water & sanitation

Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

Feasibility-level Engineering Design - Balancing Dam Sub-Report

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

March 2020

Revision: 2

Reference: 112546

Document control record

Document prepared by:

Aurecon South Africa (Pty) Ltd
1977/003711/07
Aurecon Centre
1 Century City Drive
Waterford Precinct
Century City
Cape Town
7441
PO Box 494
Cape Town
8000
South Africa

T +27 21 526 9400
F +27 21 526 9500
E capetown@aurecongroup.com
W aurecongroup.com

A person using Aurecon documents or data accepts the risk of:

a) Using the documents or data in electronic form without requesting and checking them for accuracy against the original hard copy version.
b) Using the documents or data for any purpose not agreed to in writing by Aurecon.

Document control					aurecon
Report title		Feasibility-level engineering design - Balancing Dam			
Document ID		Unnumbered	**Project number**	112546	
File path		P:\Projects\112546 Algoa Recon Support\03 Prj Delivery\17 Reports\03 FEAS\03 Feas Level Eng Design - Bal Dam\Feasibility-Level Engineering Design - Balancing Dam (Final) Rev 1.Docx			
Client		Department of Water and Sanitation	**Client contact**	Mr AJ Moore	
Rev	**Date**	**Revision details/status**	**Authors**	**Verifier**	**Approver**
0	12 February 2020	Draft v1	O Human	Dr F Denys	E van der Berg
1	6 March 2020	Draft v2	O Human	Dr F Denys	E van der Berg
2	24 March 2020	Final	O Human	Dr F Denys	E van der Berg
Current Revision		**2**			

Approval			
Author signature		Approver signature	
Name	O Human	Name	Erik van der Berg
Title	Professional Engineer (Civil)	Title	Technical Director

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-LEVEL ENGINEERING DESIGN – BALANCING DAM

Final: March 2020

Prepared by: Aurecon South Africa (Pty) Ltd
P O Box 494
Cape Town, 8000
South Africa

Tel: 021 5265790
Fax: 086 5269500
E-mail: erik.vanderberg@aurecongroup.com

Prepared for: Director: National Water Resource Planning,
Director: Options Analysis
Department of Water and Sanitation
Private Bag X313
Pretoria 0001
South Africa

Mr Tendayi Makombe (PE: NWRP, Central)
Tel: 012 336 8503
E-mail: makombet@dws.gov.za

Mr Tony Moore (CE: Options Analysis)
Tel: 012 336 8528
E-mail: mooret@dws.gov.za

This report is to be referred to in bibliographies as:

Department of Water and Sanitation, South Africa 2020, *Feasibility-level Engineering Design* – *Balancing Dam*. Prepared by Aurecon South Africa (Pty) Ltd as part of the Support of the Water Reconciliation Strategy for the Algoa Water Supply System.

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

APPROVAL

Title	:	Feasibility-level engineering design – Balancing Dam
Consultant	:	Aurecon South Africa (Pty) Ltd
Report status	:	Final
Date	:	March 2020

STUDY TEAM

Approved for Aurecon South Africa (Pty) Ltd:

....
E VAN DER BERG
Technical Director

DEPARTMENT OF WATER AND SANITATION
Directorates National Water Resource Planning and Options Analysis

Approved for Department of Water and Sanitation:

....
T MAKOMBE
PE: NWRP (Central)

....
P MLILO
DIRECTOR: NWRP

....
A J MOORE
CE: Options Analysis

Document Index

Reports that will be produced as part of this Study are indicated below.

Bold type indicates this Report.

