve)
82	0	2 119.72	N/A	-	N/A	-	15 660.08	#DIV/0!	3 095.98
83	1	15 119.61	8 619.66	**8 619.66**	7 633.51	7 633.51	3 663.59	-57%	13 330.52
84	2	34 991.88	25 055.75	**33 675.41**	24 370.96	32 004.47	19 435.55	-42%	31 803.71
85	3	54 707.50	44 849.69	**78 525.10**	44 484.10	76 488.57	65 153.84	-17%	57 228.04
86	4	84 725.68	69 716.59	**148 241.69**	69 171.66	145 660.23	143 093.00	-3%	88 465.77
87	5	121 954.56	103 340.12	**251 581.81**	102 776.68	248 436.91	255 624.26	2%	124 528.99
88	6	171 964.31	146 959.43	**398 541.24**	146 245.14	394 682.05	405 215.47	2%	164 579.56
89	7	205 132.17	188 548.24	**587 089.48**	188 304.66	582 986.71	594 431.19	1%	207 929.17
90	8	244 906.60	225 019.39	**812 108.87**	224 725.87	807 712.58	825 932.60	2%	254 039.31
91	9	304 548.68	274 727.64	**1 086 836.51**	274 186.54	1 081 899.12	1 102 477.58	1%	302 521.25
92	10	364 318.89	334 433.79	**1 421 270.30**	333 987.80	1 415 886.92	1 426 920.64	0%	353 136.09
93	11	411 187.03	387 752.96	**1 809 023.26**	387 516.71	1 803 403.63	1 802 212.98	0%	405 794.72
94	12	459 592.32	435 389.68	**2 244 412.94**	435 165.27	2 238 568.90	2 231 402.44	-1%	460 557.83
95	13	516 650.81	488 121.56	**2 732 534.50**	487 843.42	2 726 412.32	2 717 633.54	-1%	517 635.92
96	14	573 492.45	545 071.63	**3 277 606.13**	544 824.48	3 271 236.79	3 264 147.46	0%	577 389.28
97	15	632 903.94	603 198.19	**3 880 804.32**	602 954.22	3 874 191.02	3 874 282.03	0%	640 328.02
98	16	702 508.05	667 705.99	**4 548 510.31**	667 403.46	4 541 594.48	4 551 471.75	0%	707 112.03
99	17	780 334.98	741 421.52	**5 289 931.83**	741 080.89	5 282 675.36	5 299 247.79	0%	778 551.03
100	18	858 612.22	819 473.60	**6 109 405.43**	819 161.88	6 101 837.24	6 121 237.98	0%	855 604.52
101	19	944 330.25	901 471.24	**7 010 876.66**	901 131.43	7 002 968.67	7 021 166.80	0%	939 381.82
102	20	1 036 634.55	990 482.40	**8 001 359.06**	990 123.79	7 993 092.46	8 002 855.40	0%	1 031 142.02
103	21	1 125 713.46	1 081 174.00	**9 082 533.07**	1 080 868.07	9 073 960.53	9 070 221.61	0%	1 132 294.06

STORAGE

4th degree with 10 decimals non 0 intercept	
a	15660.08
b	-25178.9
c	12839.58
d	338.8153
e	4.027613



SILT

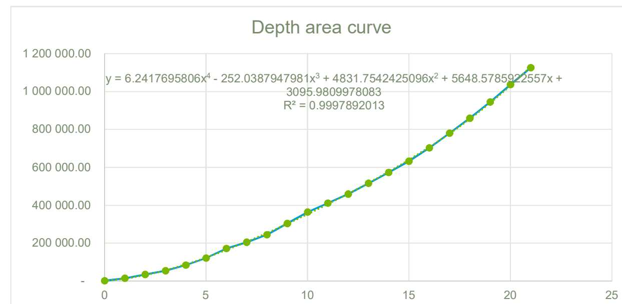
Volume		70000.00	m3
Height	3.077304812	70 000.00	m3
Elevation	85.07730481		masl

Bottom intake depth	elevation	Volume	
4	86	143 093.00	dead storage provided is sufficient

Storage req	4540000	m3
Irrigator storage	150000	m3
Total storage	4690000	m3

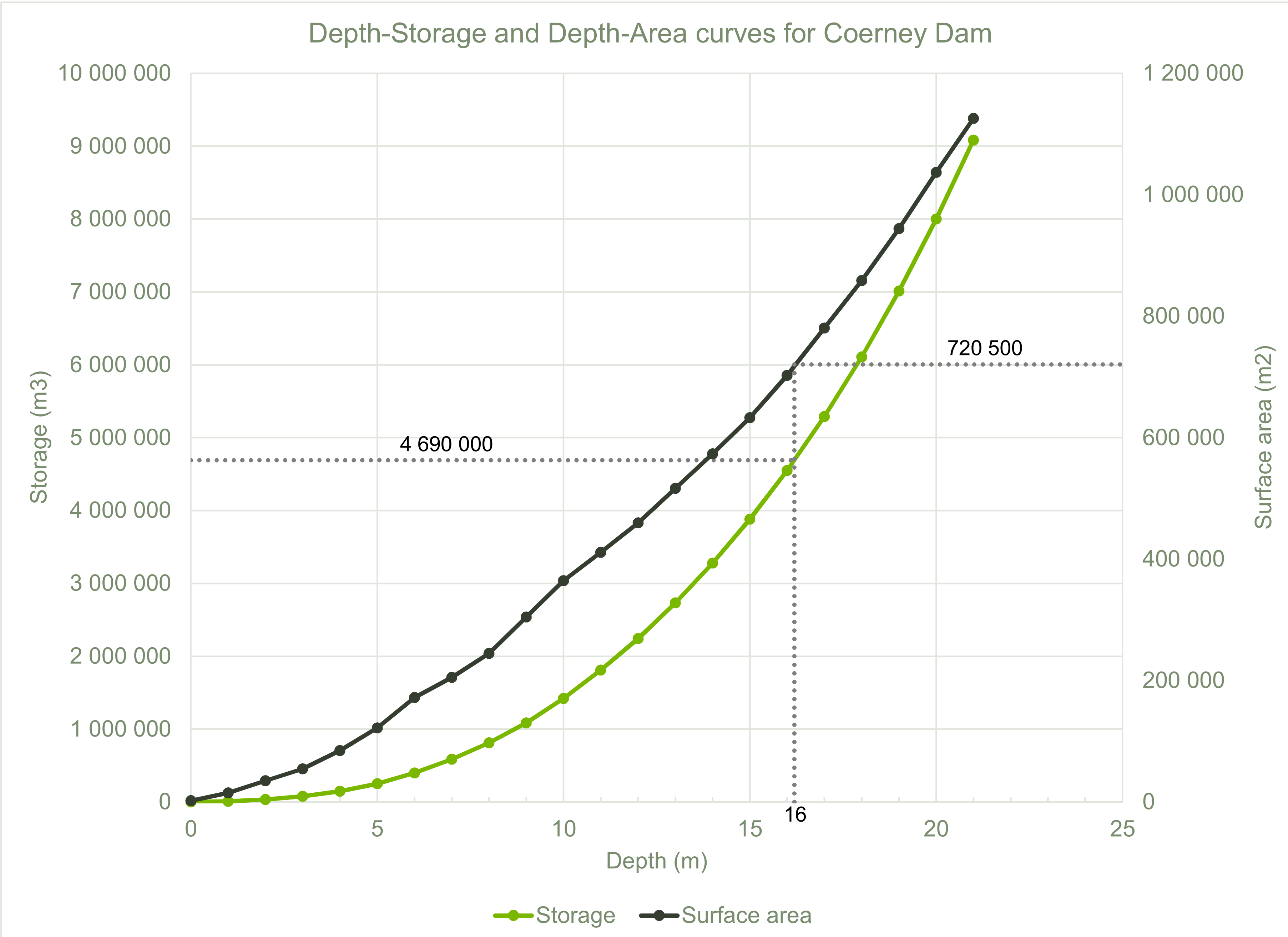
AREA

4th degree with 10 decimals non 0 intercept	
a	3095.981
b	5648.579
c	4831.754
d	-252.039
e	6.24177



FINAL FSL DETERMINATION

Elevation	Depth	Storage	Surface area
98.1927456	**16.192746**	**4 690 000.00**	**720 491.85**


Depth-Storage and Depth-Area curves for Coerney Dam
Storage (m3)
Surface area (m2)
Depth (m)
10 000 000
9 000 000
8 000 000
7 000 000
6 000 000
5 000 000
4 000 000
3 000 000
2 000 000
1 000 000
0
1 200 000
1 000 000
800 000
600 000
400 000
200 000
0
0
5
10
15
20
25
4 690 000
720 500
16
Storage
Surface area

Spillway capacity

Project No	**112546**	**Date**	**Sep-19**
Project name	**Algoa balancing dam: Lower Coerney**		

General spillway parameters

Parameter	Value			
	Spillway	NOC crest		
Discharge coefficient, C	see table below			
Crest length, L (m)	50	600	m	
NOC crest width (m)		5	m	
Full supply Level (m)	98.20	98.2	m amsl	
Non-overspill crest level (m)	102	102	m amsl	
Flash board top Level	102.00	102	m amsl	
Side slope (1V:H) RIGHT	0	4		
Side slope (1V:H) LEFT	0	4		
Freeboard	3.8	3.8	m	
Is it a full ogee? (1=yes, 0=No)	1		(if "no" is chosen the nappe will spring away at twice the design head, Ho, and drop the Cd)	

$$Q = CL_eH^{1.5}$$

Input

TR 126

Discharge coefficient for broad crested weir

$$Q = 1.448 \times C_f \times b \times H^{1.5}$$
$$h/_L < 0.36 \therefore C_f = 1.0$$
$$0.36 \le h/_L < 1.48 \therefore C_f = 0.241\, h/_L + 0.913$$
$$h/_L > 1.48 \therefore use\ sharp\ crest\ formula$$

Discharge for a sharp crest weir

$$Q = 1.838 L_e H^{1.5}$$

Ogee discharge parameters

Ho =	2.9	m	Ho/Hmax	0.75
Co =	2.18	m		

Contraction due to piers and abutments

Number of piers, N =	0	No
Pier contraction coefficient, Kp =	0.02	
Abutment contraction coefficient, Ka =	0.2	