Report Index	Report Number	Report Title
1		Inception
2		Water Requirements Chapter
3		Identifications of Options for Balancing Storage Chapter
4		Environmental Constraints Analysis
5	P WMA 15/N40/00/2517/1	Topographical Survey
7a	P WMA 07/N40/00/2619/2	Lower Coerney Dam Geotechnical Survey
7b	P WMA 07/N40/00/2619/1	Upper Scheepersvlakte Dam Geotechnical Survey
7c	P WMA 07/N40/00/2619/3	Lower Coerney Dam Supplementary Geotechnical Survey
8		Desktop assessment of short-listed options Chapter
9	P WMA 15/N40/00/2517/3	Options Analysis Report
10		Layout and Affected Land and Infrastructure
11		**Feasibility-level Engineering Design - Balancing Dam**
12		Feasibility-level Engineering Design - Conveyance Infrastructure
13		Feasibility-level Cost and Implementation Analysis
14		Implementation Support
15	P WMA 15/N40/00/2517/4	Feasibility Study Report
16	P WMA 15/N40/00/2517/5	Stakeholder Participation: Feasibility Study

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 focus of the 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 investigation of a number of 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 sub-report and future reports, as there is no Upper Coerney Dam.

Balancing dam feasibility level design

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

The proposed dam is a homogeneous earthfill embankment dam. There is some zoning of the embankment fill for slope protection, rip-rap/cobblecrete 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, along with the expected *high* hazard rating, results in a Category III dam. This 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, hence the homogeneous embankment design, which makes use of a semi-pervious to impervious fill for the entire embankment fill. Sand, gravel and rocks are not available on site for the filter zones, embankment protection or 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 to 8 m on the left abutment, to 4 m in the river section and 3 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 prove more cost effective. 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 crossing of the access road and supply pipeline. It is therefore that the left abutment spillway option is preferred.

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 will be attenuated down to 110 m^3/s, after level pool routing through the 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 down 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 option, the 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 of the same width and side slopes of 1V:1H, lined with reinforced concrete to a depth of 1.7 m, equal to the depth of the safety evaluation flood (SEF) flow 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 freeboard of the dam was determined for a category III embankment dam, using the maximum flood levels of the attenuated floods as above, and were found to be 3.64 m. The freeboard provided is 3.8 m.

Supply pipeline and inlet/outlet works

The dam will be connected with a new 1400 mm dia steel pipe to the existing water supply scheme pipe of 1400 mm dia, conveying water to the Nooitgedagt WTW. This pipe will link the new offtake to be located on the Kirkwood primary canal, the above-mentioned supply pipe and the dam. The supply pipeline to the dam will bifurcate into an inlet and outlet branch and reduce to 1200 mm dia at the downstream outlet chamber. The pipes will then reduce again to 1000 mm dia at the toe of the dam before being encased in reinforced concrete through the embankment. The pipes will enter the reservoir through a wet well tower 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 the dam and its spillway.

The river diversion strategy for the construction of Lower 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.

The legislative requirements for the implementation of the dam with regards to the environmental authorisation, water use licences and ecological water requirement will be discussed in the Implementation Support Report (to be compiled). The dam safety regulation requirements and licence requirements are briefly discussed in this report.

Recommendations

A number of recommendations emanate from the design and report and can be summarised as follows. The storage volume and losses should be refined and confirmed. Sources for sand, gravel and rock could be further investigated to refine their use and impact on the design. The embankment zoning and dimensions are based on typical values for dams of this size, these should be refined during the design process and the embankment stability investigated further. Targeted investigations should be done on the founding conditions for a spillway located on the right abutment and the comparison between left and right spillway options revised to confirm the findings of the current study. Further thought could also be given to the provision of dual spillway to reduce the capacity of the service spillway to contain the RDF only and provide capacity for the SEF in an auxiliary spillway. Site specific flood hydrology study should be undertaken to determine the Recommended Design Flood (RDF) and Safety Evaluation Flood (SEF). The spillway dimensions should be refined, and a hydraulic model study undertaken to confirm the design.

Contents

Appendices

Figures

Tables

Abbreviations

DN	Nominal diameter
DWAF	(Previous) Department of Water and Forestry
DWS	Department of Water and Sanitation
EIA	Environmental Impact Assessment
EWR	Ecological water requirements
FSL	Full supply level
GWS	Government Water Scheme
ha	hectares
ID	Internal diameter
LSRGWS	Lower Sundays River Government Water Scheme
LSRWUA	Lower Sundays River Water User Association
MOL	Minimum outlet level
Mℓ	Megalitre
masl	Metres above mean sea level
NMBM	Nelson Mandela Bay Municipality
PMF	Probable Maximum Flood
RDF	Recommended Design Flood
RL	Related level
SANCOLD	South African National Committee on Large Dams
SEF	Safety Evaluation Flood
WSS	Water Supply Scheme
WTW	Water Treatment Works
WUA	Water User Association

1 Introduction and background

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 the investigation is on providing adequate balancing storage for supply to the NMBM, to limit risks of shortfall in supply.

1.2 Purpose of this Sub-report

The purpose of this sub-report is to describe the design parameters, assumptions and feasibility design of a dam at the chosen site at Lower Coerney. This report does not include the cost estimate, which will be addressed in the Feasibility-level Cost and Implementation Analysis Sub-report to follow.

This will form a Chapter/s of the Feasibility Design Report.

1.3 Background

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 a number of potential dam sites, as documented in the Options Analysis Report (DWS, 2019), 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 sub-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 and background

Provides a brief background of the project and an introduction and background to the report.

Chapter 2: Salient features of the proposed dam design

Presents the characteristics of the dam and its appurtenant structures as addressed in more detail in the report.

Chapter 3: Description of the site

Describes the layout and topography of the chosen site, the basin characteristics and water storage requirement.

Chapter 4: Dam safety classification

Describes the classification of the proposed dam, according to the dam safety regulations.

Chapter 5: Overview of the geology and construction materials

Provides a brief overview of the outcomes and conclusions of the geotechnical investigations at the chosen dam site.

Chapter 6: Embankment design

Describes the considerations, inputs and conclusions of the design of the embankment.

Chapter 7: Flood hydrology

Presents the main points and outcomes from the flood hydrology study of the site, as well as the selection of the design floods.

Chapter 8: Spillway design

Discusses the spillway possibilities, design alternatives, and the chosen spillway arrangement. The spillway design is then developed, considering the overflow structure, discharge channel and termination structure. In light of the proposed spillway design, the results of the flood routing process are presented.

Chapter 9: Freeboard determination

Following from the design of the embankment and the spillway, as well as the results of the flood routing, the freeboard requirements are determined and checked against the freeboard guidelines.

Chapter 10: Outlet works

Describes the proposed inlet and outlet works of the dam.

Chapter 11: Associated infrastructure

Briefly discusses infrastructure associated with the proposed dam, namely electrical supply, access roads and dam monitoring instrumentation.

Chapter 12: Constructability and river diversion

Briefly describes the river diversion considerations and strategy.

Chapter 13: Legislative considerations

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 14: Conclusions

Summarises the conclusions from the feasibility design.

Chapter 15: Recommendations

Lists the recommendations emanating from the feasibility design.

2 Salient features of the proposed dam design

The salient features of the proposed Coerney Dam are presented in **Table 2-1** below.

Table 2-1: 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
Probable Maximum Flood (PMF)	835 m^3/s

Parameter	Value
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 Operating 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.

Parameter	Value
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. 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 outlet description	No allowance for environmental releases is currently included in the design. It is recommended that further studies be conducted to determine the ecological water requirement (EWR) and arrangement required to provide this.

3 Site overview

3.1 Location and layout

The proposed location of the Coerney Dam (**Figure 3-1**) is upstream of the Coerney Siphon on 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.