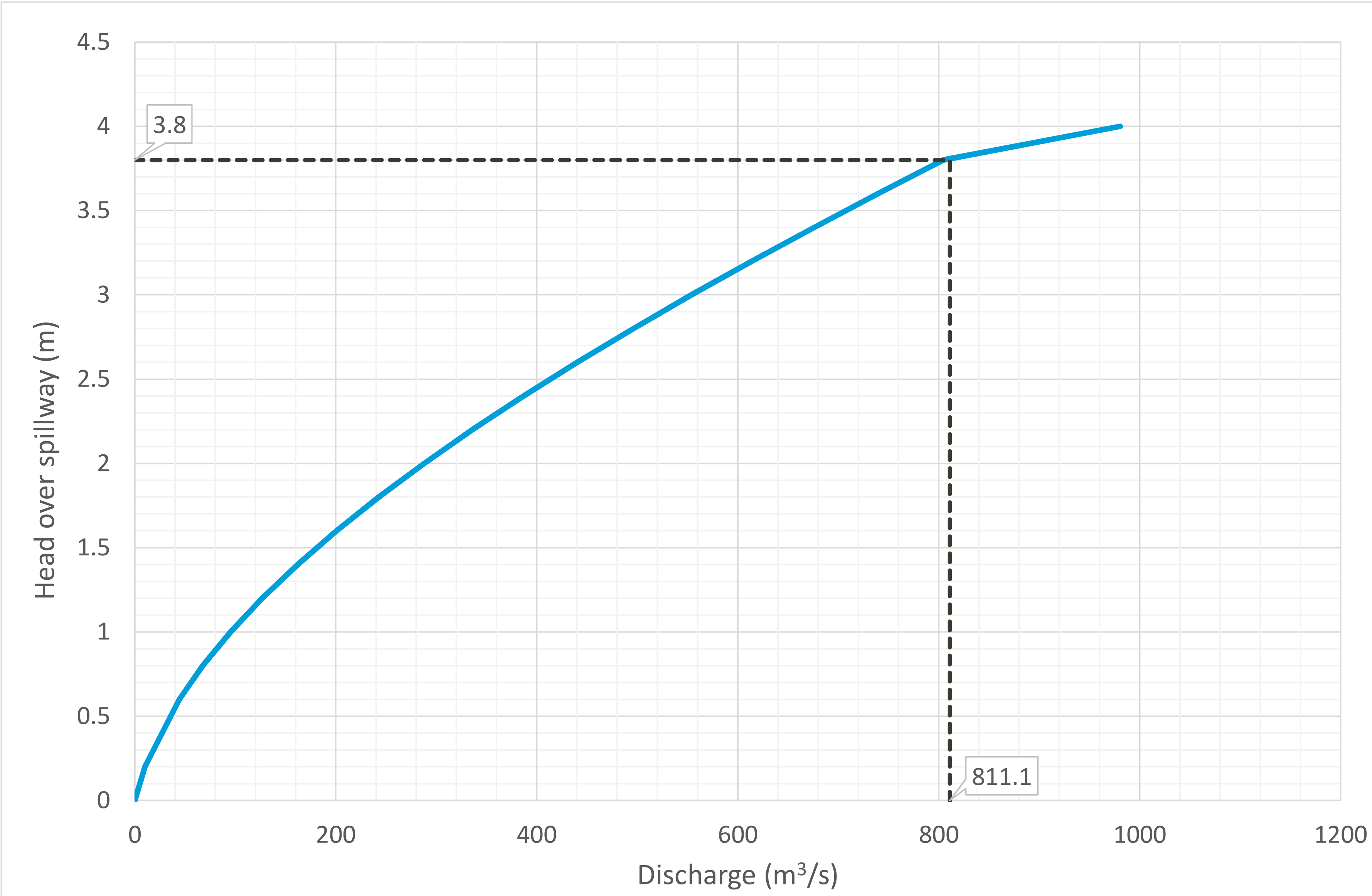
$$L' = L - 2(NK_p + K_a)H_e$$

L' = Net length of crest

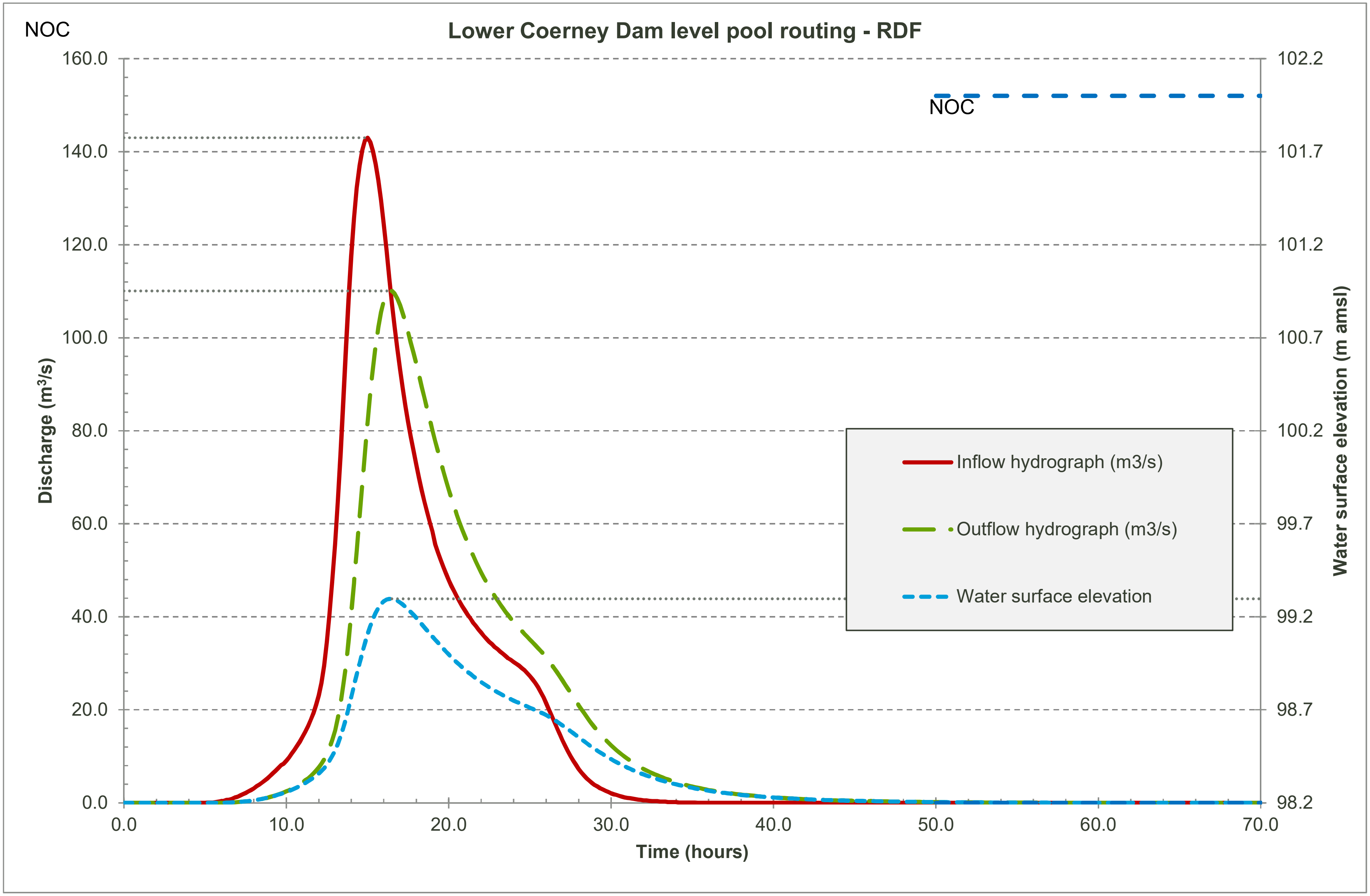
He = Head on crest

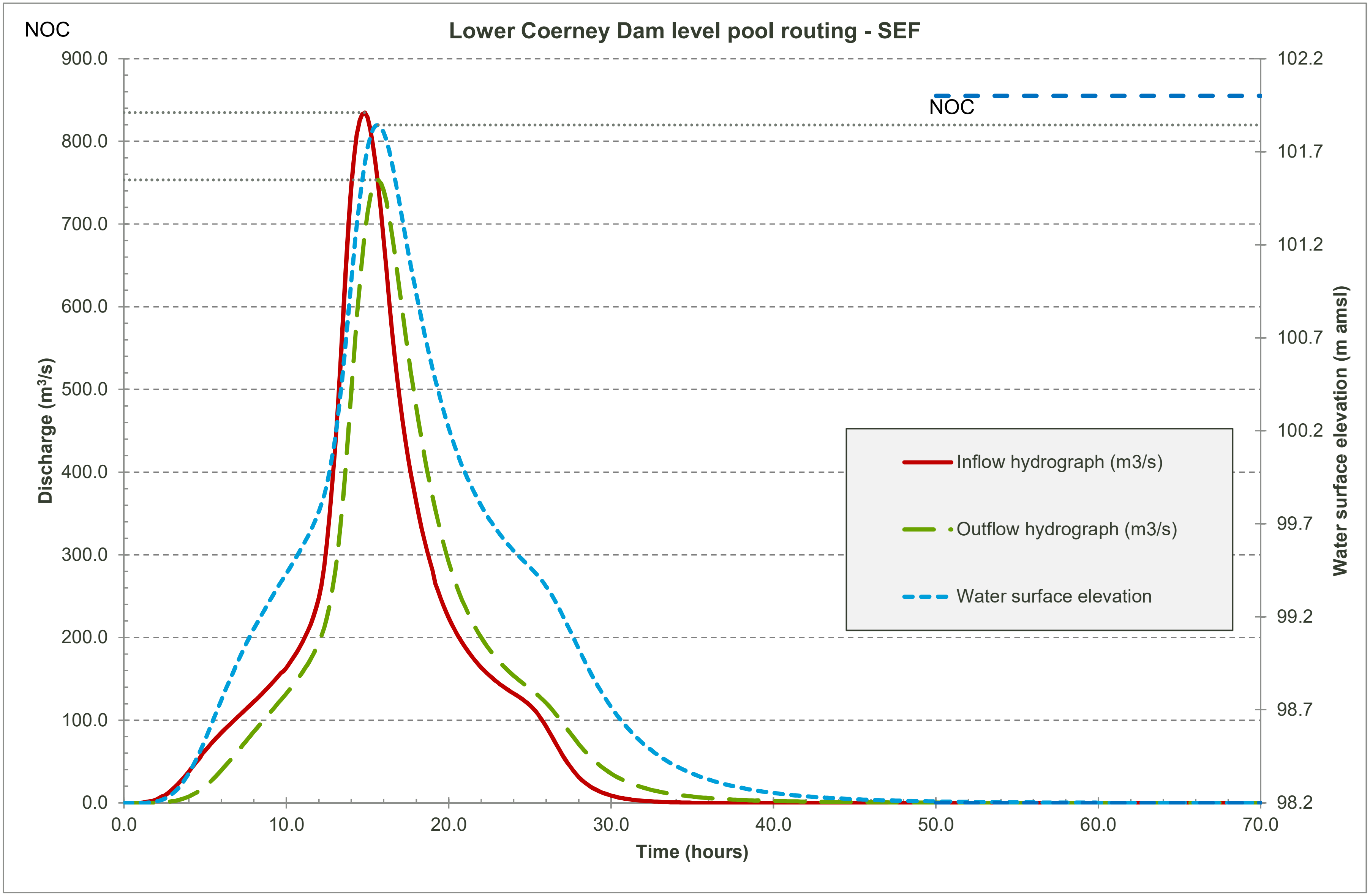
Water level (m)	Head above FSL (m)		Discharge coefficient, C		Effective crest length, L' = Le (m)	Spillway crest discharge (m^3/s)	NOC discharge (m^3/s)	TOTAL	
	Ogee Spillway	NOC (m)	Ogee Spillway	NOC, Cd (m)	Ogee Spillway				
98.20	0.00	0.00	0	1.45	50.00	0.00	0.00	0	
98.30	0.10	0.00	1.75	1.45	49.96	2.77	0.00	3	
98.40	0.20	0.00	1.75	1.45	49.92	7.84	0.00	8	
98.50	0.30	0.00	1.79	1.45	49.88	14.70	0.00	15	
98.60	0.40	0.00	1.79	1.45	49.84	22.62	0.00	23	
98.70	0.50	0.00	1.83	1.45	49.80	32.24	0.00	32	
98.80	0.60	0.00	1.86	1.45	49.76	43.00	0.00	43	
98.90	0.70	0.00	1.86	1.45	49.72	54.15	0.00	54	
99.00	0.80	0.00	1.89	1.45	49.68	67.11	0.00	67	
99.10	0.90	0.00	1.92	1.45	49.64	81.31	0.00	81	
99.20	1.00	0.00	1.92	1.45	49.60	95.15	0.00	95	
99.30	1.10	0.00	1.94	1.45	49.56	111.06	0.00	111	
99.40	1.20	0.00	1.97	1.45	49.52	128.00	0.00	128	
99.50	1.30	0.00	1.97	1.45	49.48	144.21	0.00	144	
99.60	1.40	0.00	1.99	1.45	49.44	162.83	0.00	163	
99.70	1.50	0.00	2.01	1.45	49.40	182.41	0.00	182	
99.80	1.60	0.00	2.03	1.45	49.36	202.53	0.00	203	
99.90	1.70	0.00	2.03	1.45	49.32	221.63	0.00	222	
100.00	1.80	0.00	2.05	1.45	49.28	243.87	0.00	244	
100.10	1.90	0.00	2.07	1.45	49.24	266.51	0.00	267	
100.20	2.00	0.00	2.07	1.45	49.20	287.59	0.00	288	
100.30	2.10	0.00	2.09	1.45	49.16	312.11	0.00	312	
100.40	2.20	0.00	2.10	1.45	49.12	336.84	0.00	337	
100.50	2.30	0.00	2.10	1.45	49.08	359.77	0.00	360	
100.60	2.40	0.00	2.12	1.45	49.04	386.76	0.00	387	
100.70	2.50	0.00	2.14	1.45	49.00	413.80	0.00	414	
100.80	2.60	0.00	2.14	1.45	48.96	438.51	0.00	439	
100.90	2.70	0.00	2.15	1.45	48.92	466.51	0.00	467	
101.00	2.80	0.00	2.17	1.45	48.88	496.26	0.00	496	
101.10	2.90	0.00	2.17	1.45	48.84	522.66	0.00	523	
101.20	3.00	0.00	2.18	1.45	48.80	552.79	0.00	553	
101.30	3.10	0.00	2.19	1.45	48.76	583.08	0.00	583	
101.40	3.20	0.00	2.21	1.45	48.72	615.27	0.00	615	
101.50	3.30	0.00	2.21	1.45	48.68	643.81	0.00	644	
101.60	3.40	0.00	2.22	1.45	48.64	676.73	0.00	677	
101.70	3.50	0.00	2.23	1.45	48.60	709.69	0.00	710	
101.80	3.60	0.00	2.23	1.45	48.56	739.72	0.00	740	
101.90	3.70	0.00	2.25	1.45	48.52	775.38	0.00	775	
102.00	3.80	0.00	2.26	1.45	48.48	811.06	0.00	811	
102.10	3.90	0.10	2.26	1.45	48.44	842.59	27.49	870	
102.20	4.00	0.20	2.27	1.45	48.40	879.55	77.79	957	
102.30	4.10	0.30	2.28	1.45	48.36	916.36	142.99	1059	
102.40	4.20	0.40	2.28	1.45	48.32	949.30	220.26	1170	
102.50	4.30	0.50	2.29	1.45	48.28	987.28	307.99	1295	
102.60	4.40	0.60	2.31	1.45	48.24	1026.90	405.07	1432	
102.70	4.50	0.70	2.32	1.45	48.20	1067.24	510.72	1578	
102.80	4.60	0.80	2.32	1.45	48.16	1102.10	624.32	1726	
102.90	4.70	0.90	2.33	1.45	48.12	1143.70	745.36	1889	
103.00	4.80	1.00	2.33	1.45	48.08	1179.41	873.43	2053	
103.10	4.90	1.10	2.33	1.45	48.04	1215.45	1008.21	2224	
103.20	5.00	1.20	2.33	1.45	48.00	1251.80	1149.38	2401	
103.30	5.10	1.30	2.33	1.45	47.96	1288.47	1296.69	2585	
103.40	5.20	1.40	2.33	1.45	47.92	1325.45	1449.92	2775	
103.50	5.30	1.50	2.33	1.45	47.88	1362.72	1608.86	2972	
103.60	5.40	1.60	2.33	1.45	47.84	1400.30	1773.33	3174	
103.70	5.50	1.70	2.33	1.45	47.80	1438.18	1943.18	3381	
103.80	5.60	1.80	2.33	1.45	47.76	1476.34	2118.25	3595	
103.90	5.70	1.90	2.33	1.45	47.72	1514.79	2308.94	3824	
104.00	5.80	2.00	2.33	1.46	47.68	1553.53	2506.89	4060	
104.10	5.90	2.10	2.33	1.47	47.64	1592.54	2711.55	4304	
104.20	6.00	2.20	2.33	1.48	47.60	1631.83	2922.88	4555	
104.30	6.10	2.30	2.33	1.48	47.56	1671.39	3140.85	4812	
104.40	6.20	2.40	2.33	1.49	47.52	1711.21	3365.43	5077	
104.50	6.30	2.50	2.33	1.50	47.48	1751.31	3596.60	5348	
104.60	6.40	2.60	2.33	1.50	47.44	1791.66	3834.35	5626	
104.70	6.50	2.70	2.33	1.51	47.40	1832.27	4078.66	5911	
104.80	6.60	2.80	2.33	1.52	47.36	1873.13	4329.51	6203	
104.90	6.70	2.90	2.33	1.52	47.32	1914.24	4586.91	6501	
99.33	1.13	0.00	1.94	1.45	49.55	115.00	0.00	115.00	**1:100yr=115**
99.49	1.29	0.00	1.97	1.45	49.48	143.00	0.00	143.00	**RDF=143**
102.08	3.88	0.08	2.26	1.45	48.45	835.00	0.00	835.00	**SEF=835**
102.00	3.80	0.00	2.26		48.48	811.06	0.00	811.06	**NOC**
	-98.20	0.00	#N/A		89.28	#N/A	0.00	#N/A	RDF(200)
	-98.20	0.00	#N/A		89.28	#N/A	0.00	#N/A	SEF(PMF)

RDF Recommended design flood (100 -> 1:100 yr flood, 200-> 1:200 yr flood)
SEF Safety evaluation flood
NOC Non overspill crest (lowest level)
STW Spillway training wall



Head over spillway (m)
Discharge (m³/s)
3.8
811.1
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
0
200
400
600
800
1000
1200


NOC
Lower Coerney Dam level pool routing - RDF
Discharge (m³/s)
Water surface elevation (m amsl)
Time (hours)
160.0
140.0
120.0
100.0
80.0
60.0
40.0
20.0
0.0
102.2
101.7
101.2
100.7
100.2
99.7
99.2
98.7
98.2
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
NOC
Inflow hydrograph (m3/s)
Outflow hydrograph (m3/s)
Water surface elevation


NOC
Lower Coerney Dam level pool routing - SEF
Discharge (m³/s)
Water surface elevation (m amsl)
Time (hours)
900.0
800.0
700.0
600.0
500.0
400.0
300.0
200.0
100.0
0.0
102.2
101.7
101.2
100.7
100.2
99.7
99.2
98.7
98.2
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
NOC
Inflow hydrograph (m3/s)
Outflow hydrograph (m3/s)
Water surface elevation

Freeboard Calculations for Algoa: Lower Coerney Dam

Project Number:	**112546**	**Date:**	**2019/11/08**
Title:	**Algoa: Lower Coerney Dam**	**Calculated by:**	**O Human**
River:	**Tributary to Sundays River**		Input
Location:	**Kirkwood**		Calculated

1. DAM DETAILS

Dam name	Lower Coerney		Full Supply Level	98.20	masl
Full Supply Volume	4.690	Mm^3	Non Overspill Crest	102.00	masl
Full Supply Area	72.00	ha	Bed level	82.00	masl
Depth at wall	16.2	m	Available freeboard	3.80	m
Wall height (as per regulations)	20.0	m			
Average depth	14.0	m	Spillway Type	Ogee weir, side channel	
			Spillway Width	50.00	m
Dam Size	Medium		NOC Length	600.00	m
Hazard Rating	High				
Dam Category	III		Upstream slope	3.00	H:1V
Dam Type	Earthfill dam		Upstream slope protectior	Rough - Rip-rap (single layer)	

2. FLOOD SURCHARGE

Flood level at the dam wall after taking attenuation into account (either via level pool flood routing or hydrodynamic modelling).

Recommended Design Flood (RDF)			Safety Evaluation Flood (SEF)		
Recurrence Interval	200	years	Recurrence Interval	PMF	
Inflow	143.00	m^3/s	Inflow	835.00	m^3/s
Outflow	113.00	m^3/s	Outflow	753.00	m^3/s
Maximum Water Elevation	99.30	masl	Maximum Water Elevatior	101.84	masl
Level above spillway	1.10	m	Level above spillway	3.64	m

3. DAM BREAK FLOOD SURCHARGE

Should an upstream dam fail, the additional volume of water which enters the dam should be accounted for in the flood routing.

Dam break flood surcharge (incremental above normal flood event)	0.0	m

4. GATE FAILURE SURCHARGE

Whenever there are controlled gates at a dam that are relied upon to release flood water, it must be assumed that 25% of these will not be operable (ie closed).

Gate failure surcharge (incremental above normal flood event) - 1 of 4 gates fails	0.00	m

5. WIND SPEED AND FETCH

The base wind speed can be determined either from available data, weather models or from the graphs presented in the SANCOLD freeboard guidelines, 2011 (See Figure A)

Fetch

Fetch length (longest straight line distance from the dam to the edge of the basin)	2 200	m

Note that, in certain conditions, wave effects can move around slight bends in the basin reservoir.

Wind speed

1:100yr Mean hourly wind speed (from Figure A) at 10 m elevation	24.0	m/s
Determine time required for wind to reach generation equilibrium (from Figure B)	0.22	hours
Adjustment factor to convert hourly wind speed to duration wind speed	1.04	-
Mean duration wind speed (1:100yrs)	24.92	m/s
Adjustment factor to convert overland wind speed to over water wind speed (from Figure C)	1.20	-
Over water wind speed	29.91	m/s

6. WIND SET-UP

Wind set-up is the result of surface water being driven in the downwind direction resulting in a build up of water against the dam wall.

Fetch multiple	1.0	

Note that, wind set-up effects can move around substantial bends in the basin reservoir (hence the fetch is often doubled).

Wind set-up	0.04	m

7. DESIGN WAVE HEIGHT

The calculations provided in the SANCOLD 2011 guidelines are provided in the "SANCOLD calcs" tab

Significant wave height (H_s)	1.23	m
Allowance for overtopping	1.10	

Use this factor with caution: It assumes that concrete dams can readily be overtopped whereas earthfill dams are vulnerable to downstream erosion. This may or may not be the case. Use cell U63 if needed.

Design wave height	1.35	m
2% Exceedence wave height ($H_{2\%}$)	1.89	m

8. WAVE RUN-UP

The calculations provided in the SANCOLD 2011 guidelines are provided in the "SANCOLD calcs" tab

Base Wave Run-up ($R_{2\%}$)	2.26	m
Wave angle to dam wall (0° is normal to the wall)	0	°
Adjustment for oblique wave front ()	1.00	
Foreshore slope (see figure alongside)	100	H:1V
Adjustment for shallow foreshore ()	1.00	
Additional adjustment factor (to account for berms, …)	1.00	
Recommended (Design) wave run-up	2.26	m

9. SEICHES AND SURGES

Surges refer to rises in the reservoir level induced by variations in atmospheric pressure. Only applicable to medium (0.5m for >10km2) or large reservoirs (1.0m for >100km2).

Atmospheric pressure variation surge allowance	0.00	m

Seiches refer to long-period oscillations that persist in a body of water due to resonance of its natural modes with an external wave (such as the closing of a gate, squalls, flash floods, …) - from local data.

Oscillation / Seiche allowance	0.00	m

10. EARTHQUAKES

Refer to Figures D to determine waves caused by earthquakes. Usually only applicable to concrete dams.