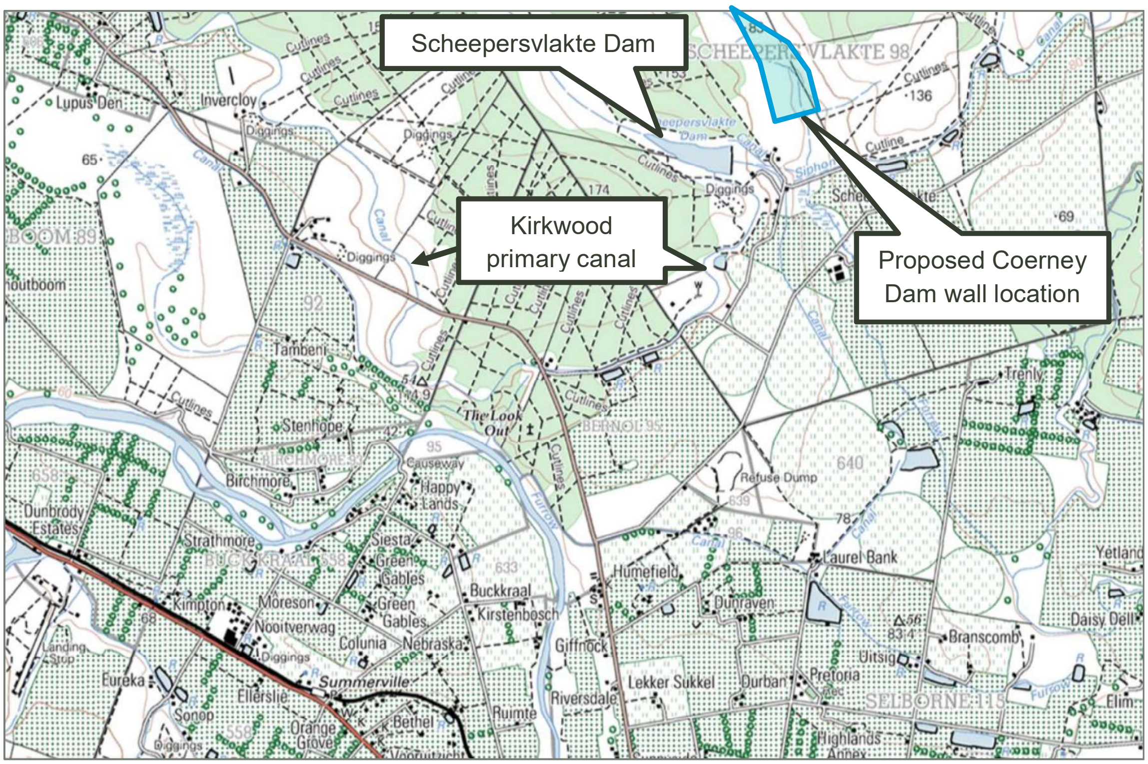


Figure 3-1: Layout plan of the Scheepersvlakte and proposed Coerney Dams

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 (refer to the report *Feasibility-level Design: Conveyance infrastructure*). The dam will also supply the Nooitgedagt WTW via a new connecting pipeline to the existing 1 400 mm Nooitgedagt pipeline. This is shown in **Figure 3-2**.

A description of the proposed scheme, its layout, its components and its operation are described in the report; *Layout and Affected Land and Infrastructure* (DWS, 2019).