Ground acceleration	0.02	g
Oscillation period	4.00	s
Amplitude of movement	0.08	m
Amplitude of wave	0.08	m

11. LAND SLIDES

Only applicable to reservoirs with steep and unstable slopes.

Water depth	14.00	m
Slide volume falling into the reservoir (ie volume of water displaced)	0	m^3
Slide width	20.0	m
Density ratio of slide material to water (ρ_s/ρ_w)	1.60	
Impact angle (α)	30.0	°
Radius from centre of slide impact	2 000	m
Propagation direction (γ) (see figure alongside)	90.00	°
Wave height	0.00	m
Wave amplitude	0.00	m

12. COMBINING FREEBOARD COMPONENTS

The above freeboard elements are to be combined using the following criteria

	RDF Water Level	SEF Water Level	Wave Run-up	Wind Set-up	Surges & Seiches	Earthquake	Landslide	Flood gates failure		
1	x		x						3.36	m
2	x		x	x	x				3.41	m
3						x			0.08	m
4	x						x		1.10	m
5	x		x	x	x			x	3.41	m
6		x							3.64	m

Dam Size	Medium	
Hazard Rating	High	
Freeboard criteria	2;3;4;5;6	
Required freeboard	3.64	m

12.1 MINIMUM FREEBOARD REQUIREMENTS

Despite the above calculations there are certain minimum freeboard requirements that should be met.

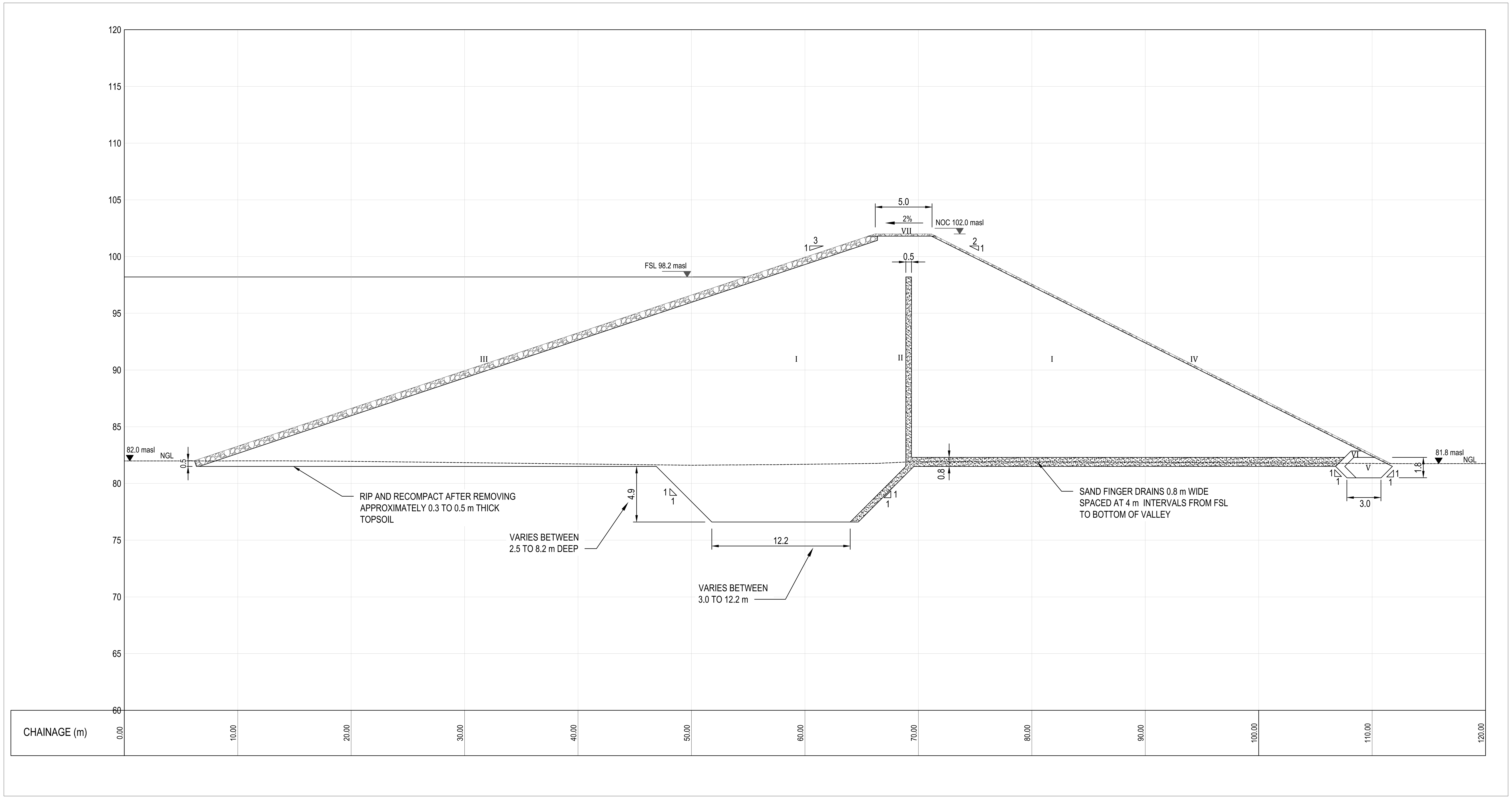
Type of dam	Minimum total freeboard (m)	Minimum difference in level between stillwater RDF surcharge level and non-overspill crest (m)
Earthfill (Category I)	0.8	0.5
Earthfill (Categories II & III)	0.0	1.5
Rockfill (Categories II & III)	0.0	1.5
Concrete (Categories II & III)	1.5	1

Minimum freeboard for a Category III, Earthfill dam	2.60	m

13. FREEBOARD RESULTS

Required freeboard	3.64	3.64	m
Provided freeboard		3.80	m

ZONES	MATERIAL
I - HOMOGENEOUS FILL	SELECTED SEMI-PERVIOUS TO IMPERVIOUS MATERIAL FROM BASIN EXCAVATIONS
II - CHIMNEY AND FINGER DRAINS	IMPORTED SAND
III - RIP RAP	COBBLECRETE OR IMPORTED ROCK
IV - TOPSOIL	STRIPPED FROM BASIN EXCAVATIONS
V - ROCK TOE	IMPORTED ROCK
VI - GRAVEL	IMPORTED GRAVEL
VII - GRAVEL CAPPING	GRAVEL CAPPING EXCAVATED IN DAM BASIN



TYPICAL SECTION SHOWING
MAXIMUM EMBANKMENT HEIGHT

1:200

NOTES:

1.	NON-OVERSPILL CREST LEVEL:	102.0 masl
2.	FULL SUPPLY LEVEL :	98.2 masl
3.	FREEBOARD :	3.8 m
4.	WATER SURFACE AREA AT FSL :	72 ha
5.	GROSS CAPACITY :	4.69 million m^3
6.	CREST LENGTH :	441 m
7.	CREST WIDTH :	5 m
8.	MAXIMUM WALL HEIGHT :	20.5 m
9.	UPSTREAM SLOPE :	1V:3H
10.	DOWNSTREAM SLOPE :	1V:2H
11.	MINIMUM BASIN LEVEL :	82.0 masl
12.	DOWNSTREAM TOE LEVEL:	81.5 masl

ADDITIONAL NOTES

A. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.
B. ALL LEVELS IN METRES ABOVE SEA LEVEL (masl).
C. DAM EMBANKMENT TO BE CONSTRUCTED 2% HIGHER FOR SETTLEMENT ALLOWANCE I.E. NOC 102.0 masl + (2% x 20 m) = NOC 102.4 masl.
D. DAM CREST TO BE CONSTRUCTED WITH 2% CROSSFALL SLOPE TOWARDS DAM BASIN.
E. EXCAVATION DEPTH OF CUT-OFF TRENCH TO BE APPROVED BY ENGINEER ON SITE.
F. GROUTING DETAILS TO BE CONFIRMED.
G. SHOULD MATERIALS NOT BE AVAILABLE IN DAM BASIN OR ESSENTIAL EXCAVATIONS THEY ARE TO BE IMPORTED FROM COMMERCIAL SOURCES.
H. FOR OUTLET WORKS & MISCELLANEOUS DETAILS REFER TO DRAWING 112546-0000-DRG-CC-003.
I. FOR SPILLWAY DISCHARGE CHANNEL DETAILS REFER TO DRAWING 112546-0000-DRG-CC-004.
J. FOR OUTLET TOWER DETAILS REFER TO DRAWING 112546-0000-DRG-CC-005.

ABBREVIATIONS

NGL - NATURAL GROUND LEVEL
FSL - FULL SUPPLY LEVEL
NOC - NON-OVERSPILL CREST

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CLIENT

REV	DATE	REVISION DETAILS	APPROVED
A	06/02/20	FOR INFORMATION	E. VAN DER BERG

SCALE: AS SHOWN
SIZE: A1
FOR INFORMATION

DRAWN: A. KNOX
DESIGNED: O. HUMAN
CHECKED: F. DENEYS
APPROVED: E. VAN DER BERG
DATE
ECSA 920455

PROJECT
SUPPORT OF THE WATER RECONCILIATION STRATEGY OF THE ALGOA WATER SUPPLY SYSTEM

TITLE
LOWER COERNEY BALANCING DAM EMBANKMENT SECTION

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
112546	0000	DRG	CC	003	A

Design Flood Peak Estimates for Proposed Lower Coerney and Upper Scheepersvlakte Dam Sites

1 Design Flood Analysis

1.1 Introduction

The design flood peaks for various recurrence intervals were estimated for the Lower Coerney and Upper Scheepersvlakte dam sites. **Figure 1.1** shows the two sites and their catchment areas in relation to the existing Scheepersvlakte Dam.

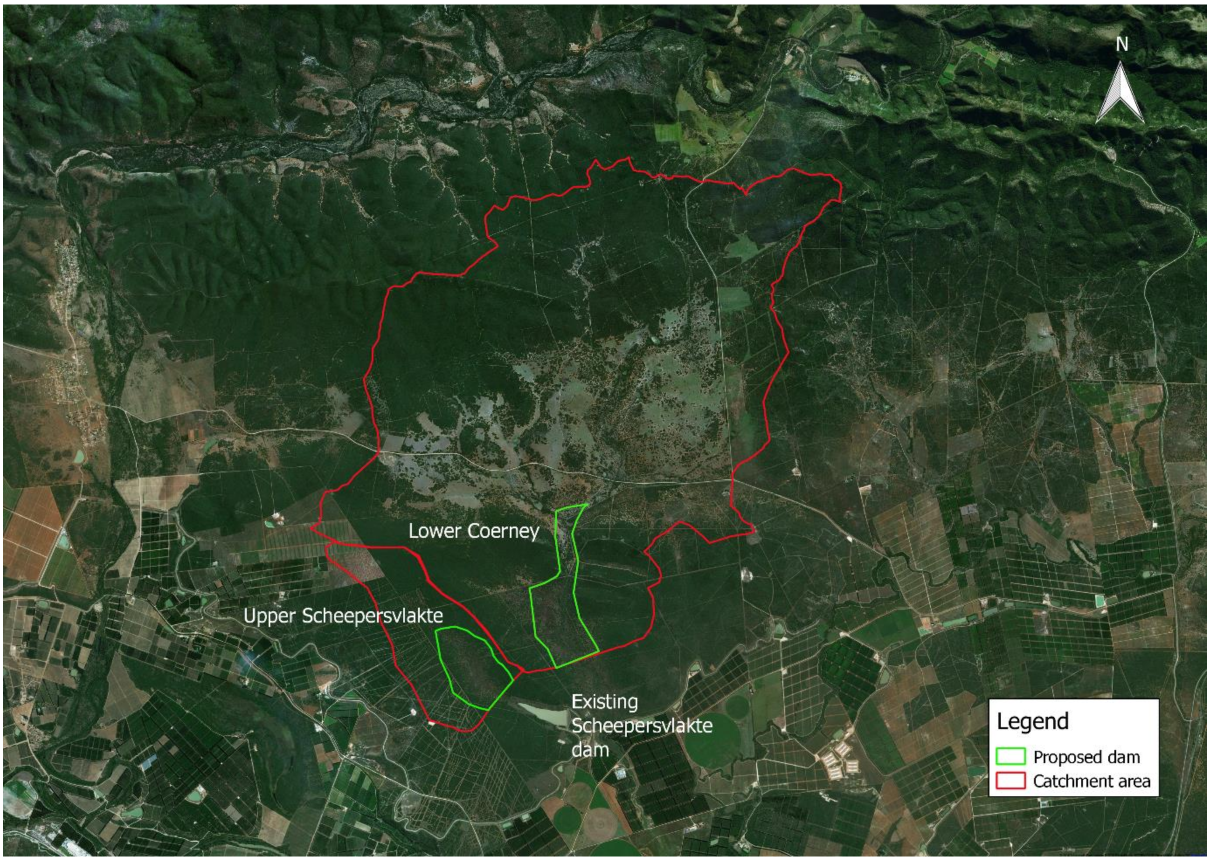

Figure 1.1: Proposed Lower Coerney and Upper Scheepersvlakte dam sites

1.2 Objective

Determination of design flood peaks for recurrence intervals (RIs) of 1:2, 1:5, 1:10, 1:20, 1:50 1:100 and 1:200 years.

1.3 Scope of Work

The scope of work for this component of the Study encompassed the following sequential phases:

- Determining design rainfall for the Study catchments
- Determining physiographic catchment characteristics

- Undertaking appropriate deterministic design flood analyses.

1.4 Approach and Methodology

Based on the size of the Study catchments and the lack of streamflow records in the Study catchments, we decided to follow only a deterministic approach for the determination of the design floods.

Conventionally, under the deterministic category, three alternatives for design flood analyses are followed, namely the Unit Hydrograph-, SCS- and Rational Method-approaches. The non-availability of sub-daily rainfall records, as well as the non-availability of sub-daily streamflow records in or near the Study catchments negated the derivation of Unit Hydrographs. Hence, we applied the SCS- and Rational Method-approaches for design flood determination under this category.

1.5 Design Rainfall

Deterministic design flood methods require daily or sub-daily design rainfall as input. To this end we used the WRC Report by Smithers and Schulze (2000). The station nearest to the two Study catchments is SAWB number 0055655 W (Twembani) and the 1-day point rainfalls for various RIs are presented in **Table 1-1** below. The location of the station relative to the Study catchments is shown in **Figure 1.2**.

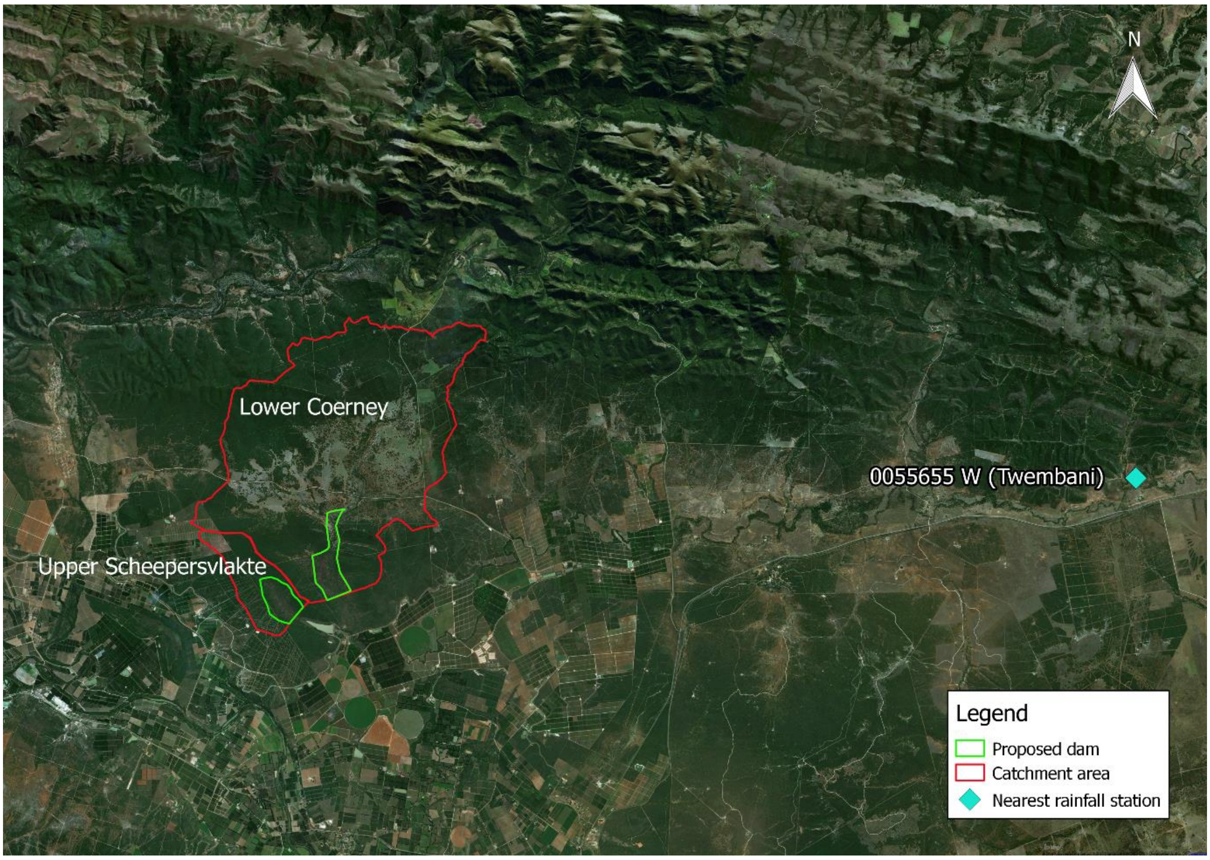

Figure 1.2: Nearest rainfall station to Study catchments

Table 1-1: 1-Day point design rainfalls (mm)

Recurrence Interval (y)	Point Design Rainfall (mm)
1:2	51
1:5	76
1:10	95
1:20	116
1:50	146
1:100	172
1:200	202

The 1-day point rainfalls were converted to the 24-hour point rainfall using Adamson's (1981) conversion factor of 1.11. These 24-hour point design rainfalls were then converted into respective storm-duration rainfalls by applying the Adamson (1981) sub-daily ratios for the winter rainfall (coastal) region (R2). **Table 1-2** provides the 24-hour point design rainfall values for various RIs.