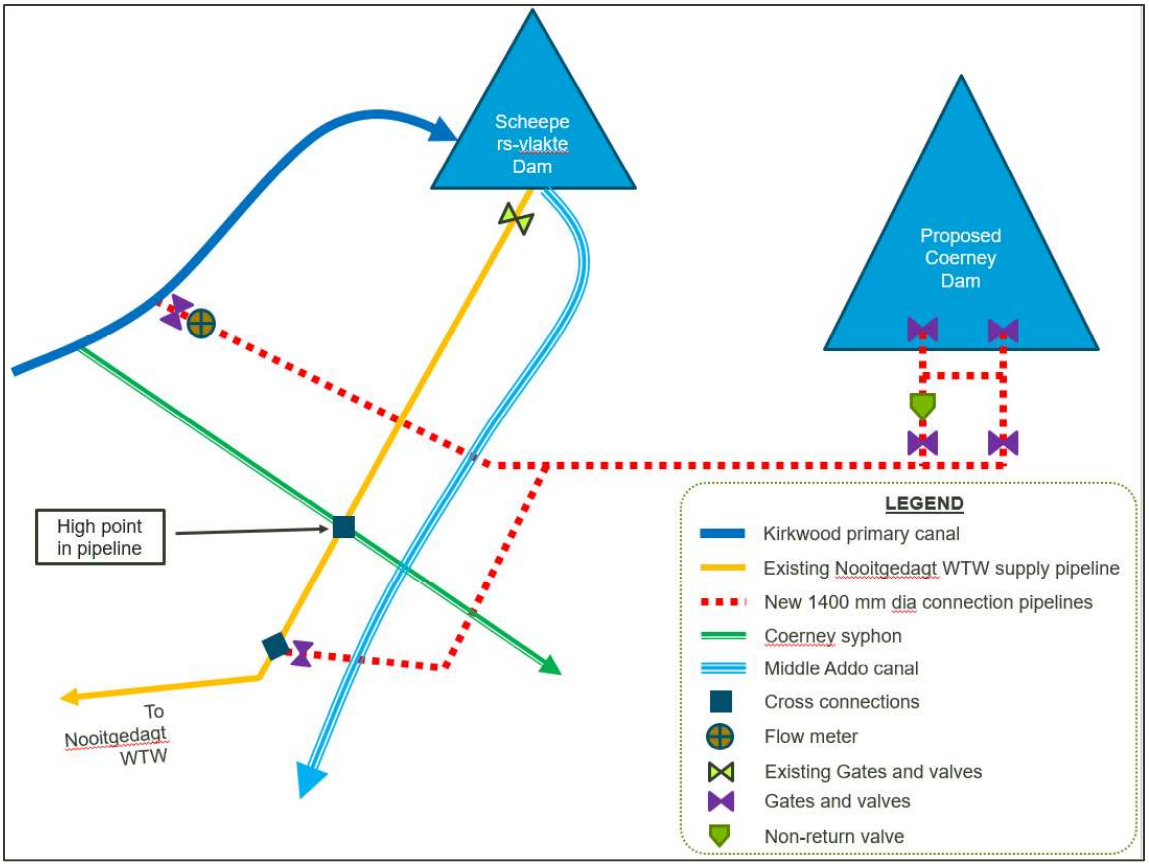


Figure 3-2: Schematic layout of the proposed Coerney Dam and connecting pipelines

3.2 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.

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.

The sedimentation from the catchment and the canal inflows have been estimated as follows:

- Sedimentation from catchment inflows: 15 m^3/km^2/annum.
- Sedimentation from canal inflows: 0.002 % of inflows[2]

The volume of sediment due to canal inflows over a 50-year period is thus estimated at approximately 32 500 m^3. The following assumptions were made:

- Canal flows to replace evaporation losses (mean annual quaternary evaporation of 1 650 mm/year) of 1.19 million m^3 per annum
- Use of the dam's storage during weekend supply to the WTW amounting to 1.68 million m^3 per annum
- Infiltration losses of 10% of storage volume amounting to 0.46 million m^3 per annum

Furthermore, sediment from the catchment inflow is estimated using a sediment load of 15 m^3/km^2/annum (from Scheepersvlakte Dam), which equates to 25 000 m^3 over a 50-year period.

The total estimated sediment equals 57 500 m^3 over 50 years. The dead storage provided with the minimum outlet level of 86.0 masl is thus more than sufficient to provide for siltation dead storage as well as some buffer capacity.

3.3 Topographical survey

A topographical survey was completed by Department of Water and Sanitation (DWS) Southern Operations (National Water Resource Infrastructure) for (Lower) Coerney and Upper

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

Scheepersvlakte Dams in May 2018. The results are reported in the relevant survey reports; Upper Scheepersvlakte Dam, Contour Survey (EC004/2018) and Coerney Dam Contour Survey (EC 003/2018).

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, were digitised. Nine test sections were surveyed for the Coerney site, to compare and verify the digitised data to the actual ground data, which resulted in a good match.

Then, in August 2018, the survey was updated and expanded to include the immediate surrounding infrastructure, which is reported in the Scheepersvlakte Contour and Detail Survey Report (EC026/2018).

3.4 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 3-3** below.