Table 1-2: 24-Hour point design rainfalls (mm)

Recurrence Interval (y)	24-Hour Point Design Rainfall (mm)
1:2	56.6
1:5	84.3
1:10	105.5
1:20	128.8
1:50	162.1
1:100	190.9
1:200	224.2
PMP	666

For the application of the deterministic design flood determination methods, areal design rainfalls for the applicable critical durations are required. To convert the 24-hour point design rainfall values to areal design rainfalls for the Lower Coerney and Upper Scheepersvlakte Study catchments, Areal Reduction Factors (ARFs) of 96.7% and 100% were applied respectively, based on the ARF relationships for South Africa developed by Alexander (1990).

2 Deterministic Design Flood Determination

2.1 Catchment Characteristics

Relevant parameters describing the physiographic catchment characteristics that are required for design flood calculations were derived from the 30m x 30m NASA Shuttle Radar Topographic Mission (SRTM) DEM for the Study area using GIS software. Key catchment parameters for the catchment upstream of the dam sites are presented in **Table 2-1**. The characteristics of the soil were based on the Soil and Terrain Database for Southern Africa (SOTERSAF, 2003).

Table 2-1: Catchment characteristics upstream of the Lower Coerney and Upper Scheepersvlakte Intakes

Characteristic	**Quantum**	
	Lower Coerney	**Upper Scheepersvlakte**
Area	33.6 km^2	3.4 km^2
Length of longest watercourse	9.83 km	3.67 km
Equal-Area Slope of longest watercourse	0.0148 m/m	0.0222 m/m
Average catchment slope	6.55 %	3.27 %

2.2 Rational Method

The Rational Method yields a design flood peak only (i.e. no flood hydrograph). The flood response of the catchment is expressed by two quasi-physical parameters: Time of Concentration (T_c), which is a function of the length of the longest watercourse and the average slope of that watercourse; and the Runoff Coefficient (C). T_c for the Lower Coerney catchment is 1.95 hours, while T_c for the Upper Scheepersvlakte catchment is 0.78 hours.

Five catchment characteristics affect the value of C, namely: catchment slope, permeability of the soil, land use, mean annual precipitation and flood recurrence interval. As there is no objective theoretical method for determining C, subjective elements of experience and engineering judgement play a key role in applying this method reliably. The 1:100-year and 1:200-year RI runoff coefficient for the Lower Coerney catchment is 0.225 (30% thick bush and forest, 60% light bush and farmland and 10% poor grasslands; 50% permeable, 45% semi-permeable and 5% impermeable). The results of the Rational Method calculations at the Lower Coerney dam site are summarised in **Table 2-2** along with the runoff coefficient for each recurrence interval.

Table 2-2: Rational Method – design flood peaks at the Lower Coerney dam site (m^3/s)

Recurrence Interval (y)	Runoff Coefficient	Average Rainfall Intensity (mm/h)	Flood Peak (m^3/s)
1:2	0.113	14.8	16
1:5	0.124	22.1	26
1:10	0.135	27.7	35
1:20	0.151	33.8	48
1:50	0.187	42.5	74
1:100	0.225	50.1	105
1:200	0.225	58.8	124
PMF	0.533	174.7	869

The 1:100-year and 1:200-year RI runoff coefficient for the Upper Scheepersvlakte catchment is 0.192 (50% thick bush and forest, 40% light bush and farmland and 10% poor grasslands; 50% permeable and very permeable, 45% semi-permeable and 5% impermeable). The results of the Rational Method calculations at the Upper Scheepersvlakte dam site are summarised in **Table 2-3** along with the runoff coefficient for each recurrence interval.

Table 2-3: Rational Method – design flood peaks at the Upper Scheepersvlakte dam site (m^3/s)

Recurrence Interval (y)	Runoff Coefficient	Average Rainfall Intensity (mm/h)	Flood Peak (m^3/s)
1:2	0.096	29.7	3
1:5	0.105	44.2	4
1:10	0.115	55.3	6
1:20	0.128	67.5	8
1:50	0.159	85.0	13
1:100	0.192	100.1	18
1:200	0.192	117.6	21
PMF	0.507	349.2	167

2.3 SCS Method

The SCS method estimates peak discharges and flood volumes based on the catchment soil retention (S), lag time (T_L), hydrological soil group and the Curve Number (CN). The SCS takes most factors into account that affect runoff, such as the size and characteristics of the catchment, temporal distribution of rainfall, soil type, and land-use. A key input in the SCS method is the Curve Number (CN) of the catchment area. The CN is used as an index of a catchment's stormflow response to a rainfall event and is a function of catchment characteristics such as hydrological soil properties and land cover. Given the distribution of soils in the study catchment mentioned in Section 2.1, a Type B soil type was selected, which describes a soil with a moderately low runoff potential. For both Study catchments, a land cover of 70% natural forest and 30% poor veld on a type B soil indicates a CN of 62.2 (Schmidt and Schulze, 1987).

The SCS model makes use of 24-hour design rainfall depths to compute the total stormflow depth. The 24-hour point design rainfall values presented in **Table 1-2** were used as input to the SCS model. Design point rainfalls were converted to areal rainfall by applying an ARF of 100% for both Study catchments.

The time distribution of rainfall over the course of a day is dependent on the synoptic conditions and rainfall mechanisms that typically produce design storms. Given the relatively evenly sustained nature of rainfall intensities in the study region, the SCS Curve Type 1 was chosen to represent the design rainfall distribution over a day.

Table 2-4 summarises the physical catchment characteristics of the catchment upstream of the Lower Coerney and Upper Scheepersvlakte Dam sites.

Table 2-4: SCS method catchment characteristics

Characteristic	Value	
	Lower Coerney	Upper Scheepersvlakte
Hydrological Soil Group	B	B
1:2-year, 30min rainfall intensity (mm/hr)	24.5	24.5
Lag time (hours)	2.57	1.42
Storm Type	1	1
Land use: Natural Forest; Fair/Poor Veld	100%	100%
Curve number (CN) for RI floods	62.2	62.2
Curve number (CN) for PMF	90	90
Soil retention (S) (mm)	154.4	154.4
Initial abstraction (I_a) (mm)	15.4	15.4

The above characteristics were used in the SCS utility included in the HEC-HMS modelling package (U.S. Army Corps of Engineers, 2015) to calculate the RI flood peaks. The resultant SCS flood peak estimates are presented in **Table 2-5.**

Table 2-5: SCS method design flood peaks at the Lower Coerney and Upper Scheepersvlakte dam sites (m^3/s)

Recurrence Interval (y)	Design Flood Peak (m^3/s)	
	Lower Coerney	Upper Scheepersvlakte
1:2	10	1
1:5	27	4
1:10	43	6
1:20	63	9
1:50	95	14
1:100	125	18
1:200	161	24
PMF	801	115

3 Recommended Design Flood Peaks

The foregoing Rational Method-based and SCS Method-based RI-flood peak estimates are subjected to similar types of uncertainties. Therefore, we recommend that the design flood peaks for the Lower Coerney and Upper Scheepersvlakte dam sites be based on the averages of the above two sets of estimates, as presented in **Table 3-1**.

Table 3-1: Recommended design flood peaks at the Lower Coerney and Upper Scheepersvlakte Dam sites

Recurrence Interval (y)	Design flood according to SANCOLD recommendations	Recommended Design Flood Peaks (m^3/s)	
		Lower Coerney	Upper Scheepersvlakte
1:2		13	2
1:5		27	4
1:10		39	6
1:20		56	9
1:50		85	14
1:100		115	18
1:200	Recommended Design Flood (RDF)	143	23
PMF	Safety Evaluation Flood (SEF)	835	141

4 References

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SMITHERS, J.C. AND SCHULZE, R.E., 2000. Development and evaluation of techniques for estimating short duration design rainfall in South Africa. WRC Report No. 681/1/00. Pretoria.

RATIONAL METHOD

Description of the catchment	Lower Coenery		
River details	Lower Coenery		
Calculated by	L.Garlick	Date	12-Sep-18

Physical characteristics

Size of the catchment (A)	33.6	km^2		Rainfall region	
Longest watercourse (L)	9.83	km		**Area distribution factors**	
Average slope (S_{av})	0.01478	m/m		Rural (α)	1
Dolomite area ($D_{\%}$)	0	%		Urban (β)	0
Mean annual rainfall (MAP)	480	mm		Lakes (γ)	0

Rural				Urban			
Surface slope	**%**	**Factor**	**C_s**	**Description**	**%**	**Factor**	**C_2**
Vleis and pans	8.993902	0.01	0.001	**Lawns**			
Flat areas	78.81098	0.06	0.047	Sandy, flat (<2%)			
Hilly	12.19512	0.12	0.015	Sandy, steep (>7%)			
Steep areas	0	0.22	0.000	Heavy soil, flat (<2%)			
Total (C_s)	**100**		**0.063**	Heavy soil,steep (>7%)			
Permeability	**%**	**Factor**	**C_p**	**Residential area**			
Very permeable	0	0.03	0.000	Houses			
Permeable	50	0.06	0.030	Flats			
Semi-permeable	45	0.12	0.054	**Industry**			
Impermeable	5	0.21	0.011	Light industry			
Total (C_p)	**100**		**0.095**	Heavy industry			
Vegetation	**%**	**Factor**	**C_v**	**Business**			
Thick bush & plantation	30	0.03	0.009	City centre			
Light bush and farmlands	60	0.07	0.042	Suburban			
Grasslands	10	0.17	0.017	Streets			
No vegetation	0	0.26	0.000	Maximum flood			
Total (C_v)	**100**	-	**0.068**	**Total (C_2)**	0		0

Time of concentration (T_c)						Notes
Overland flow			**Defined watercourse**			24-hour factor =1.11
$Tc = 0.604\left(\frac{rL}{\sqrt{S_{av}}}\right)^{0.467}$			$Tc = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$			D-hour factor = 0.53
						ARF (%) = 96.65
Tc =		hours	Tc =	**1.95**	hours	

Run-off coefficient

Return period (years), T	**2**	**5**	**10**	**20**	**50**	**100**	**200**
Run-off coefficient C_1 ($C_1 = C_s + C_p + C_v$)	0.225	0.225	0.225	0.225	0.225	0.225	0.225
Adjusted for dolomitec areas, C_{1D} ($=C_1 (1 - D_{\%}) + C1\ D_{\%}(\sum(D_{factor} \times C_{S\%}))$)	0.225	0.225	0.225	0.225	0.225	0.225	0.225
Adjustment factor for initial saturation, F_t (Taken as average of two set of values provided in DM)	0.50	0.55	0.60	0.67	0.83	1.00	1.00
Adjusted run-off coefficient, C_{1T} ($= C_{1D} \times F_t$)	0.113	0.124	0.135	0.151	0.187	0.225	0.225
Combined run-off coefficient C_t ($= \alpha C_{1T} + \beta_{C2} + \gamma_{C3}$)	0.113	0.124	0.135	0.151	0.187	0.225	0.225

Rainfall

Return period (years), T	**2**	**5**	**10**	**20**	**50**	**100**	**200**
Point rainfall (mm), P_T, with D(hour) factor applied	30	45	56	68	86	101	119
Point intensity (mm/hour), P_{iT} (= P_T / T_C)	15.4	22.9	28.6	34.9	44.0	51.8	60.9
Area reduction factor (%), ARF_T	96.7	96.7	96.7	96.7	96.7	96.7	96.7
Average intensity (mm/hour), I_T (= P_{iT}xARF_T)	14.8	22.1	27.7	33.8	42.5	50.1	58.8

Peak flow

Return period (years), T	**2**	**5**	**10**	**20**	**50**	**100**	**200**
Peak flow (m^3/s) $Q_T = \frac{C_T I_T A}{3.6}$	**16**	**26**	**35**	**48**	**74**	**105**	**124**

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FLOOD HYDROLOGY: SCS METHOD

Project No		Calculated by:	L Garlick
Project Title	Algoa Recon	Date:	12/09/2018
Place			
River	Lower Coerney		Input Data
			Calculated

INPUT DATA: Catchment characteristics

Total catchment area		33.6	km^2
Effective catchment area		33.60	km^2
Length of watercourse (L)		9.83	km
Elevation: 1.0 L	9.83		m amsl
0.85 L	8.3555	200.24	m amsl
0.1 L	0.983	91.286	m amsl
0.0 L	0		m amsl
Watercourse slope (10/85)		0.0148	m/m

		Deg	Min	Sec
Location	Lat	33	26	54
	Long	25	37	33
Approx. location of centroid	Lat			
	Long			

Catchment slope: where; y

$$y = \frac{M \cdot N}{A \cdot 10000}$$

y	6.55	Average slope (%)
	0.0655	Average slope (m/m)
M		Total length of contours (m)
N		Contour interval (m)
A		Effective area (km2)

INPUT DATA: Rainfall

Mean Annual Precipitation 480 mm ARF = 100.0%

1-day point design rainfall (mm) / 1-day areal design rainfall (mm)

Recurrence Interval years	AEP (%)	Storm duration 1-day	24 hours	1-day areal design rainfall (mm)
2yr	50%	51	57	57
5yr	20%	76	84	84
10yr	10%	95	105	105
20yr	5%	116	129	129
50yr	2%	146	162	162
100yr	1%	172	191	191
200yr	0.5%	202	224	224
Probable Maximum Precipitation		600	666	666

INPUT DATA: Landuse and Runoff Curve Number (CN) coefficients

Hydrological Soil Group: #N/A Obtained from Soils Group map

Note: can adjust these generalised groups upward or downward one group to allow for soil depths or topographical position

Land Use: Curve Numbers can be found in the latest Drainage Manual

Type	Proportion	A	A/B	B	B/C	C	C/D	D	CN	
Veld - Fair	0%								**0**	0.00
Garden Crops	0%								**0**	0.00
Fallow	0%								**0**	0.00
Natural Forest	70%			55					**55**	38.50
Veld - Poor	30%			79					**79**	23.70
	100%								Curve Number:	**62.20**
									PMF Curve Number:	**90.00**

CALCULATIONS: Time of concentration

Percentage of longest watercourse:

Note: Overland flow should usually only be allowed for in catchments <5km2 and areas flatter than 5%

Overland flow:	0%
Channel flow:	100%
	100%

Overland flow (over fairly even and flat ground)

$$T_c = 0.604 \cdot \left(\frac{r \cdot L}{S^{0.5}}\right)^{0.467}$$

where;			
Tc	Time of concentration (overland)	8.60	hours
r	Roughness coefficient	Sparse grass over fairly rough surface	
		0.3	
L	Hydraulic length of catchment	9.83	km
S	Overland slope	0.0001	m/m

Channel flow

$$T_c = \left(\frac{0.87 \cdot L^2}{1000 \cdot S_{av}}\right)^{0.385}$$

where;			
Tc	Time of concentration (channel)	1.95	hours
L	Hydraulic length of catchment	9.83	km
S	Average channel slope	0.0148	m/m

Total time of concentration 1.95 hours

CALCULATIONS: Catchment response

For a well vegetated catchment use the South African Lag Time equation. For more arid conditions use the American SCS method

Catchment type: Well vegetated ------> Lag time: 2.57 hours

154.3 minutes

Storm Type: 1

Areal Reduction Factor 100.0%

Convert 2-year 1-day rainfall to 30min rainfall intensity for lagtime equation

	2-year 1-day rainfall	56.6	mm
	Aerial rainfall	56.6	mm
	30min factor	0.433	
	2-year 30min rainfall	24.5	mm/hr
Soil retention	normal	154.4	mm
	PMF	28.2	mm
Initial abstraction	normal	15.4	mm
	PMF	2.8	mm

$$S = \frac{25400}{CN} - 254$$

$$I_a = 0.1 \cdot S$$

Lagtime

SA:

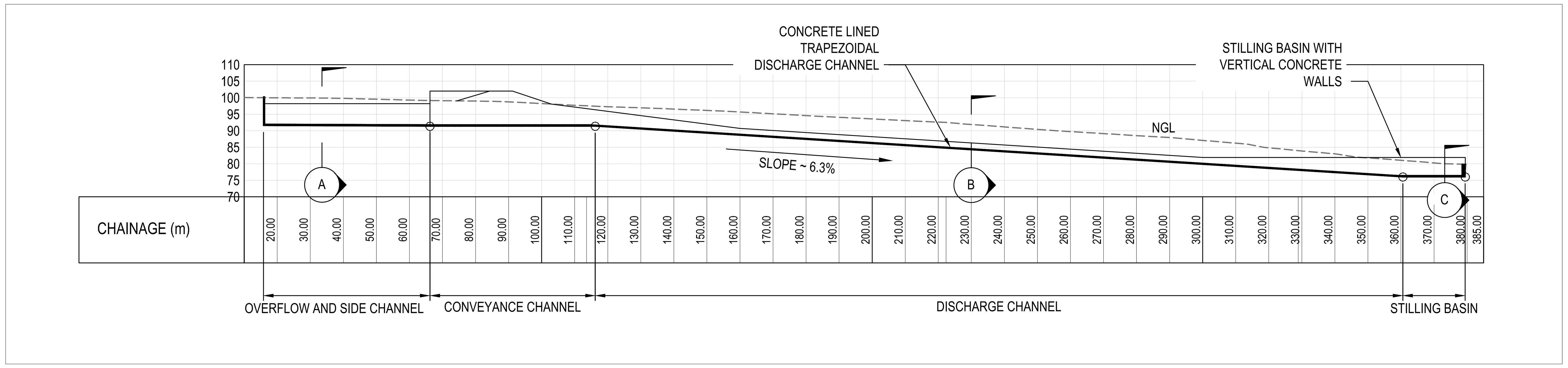
$$L = \frac{A^{0.35} MAP^{1.1}}{41.67 \cdot y^{0.3} {I_{30min}}^{0.87}}$$

where;		
A	33.60	km2
MAP	480	mm
y	6.55	%
I_{30}	24.5	mm/hr
L	2.57	hrs

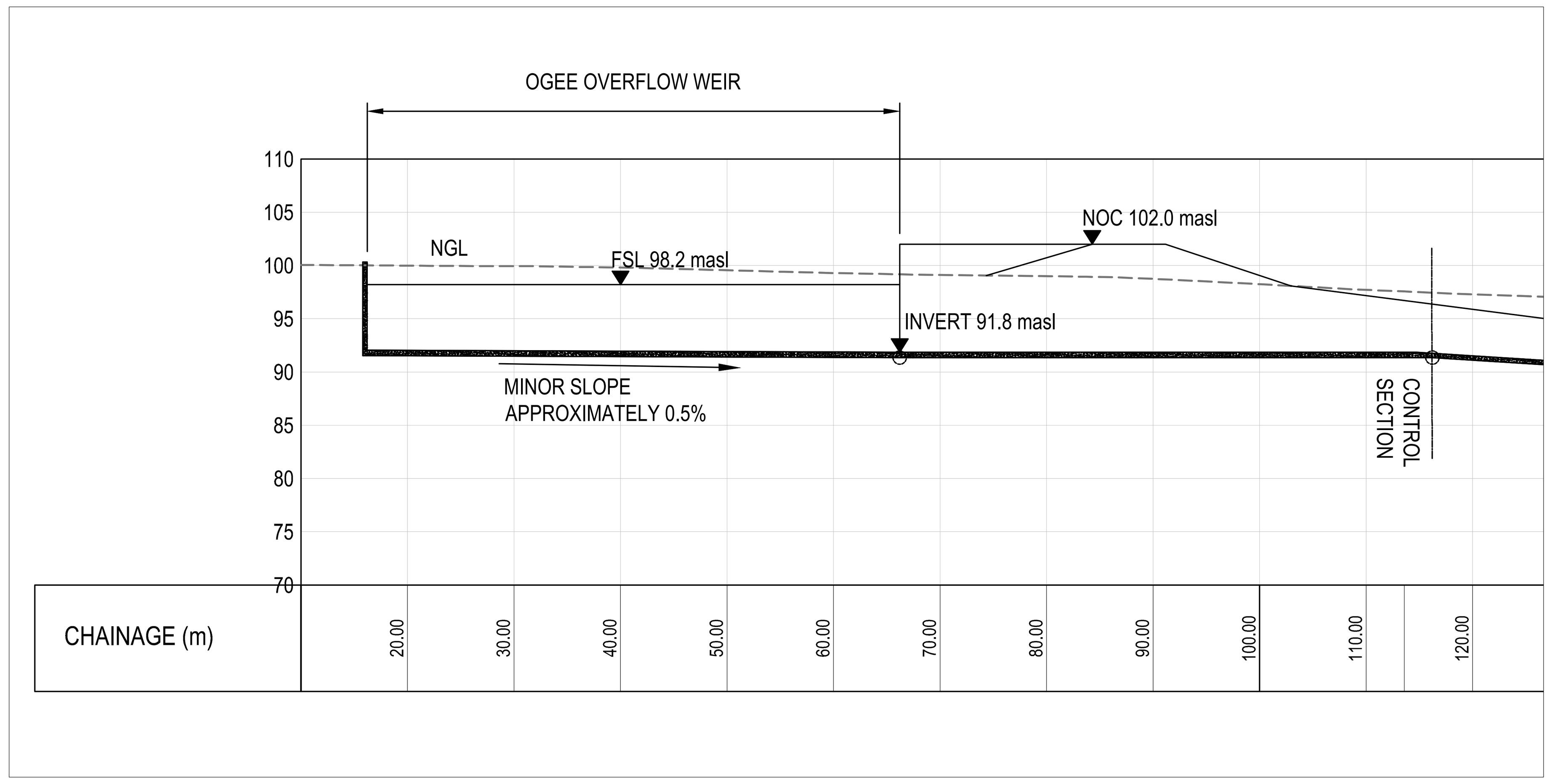
USA:

$$L = \frac{l^{0.8}(S + 25.4)^{0.7}}{7069 \cdot y^{0.5}}$$

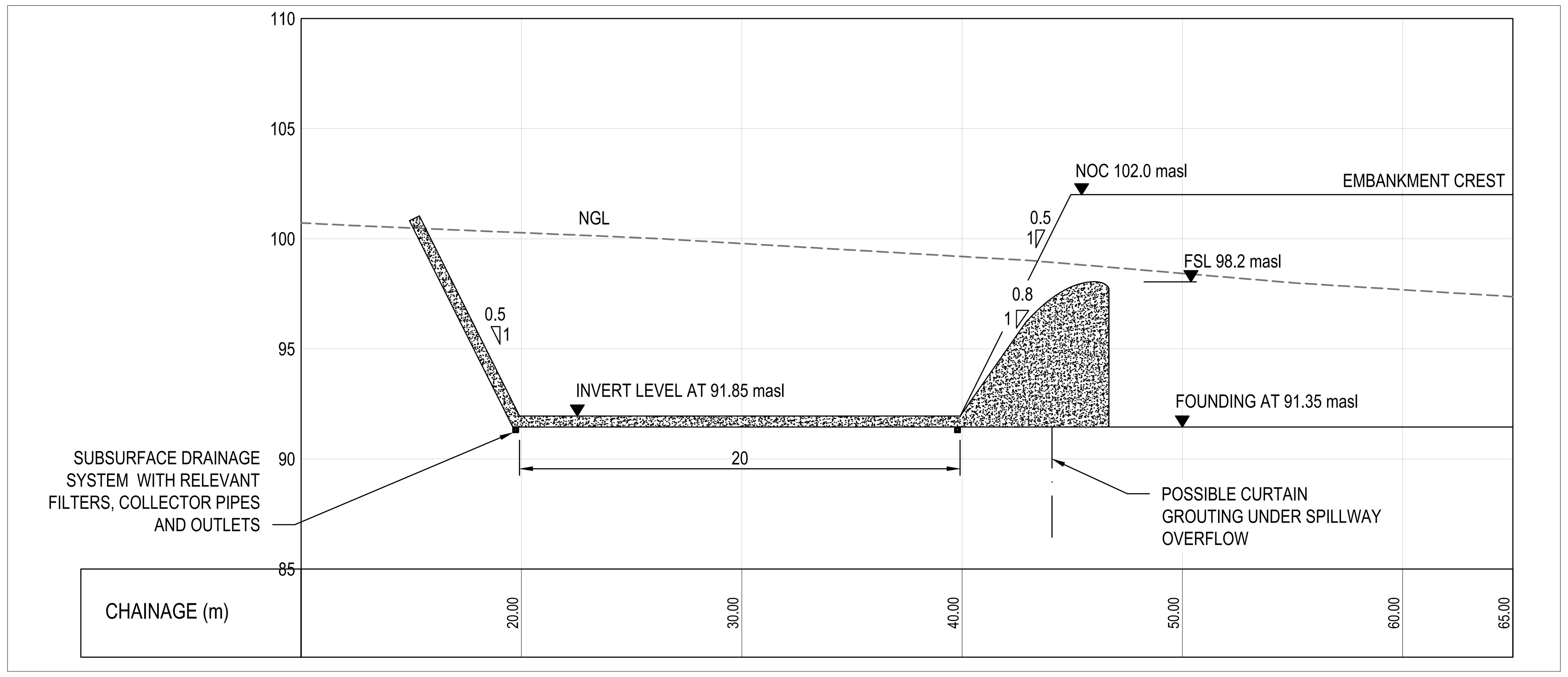
where;		
l	9 830	m
S	154.4	mm
y	6.55	%
L	3.27	hrs



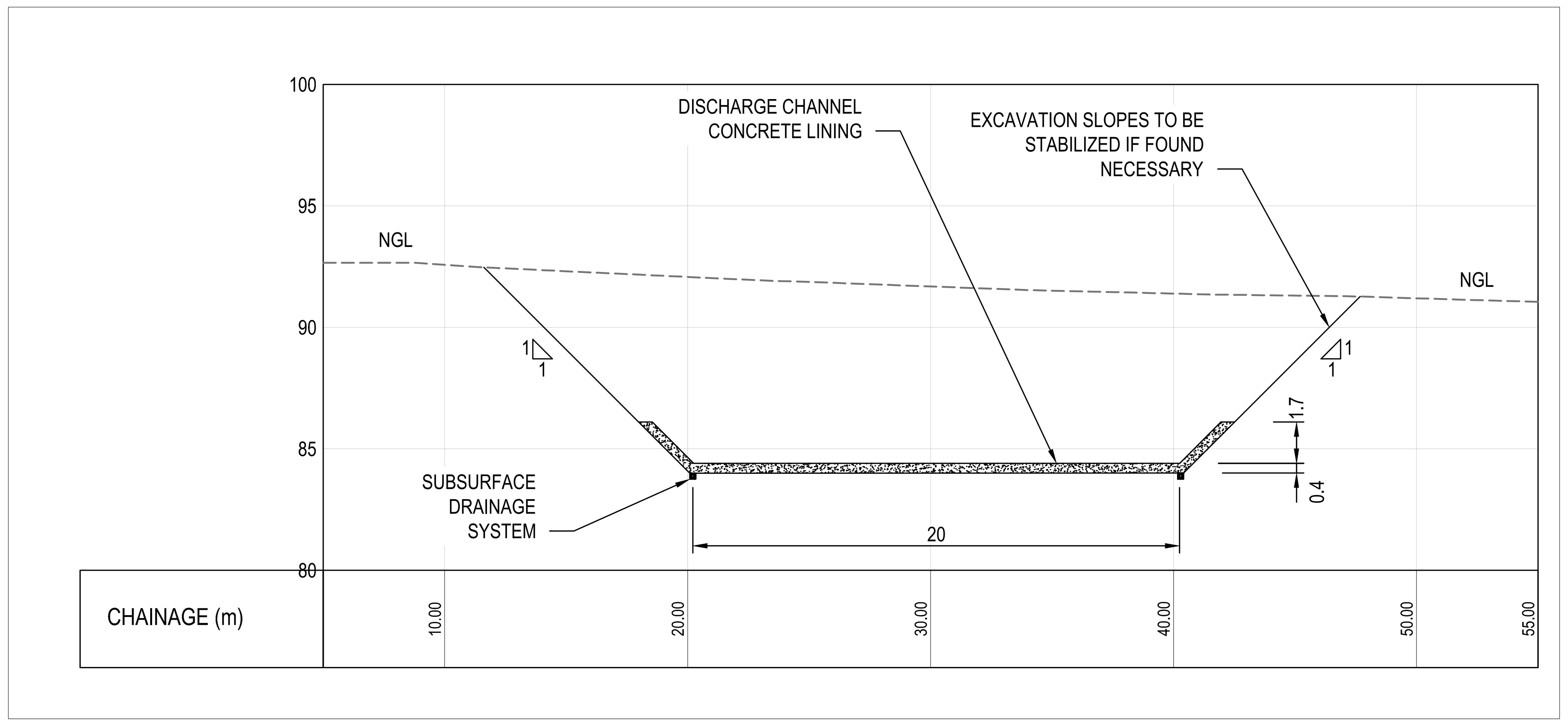
LONG SECTION THROUGH SPILLWAY
1:1000



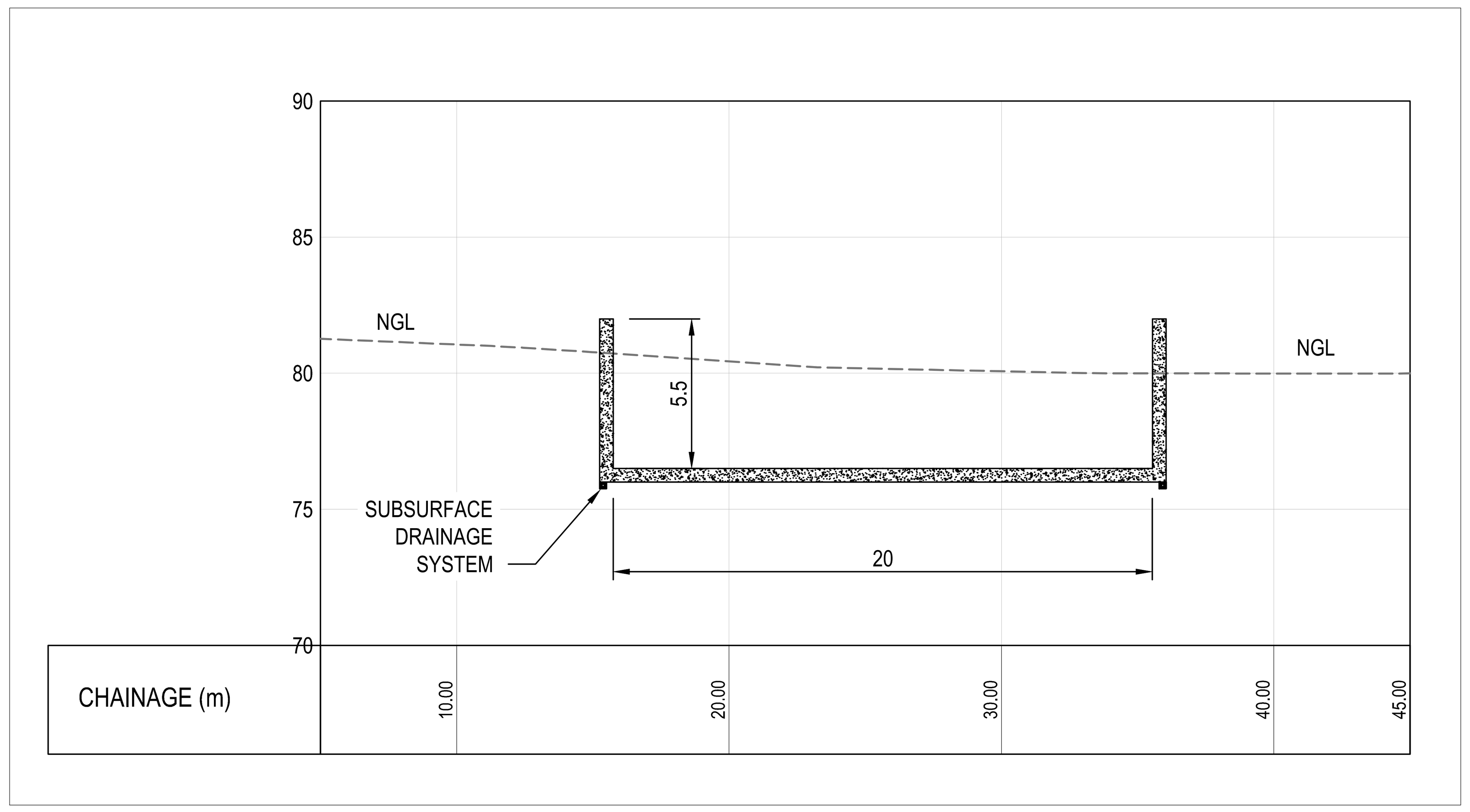
SECTION AT SPILLWAY OVERFLOW
1:500



DETAIL A SPILLWAY SIDE CHANNEL SECTION
1:200



DETAIL B SPILLWAY DISCHARGE CHANNEL SECTION
1:200



DETAIL C SPILLWAY STILLING BASIN SECTION
1:200

NOTES

A. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.
B. ALL LEVELS IN METRES ABOVE SEA LEVEL (masl).
C. DAM EMBANKMENT TO BE CONSTRUCTED 2% HIGHER FOR SETTLEMENT ALLOWANCE I.E. NOC 102.0 masl + (2% x 20 m) = NOC 102.4 masl.
D. DAM CREST TO BE CONSTRUCTED WITH 2% CROSSFALL SLOPE TOWARDS DAM BASIN.
E. EXCAVATION DEPTH OF CUT-OFF TRENCH TO BE APPROVED BY ENGINEER ON SITE.
F. GROUTING DETAILS TO BE CONFIRMED.
G. SHOULD MATERIALS NOT BE AVAILABLE IN DAM BASIN OR ESSENTIAL EXCAVATIONS THEY ARE TO BE IMPORTED FROM COMMERCIAL SOURCES.
H. FOR OUTLET WORKS & MISCELLANEOUS DETAILS REFER TO DRAWING 112546-0000-DRG-CC-003.
I. FOR SPILLWAY DISCHARGE CHANNEL DETAILS REFER TO DRAWING 112546-0000-DRG-CC-004.
J. FOR OUTLET TOWER DETAILS REFER TO DRAWING 112546-0000-DRG-CC-005.

ABBREVIATIONS

NGL - NATURAL GROUND LEVEL
FSL - FULL SUPPLY LEVEL
NOC - NON-OVERSPILL CREST

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REV	DATE	REVISION DETAILS	APPROVED
A	06/02/20	FOR INFORMATION	E. VAN DER BERG
B	06/03/20	FOR INFORMATION	E. VAN DER BERG

SCALE	SIZE	
AS SHOWN	A1	FOR INFORMATION

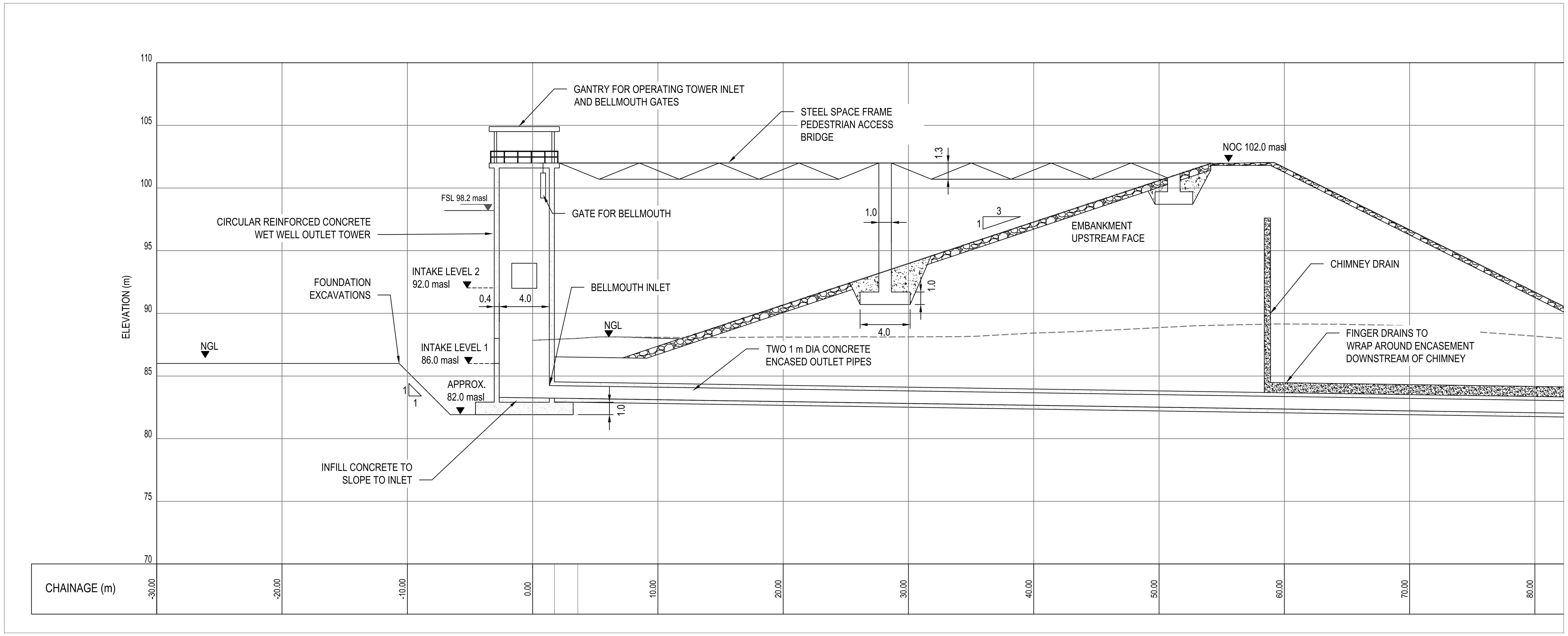
DRAWN: A. KNOX
DESIGNED: O. HUMAN
CHECKED: F. DENEYS
APPROVED: E. VAN DER BERG — DATE — ECSA 920455

PROJECT
SUPPORT OF THE WATER RECONCILIATION STRATEGY OF THE ALGOA WATER SUPPLY SYSTEM

TITLE
LOWER COERNEY BALANCING DAM SPILLWAY SECTION

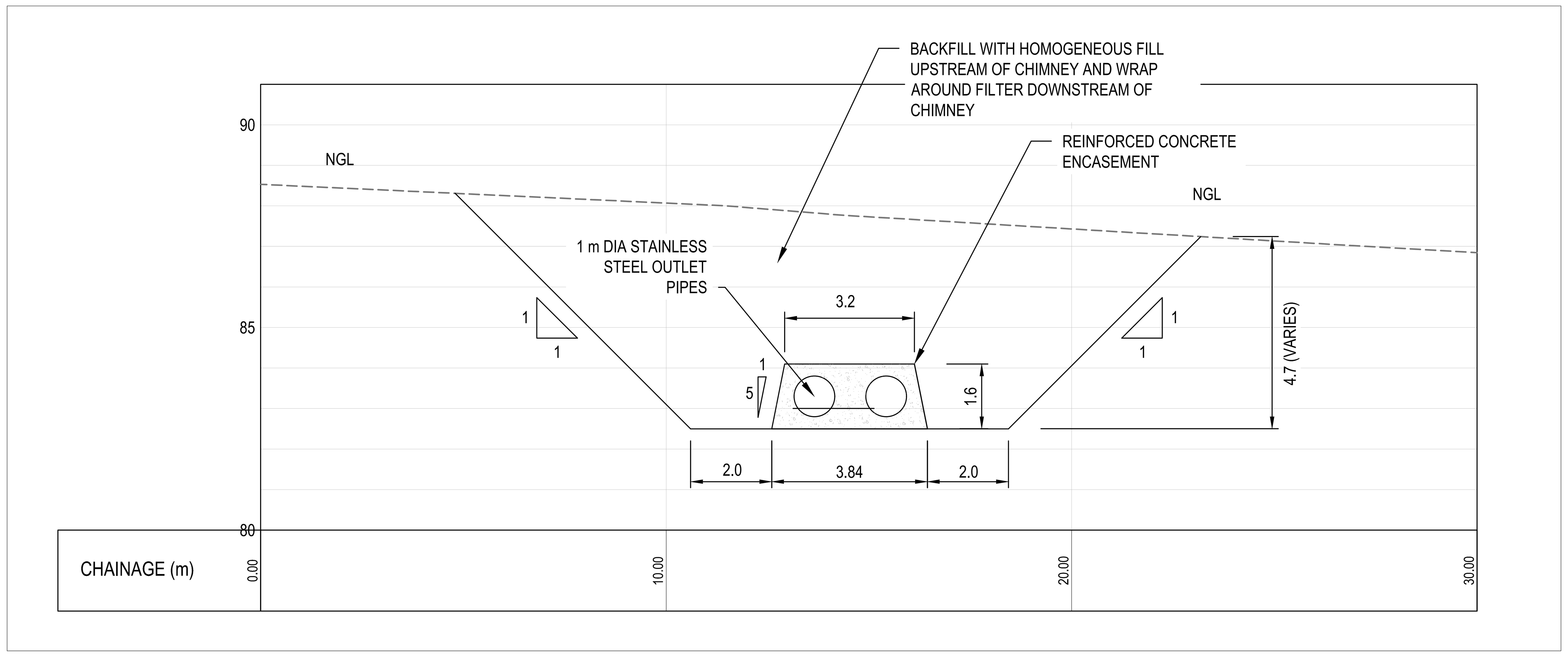
DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
112546	0000	DRG	CC	004	B



ELEVATION VIEW OF OUTLET TOWER

1:200



SECTION THROUGH OUTLET PIPE ENCASEMENT

1:100

NOTES

A. ALL DIMENSIONS IN METRES UNLESS OTHERWISE SHOWN.
B. APPROXIMATE PIPE INVERT LEVEL AT INTAKE 83.2 masl.
C. ALL LEVELS IN METRES ABOVE SEA LEVEL (masl).
D. DAM EMBANKMENT TO BE CONSTRUCTED 2% HIGHER FOR SETTLEMENT ALLOWANCE I.E. NOC 102.0 masl + (2% x 20 m) = NOC 102.4 masl.
E. DAM CREST TO BE CONSTRUCTED WITH 2% CROSSFALL SLOPE TOWARDS DAM BASIN.
F. EXCAVATION DEPTH OF CUT-OFF TRENCH TO BE APPROVED BY ENGINEER ON SITE.
G. GROUTING DETAILS TO BE CONFIRMED.
H. SHOULD MATERIALS NOT BE AVAILABLE IN DAM BASIN OR ESSENTIAL EXCAVATIONS THEY ARE TO BE IMPORTED FROM COMMERCIAL SOURCES.
I. FOR OUTLET WORKS & MISCELLANEOUS DETAILS REFER TO DRAWING 112546-0000-DRG-CC-003.
J. FOR SPILLWAY DISCHARGE CHANNEL DETAILS REFER TO DRAWING 112546-0000-DRG-CC-004.
K. FOR OUTLET TOWER DETAILS REFER TO DRAWING 112546-0000-DRG-CC-005.

ABBREVIATIONS

NGL - NATURAL GROUND LEVEL
FSL - FULL SUPPLY LEVEL
NOC - NON-OVERSPILL CREST

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REV	DATE	REVISION DETAILS	APPROVED
A	06/02/20	FOR INFORMATION	E. VAN DER BERG
B	06/03/20	FOR INFORMATION	E. VAN DER BERG

SCALE: AS SHOWN
SIZE: A1
FOR INFORMATION
DRAWN: A. KNOX
DESIGNED: O. HUMAN
CHECKED: F. DENEYS
APPROVED: E. VAN DER BERG
DATE
ECSA 920455

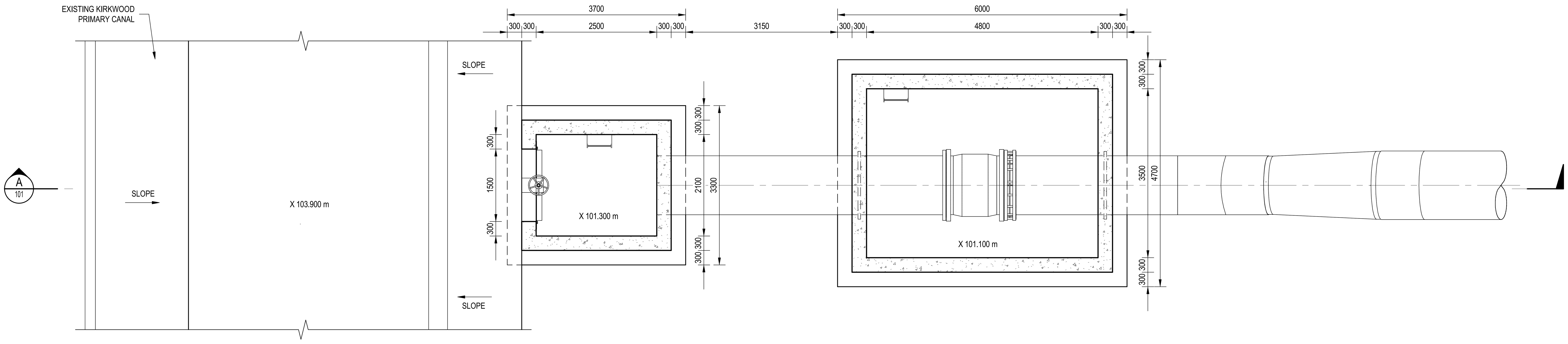
PROJECT
SUPPORT OF THE WATER RECONCILIATION STRATEGY OF THE ALGOA WATER SUPPLY SYSTEM

TITLE
LOWER COERNEY BALANCING DAM OUTLET TOWER

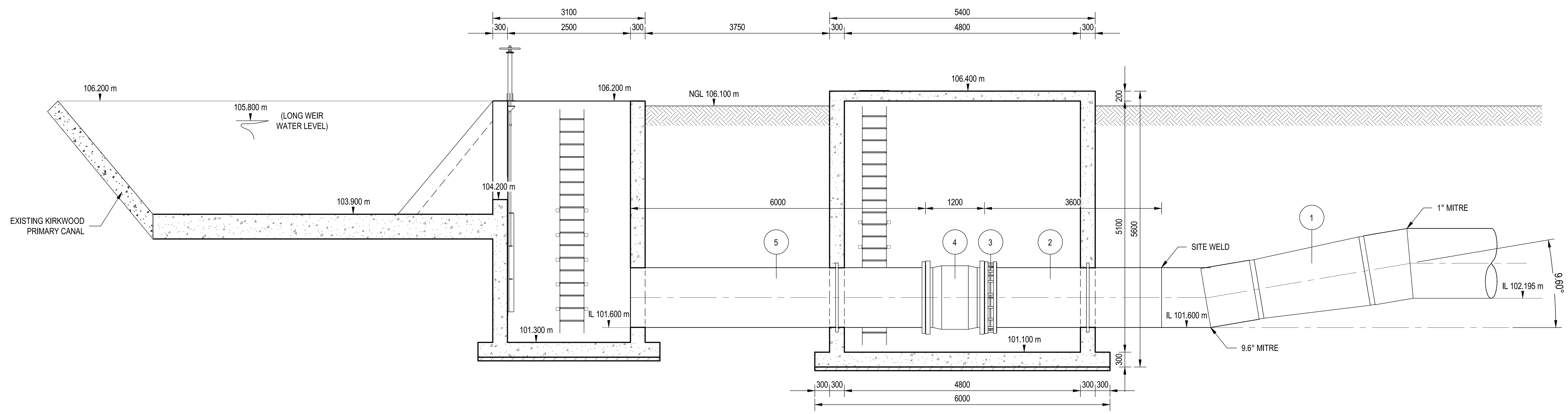
DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
112546	0000	DRG	CC	005	B

Appendix D: Conveyance Infrastructure Plans and Drawings



SECTIONAL PLAN VIEW
1 : 50



SECTION A
1 : 50 (101)

MATERIAL LIST		
ITEM	QTY	DESCRIPTION
1	1	DN 1400 x DN 1200 PLAIN ENDED REDUCER; 2375mm F/F
2	1	DN 1200 PUDDLE PIPE; FLANGED ONE END; 3600mm F/F
3	1	DN 1200 RESTRAINED FLANGE ADAPTOR
4	1	DN 1200 FLANGED ELECTRO-MAGNETIC FLOW METER
5	1	DN 1200 PUDDLE PIPE; FLANGED ONE END; 6000mm F/F

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REV	DATE	REVISION DETAILS	APPROVED
A	02/2020	FOR INFORMATION	

SCALE: AS INDICATED
SIZE: A1
FOR INFORMATION
DRAWN: B. MCCAROGHER
DESIGNED: M. WAHL
REVIEWED: S. KLEYNHANS
APPROVED
DATE

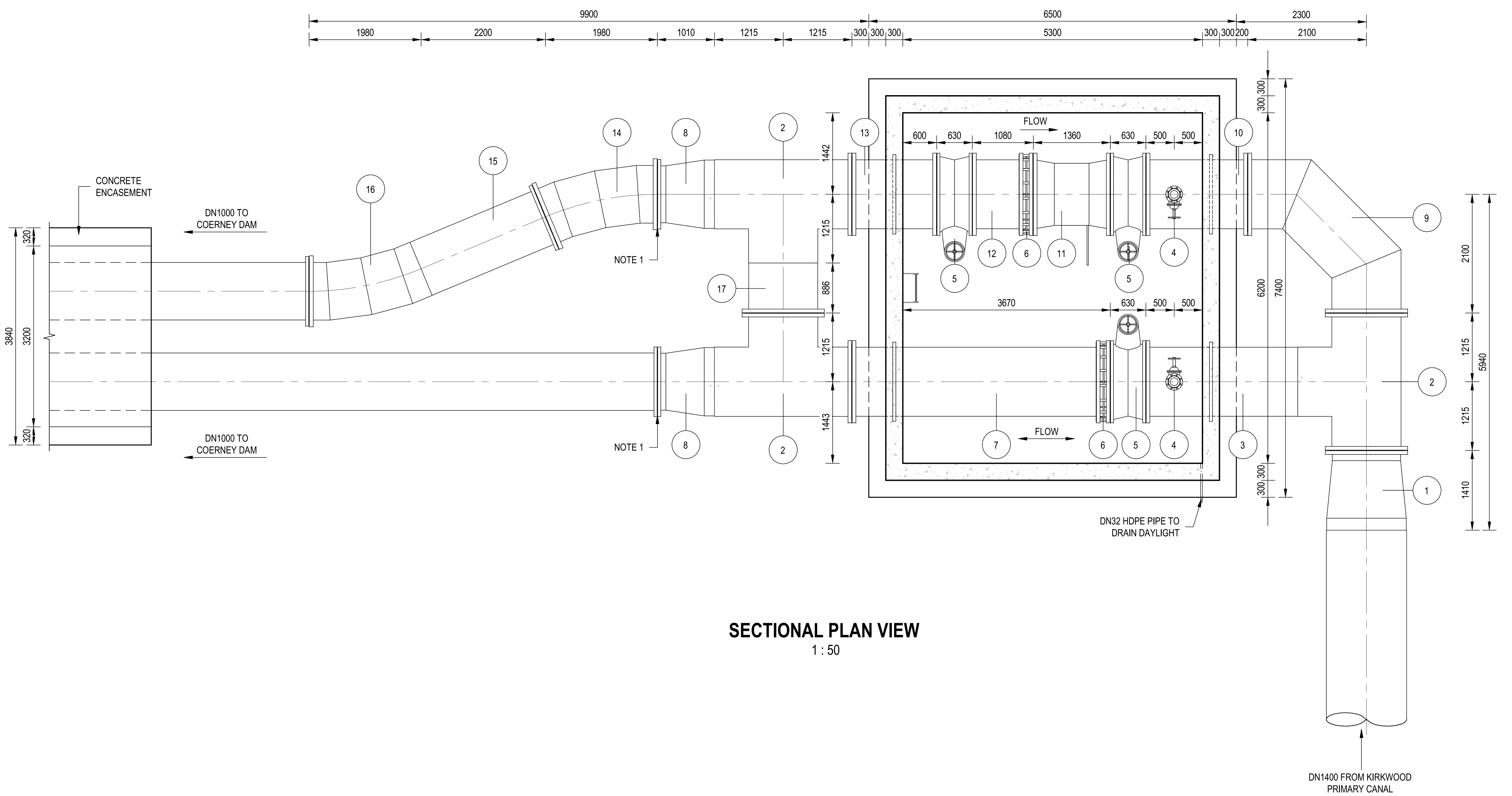
PROJECT
SUPPORT OF THE WATER RECONCILIATION STRATEGY OF THE ALGOA WATER SUPPLY SYSTEM

TITLE
OFFTAKE AT KIRKWOOD PRIMARY CANAL

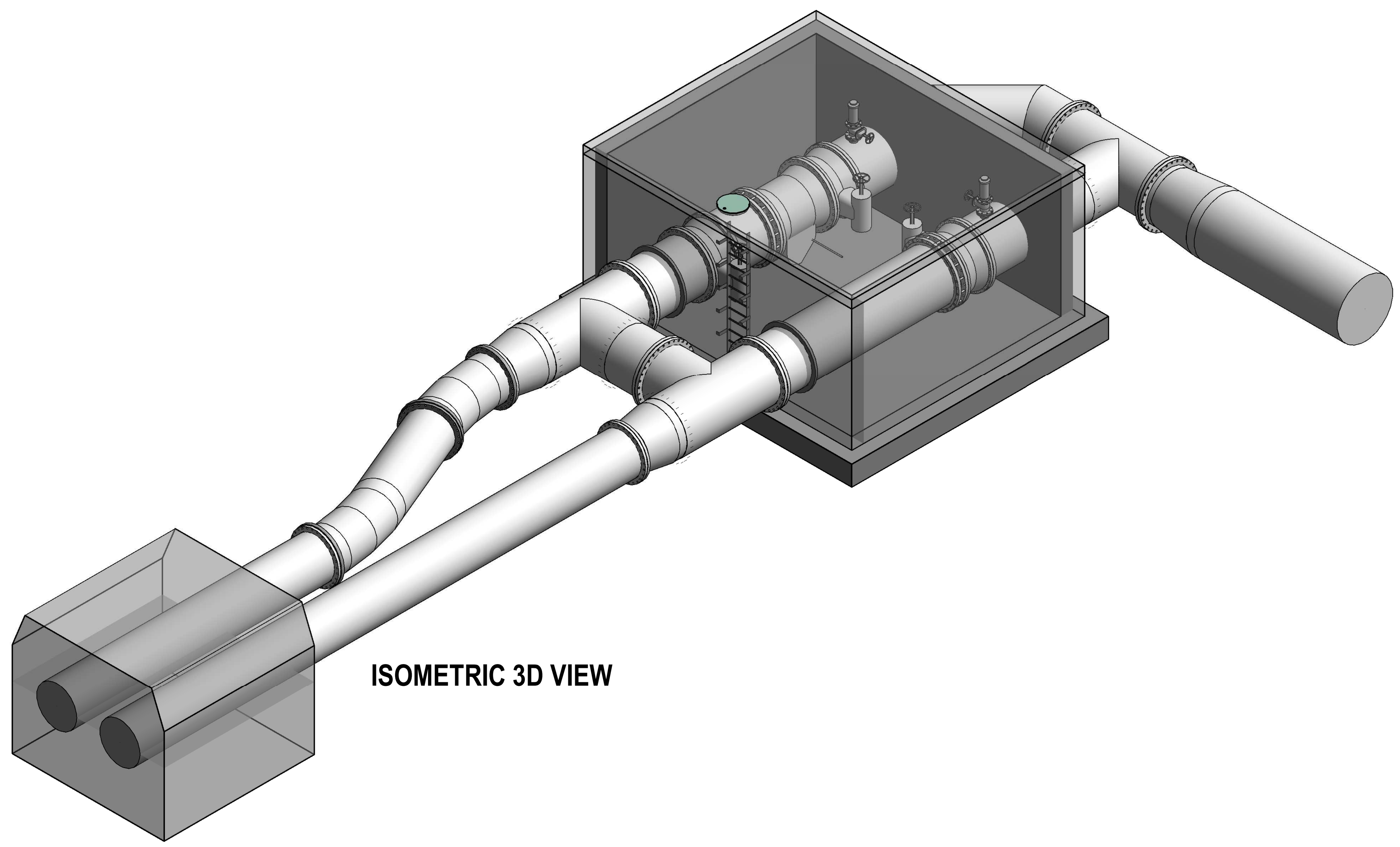
DRAWING NUMBER

PROJECT No	WBS	TYPE	DISC	NUMBER	REV
112546	0000	DRG	CC	101	A

NOTES:
1. FLANGES LOCATED OUTSIDE CHAMBER CAN BE REPLACED WITH SITE WELDS.



SECTIONAL PLAN VIEW
1 : 50



ISOMETRIC 3D VIEW

MATERIALS LIST		
ITEM	QTY	DESCRIPTION
1	1	DN 1400 x DN 1200 REDUCER; FLANGED ONE END
2	3	DN 1200 FLANGED EQUAL TEE; ONE END PLAIN
3	1	DN 1200 PUDDLE PIPE; FLANGED ONE END; 2685mm F/F
4	2	DN 150 STUB; 1070mm C/F; WITH DN 150 GATE VALVE AND DN 150 AIR VALVE
5	3	DN 1200 FLANGED BUTTERFLY VALVE
6	2	DN 1200 RESTRAINED FLANGE ADAPTOR
7	1	DN 1200 FLANGED PUDDLE PIPE; 4570mm F/F
8	2	DN 1200 x DN 1000 PLAIN ENDED REDUCER
9	1	DN 1200 PLAIN ENDED 90° BEND (1 x D)
10	1	DN 1200 PUDDLE PIPE; FLANGED ONE END; 1800mm F/F
11	1	DN 1200 FLANGED NON-RETURN VALVE
12	1	DN 1200 PIPE FLANGED ONE END; 1080mm F/F
13	1	DN 1200 FLANGED PUDDLE PIPE; 1500mm F/F
14	1	*DN 1000 FLANGED 22.7° BEND
15	1	DN 1000 PIPE; FLANGED ONE END; 2385mm F/F
16	1	*DN 1000 22.7° BEND; FLANGED ONE END
17	1	DN 1000 PIPE; FLANGED ONE END; 885mm F/F

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REV	DATE	REVISION DETAILS	APPROVED
A	02/2020	FOR INFORMATION	
B	03/2020	FOR INFORMATION	

SCALE: AS INDICATED
SIZE: A1
FOR INFORMATION
DRAWN: B. MCCAROGHER
DESIGNED: M. WAHL
REVIEWED: S. KLEYNHANS
APPROVED
DATE

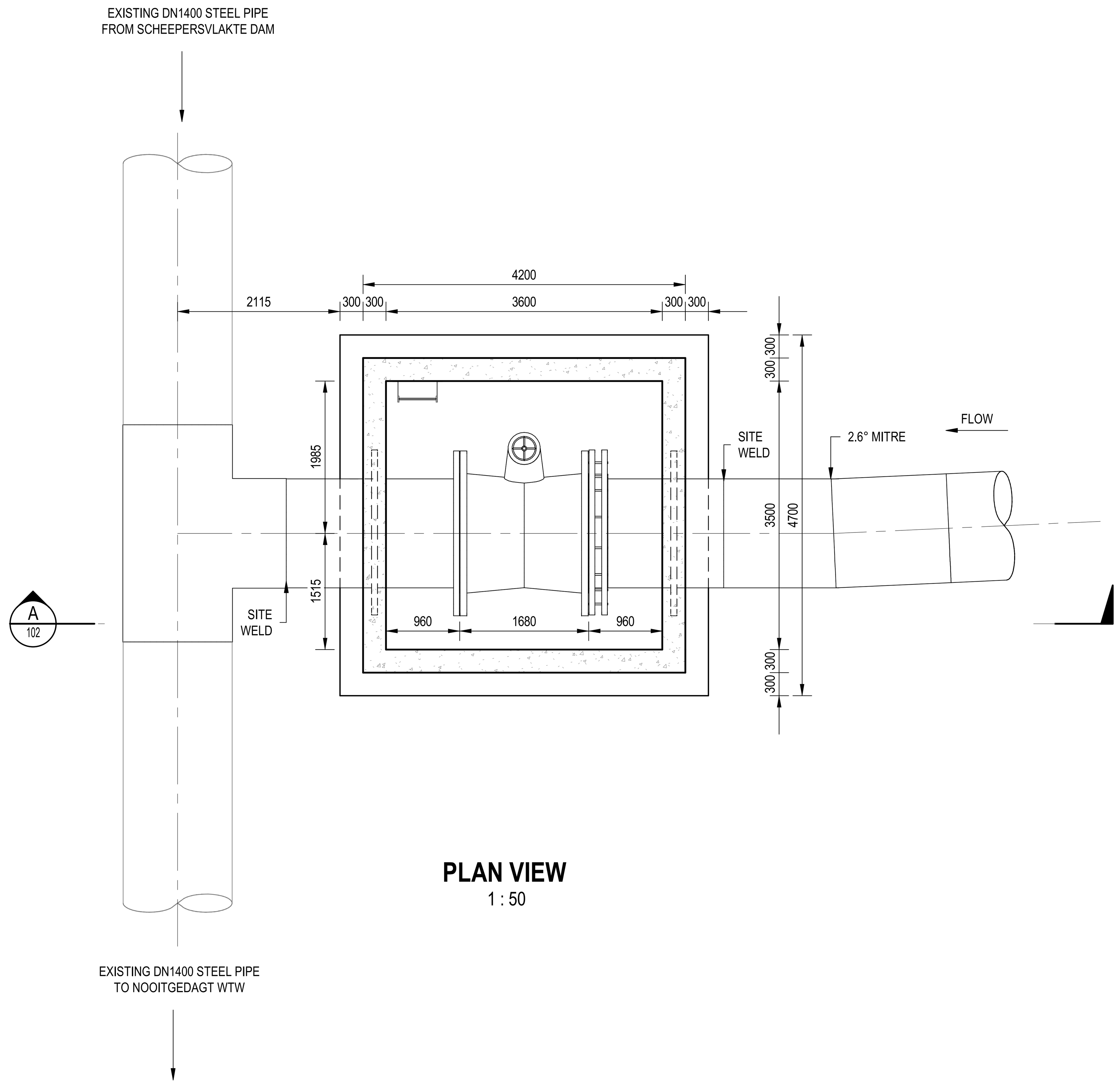
PROJECT
SUPPORT OF THE WATER RECONCILIATION STRATEGY OF THE ALGOA WATER SUPPLY SYSTEM

TITLE
LOWER COERNEY DAM INLET/OUTLET CHAMBER

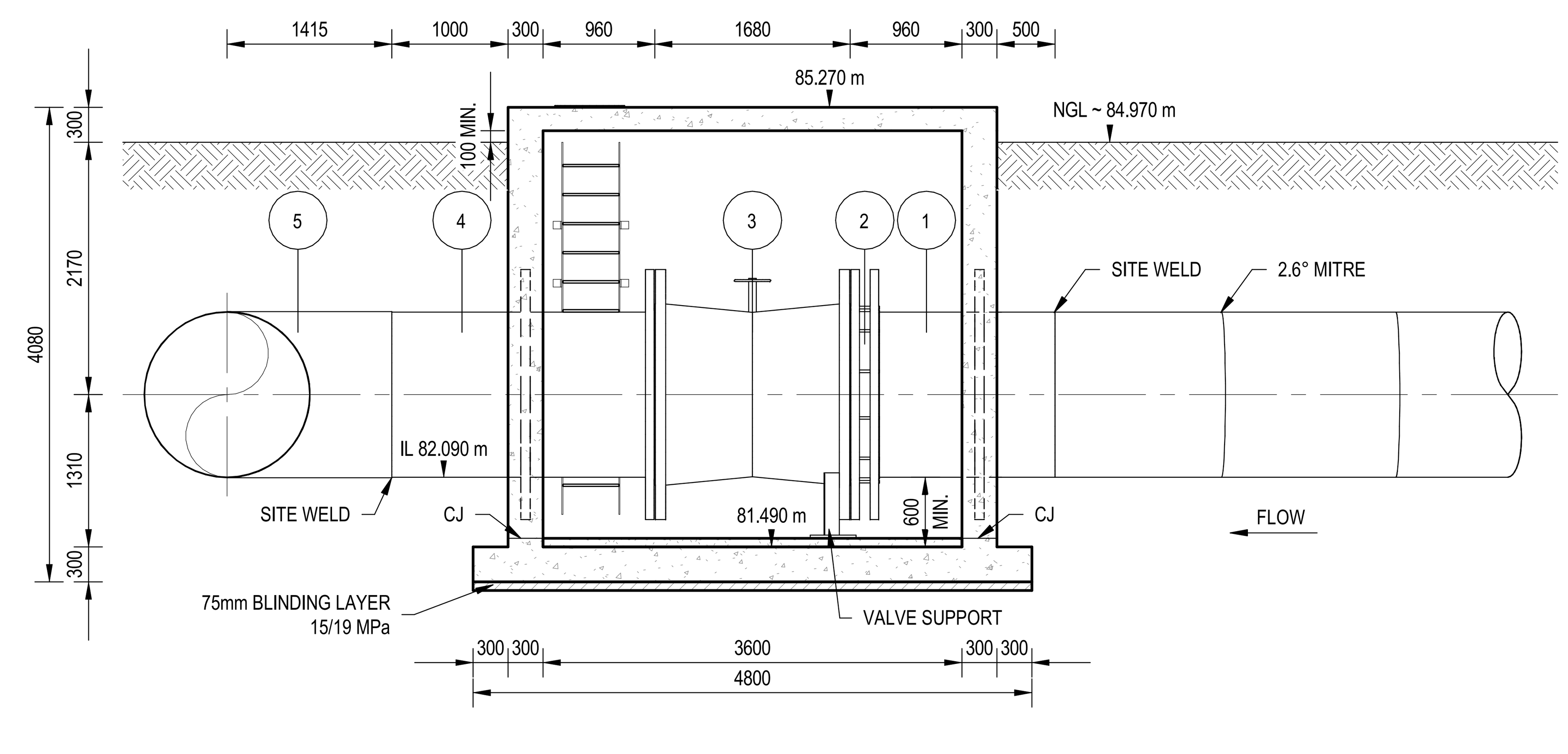
DRAWING NUMBER

PROJECT No	WBS	TYPE	DISC	NUMBER	REV
112546	0000	DRG	CC	100	B

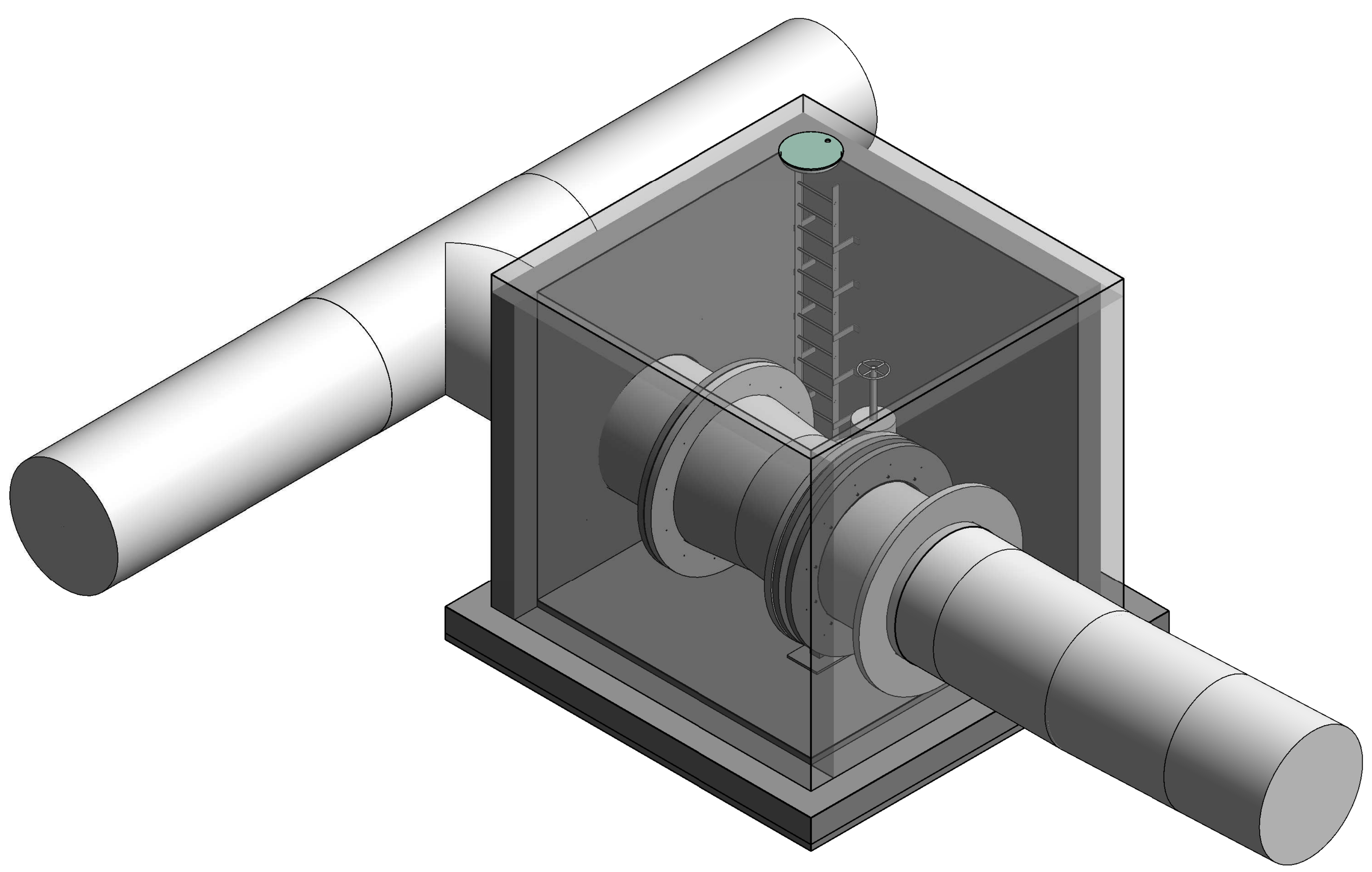
Plot Date: 3/4/2020 8:23:36 AM Office: ZAWAT Filename: P:\Projects\112546 Algoa Recon Support\03 Pr Delivery\12 Feasibility Design\04 Pipeline design\Drawings\Revit\112546-0000-DRG-CC-100.rvt



PLAN VIEW
1 : 50



SECTION A
1 : 50 (102)



3D VIEW
NTS

MATERIALS LIST

ITEM	QTY	DESCRIPTION
1	1	DN 1400 PLAIN ENDED PUDDLE PIPE; 1760mm F/F
2	1	DN 1400 RESTRAINED FLANGE ADAPTOR
3	1	DN 1400 FLANGED BUTTERFLY VALVE
4	1	DN 1400 PUDDLE PIPE; FLANGED ONE END; 2260mm F/F
5	1	DN 1400 PLAIN ENDED EQUAL TEE

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CLIENT

REV	DATE	REVISION DETAILS	APPROVED
A	02/2020	FOR INFORMATION	

SCALE: AS INDICATED
SIZE: A1
FOR INFORMATION

DRAWN: B. MCCAROGHER
DESIGNED: M. WAHL
REVIEWED: S. KLEYNHANS
APPROVED
DATE

PROJECT
SUPPORT OF THE WATER RECONCILIATION STRATEGY OF THE ALGOA WATER SUPPLY SYSTEM

TITLE
TIE-IN TO EXISTING NOOITGEDAGT SUPPLY PIPELINE

DRAWING NUMBER

PROJECT No	WBS	TYPE	DISC	NUMBER	REV
112546	0000	DRG	CC	102	A

Appendix E: Costing

EARTHFILL DAM COST MODEL

Spillway on left abutment

09-Mar-20

Proposed Lower Coerney : 4.7 Mm3 gross storage capacity

Lower Coerney Dam Costing v 18 - Spillway 50 m - Left abutment V4.xlsx

Max wall height (m)= 20 m

NOCL= RL 102.0 m

FSL= RL 98.2 m

Crest width (m)= 5 m

No	DESCRIPTION	UNIT	RATE Jan 20 Rand	QUANTITY	AMOUNT (Excl VAT) Rand
1	**Clearing**				
	(a) sparse	ha	25 000	0.0	0
	(b) bush	ha	35 000	2.2	75 951
	(c) trees	ha	55 000	2.2	119 352
2	**River diversion**	Sum	5 250 000		5 250 000
3	**Excavation**				
	(a) Bulk				
	(i) all materials	m³	80	136 674	10 933 943
	(ii) extra over for rock	m³	400	13 667	5 466 972
	(b) Confined				
	(i) all materials	m³	200	8 928	1 785 516
	(ii) extra over for rock	m³	600	893	535 655
	(c) Preparation of solum				
	(i) all materials	m²	35	36 201	1 267 024
	(ii) extra over for rock	m²	150	1 810	271 505
4	**Drilling & Grouting**				
	(a) Curtain grouting	m drill	1 500	376	563 423
	(b) Consolidation grouting	m drill	1 500	76	114 000
	(c) Slurry trench - fill	m3		0	0
5	**Embankment**				
	(a) Earthfill	m³	80	238 173	19 053 831
	(b) Filters	m³	700	5 096	3 567 383
	(c) Rip rap	m³	800	10 084	8 066 947
	(d) Overhaul beyond 1km (one way)	m³km	10	0	0
	(e) Toe drain	m³	600	4 745	2 847 240
	(f) Spillway channel protection with reno	m³	3 000	346	1 036 800
6	**Concrete Works**				
	(a) Formwork				
	(i) gang formed	m²	675	6 661	4 496 375
	(ii) intricate	m²	850	10 905	9 269 519
	(b) Concrete				
	(i) mass	m³	2 700	8 538	23 052 015
	(ii) structural	m³	3 000	900	2 699 747
	(c) Reinforcing	t	16 500	791	13 051 707
7	**Mechanical Items**				
	(a) Valves & gates	No	2 000 000	5	10 000 000
	(b) Cranes & hoists	Sum	5 000 000		5 000 000
	(c) Structural steelwork	Sum	2 000 000		2 000 000
	(d) Outlet pipe (SS316)	m	15 000	368	5 524 920
	SUB-TOTAL				136 049 825

Proposed Lower Coerney : 4.7 Mm3 gross storage capacity

No	DESCRIPTION	UNIT	RATE Jan 20 Rand	QUANTITY	AMOUNT (Excl VAT) Rand
8	**Miscellaneous** (% of 1-9)	%	6		8 162 990
	SUB TOTAL A				144 212 815
9	**Preliminary & General** (% of sub-total A)	%	30		43 263 844
10	**Preliminary works**				
	(a) Access road (construction and maintenance, 3yr)	km	1 500 000	1.50	2 250 000
	(b) Electrical supply to site	Sum			100 000
	SUB TOTAL B				189 826 659
11	**Contingencies** (% of sub total B)	%	25		47 456 665
	SUB TOTAL C				237 284 000
12	**Professional fees** (% of sub total C)	%	10		23 728 400
	SUB TOTAL D				261 013 000
	VAT 15%				39 151 950
	TOTAL CONSTRUCTION COST				**R 300 164 950**
13	**Cost of relocations**	Sum	0		0
14	**Cost of land acquisition**				
	(a) Irrigated	ha	1 000 000	0.0	0
	(b) Dryland farming	ha	600 000	0.0	0
	(c) Undeveloped	ha	250 000	85.0	21 250 000
	(d) Homesteads	No	0		0
	TOTAL PROJECT COST (as at Jan 2020)				**R 321 414 950**
	(Rounded to nearest R 100 000)				**R 321 500 000**

CONVEYANCE INFRASTRUCTURE COST

18-Feb-20

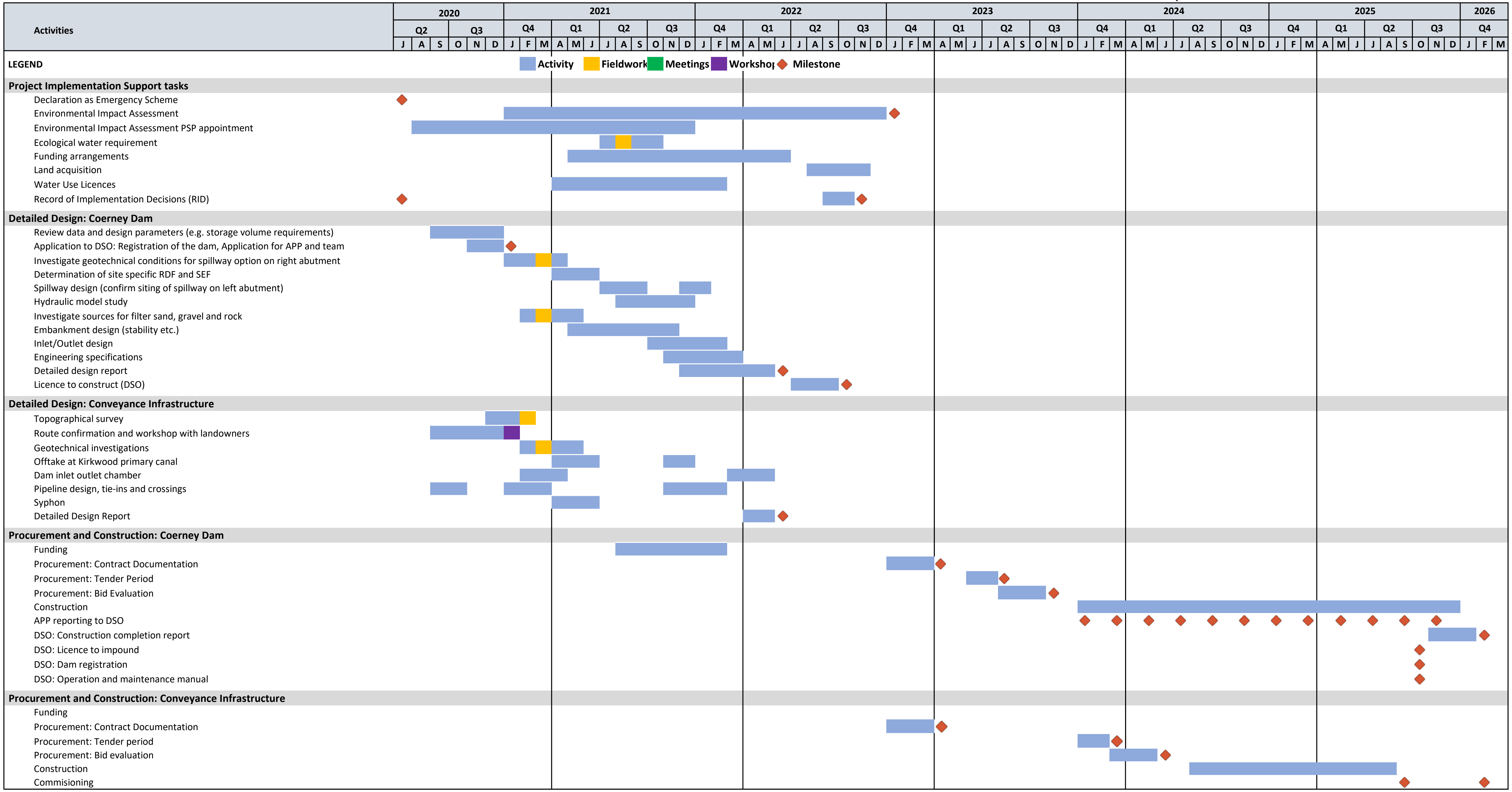
No	DESCRIPTION	UNIT	RATE Jan 20 Rand	QUANTITY	AMOUNT (Excl VAT) Rand
1	**Steel pipelines**				
	a) DN 1000	m	12,000	10	120,000
	b) DN 1400	m	19,200	1700	32,640,000
2	**Tie-in to Nooitgedagt**				
	a) DN1400 Butterfly valve	No	300,000	1	300,000
	b) DN1400 Equal tee	No	250,000	1	250,000
	c) DN1400 Restrained Flange Adaptor	No	50,000	1	50,000
	d) DN1400 Puddle Pipe Plain Ended, 1760	No	80,000	1	80,000
	e) DN1400 Puddle Pipe Flanged one end, 2260	No	90,000	1	90,000
	f) Cut and remove existing, plus joint repairs	Sum	150,000		150,000
	g) Chamber (28m3)	Sum	156,800		156,800
3	**Lower Coerney Dam Inlet/Outlet Chamber**				
	a) DN 1200 90 Bend	No	115,000	1	115,000
	b) DN 1200 22.5 Bend	No	80,500	2	161,000
	c) DN 1200 NRV - PN 16	No	2,400,000	1	2,400,000
	d) DN 1200 Butterfly valve	No	250,000	3	750,000
	e) Flanges	No	80,000	10	800,000
	f) DN 150 AV	No	13,600	2	27,200
	g) DN 150 RSV	No	4,650	2	9,300
	h) DN 1400 x DN1200 Reducer	No	150,000	1	150,000
	i) DN 1200 x DN1000 Reducer	No	130,000	2	260,000
	j) DN 1200 Restrained Flange Adaptor	No	40,000	2	80,000
	k) DN 1200 Flanged Equal tee	No	187,500	3	562,500
	l) DN 1200 Puddle pipe, 2685	No	60,000	1	60,000
	m) DN 1200 Puddle pipe, 4570	No	75,000	1	75,000
	n) DN 1200 Puddle pipe, 1800	No	55,000	1	55,000
	o) DN 1000 Puddle pipe, 2385	No	50,000	1	50,000
	p) DN 1000 Puddle pipe, 885	No	45,000	1	45,000
	q) Chamber (50m3)	Sum	700,000		700,000
4	**Middle Addo Canal Crossing**				
	a) DN 1400 45° Bends	No	103,500	4	414,000
	b) Concrete plinths	No	56,000	5	280,000
	c) DN 150 AV	No	13,600	1	13,600
	d) DN 150 RSV	No	4,650	1	4,650
	e) Support brackets, etc.	Sum	100,000		100,000
5	**Syphon Sundays River**				
	a) DN1400	m	19,200	110	2,112,000
	b) Concrete Encasement	m^3	2,800	600	1,680,000
	c) Chambers	No	500,000	2	1,000,000
	d) DN 150 AV	No	13,600	1	13,600
	e) DN 150 RSV	No	4,650	1	4,650
	f) DN 200 SV	No	15,000	1	15,000
	g) DN 1400 x DN 200 Tee	No	60,000	1	60,000
	h) Spare DN 1400 Pipes	m	19,200	60	1,152,000
	i) Allowance for river diversion / coffer dam	Sum	1,500,000		1,500,000
	j) DN 1400 isolating valves	No	300,000	4	1,200,000
	k) DN 1400 x DN 1400 equal tee	No	250,000	2	500,000
	l) Cut into existing pipe and repair	No	150,000	2	300,000
	m) Bends	Sum	100,000		100,000

No	DESCRIPTION	UNIT	RATE Jan 20 Rand	QUANTITY	AMOUNT (Excl VAT) Rand
6	**Kirkwood Canal**				
	a) Weir Chamber (20m3)	Sum	224,000		224,000
	b) Flowmeter Chamber (41m3)	Sum	459,200		459,200
	c) DN 1200 flowmeter	No	300,000	1	300,000
	d) DN 1200 restrained flange adaptor	No	40,000	1	40,000
	e) Sluice Gate	No	400,000	1	400,000
	f) DN 1400 x DN 1200 Reducer	No	150,000	1	150,000
	g) DN 1200 Puddle pipe, flanged one end, 3600	No	100,000	1	100,000
	h) Break and repair existing canal	Sum	80,000		80,000
	i) DN 1200 Puddle pipe, flanged one end, 6000	No	120,000	1	120,000
	SUB-TOTAL A				52,459,500
	Preliminary & General (% of sub-total A)	%	30		15,737,850
	SUB TOTAL B				68,197,350
	Contingencies (% of sub total B)	%	10		6,819,735
	SUB TOTAL C				75,017,085
	Professional fees (% of sub total C)	%	10		7,501,709
	SUB TOTAL D				82,518,794
	VAT 15%				12,377,819
	TOTAL CONSTRUCTION COST				**R 94,896,613**
	(Rounded to nearest R 100 000)				**R 94,900,000**

Appendix F: Implementation Programme

Water Reconciliation Strategy for the Algoa Water Supply System

IMPLEMENTATION PROGRAMME - Implementation support, detailed design and construction of the Coerney Dam and conveyance infrastructure



Abbreviations:

APP	Approved Professional Person
DSO	Dam Safety Office
RDF	Recommended Design Flood
SEF	Safety Evaluation Flood

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