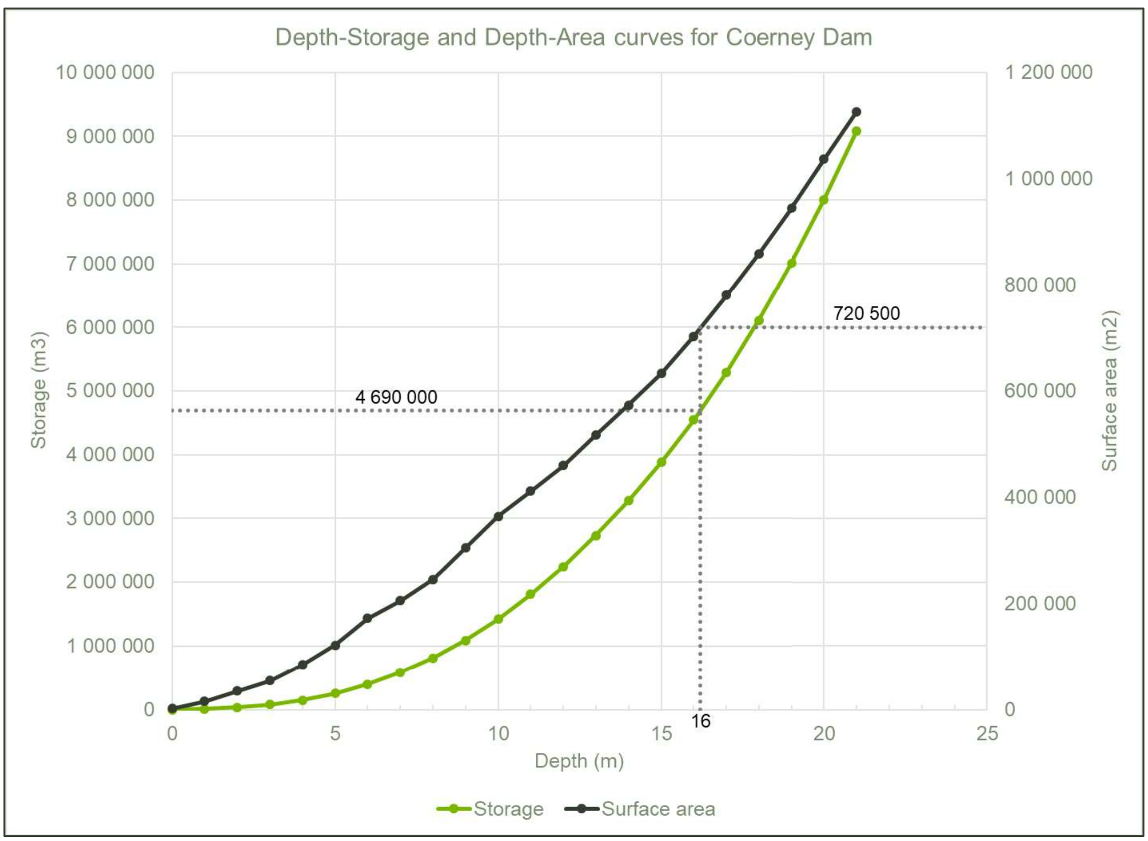


Figure 3-3: Depth-Storage and Depth-Area curves for Coerney Dam

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.

4 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 4-1**, as reproduced from the regulations.

Table 4-1: 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 4-1**, 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 Overview of the geology and construction materials

5.1 Introduction and background

Geotechnical investigations for the options analysis between Upper Scheepersvlakte and (Lower) Coerney Dam sites were conducted in 2018. These investigations were conducted to inform a recommendation on the preferred dam site. These investigations included geophysical surveys (resistivity), test pitting using a tractor-loader-backhoe (TLB), in-situ field testing including standard penetrometer tests (SPT), sampling and laboratory testing, as well as rotary core drilling and water pressure (Lugeon) testing. The findings of this investigation relevant to the (Lower) Coerney site are reported in *Lower Coerney Dam Geotechnical Survey* (Report no P WMA 07/N40/00/2619/2).

With 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. The investigation focussed on confirming available material quantities in the basin area, determining probable founding conditions for the spillway chute and particularly its termination structure, and providing some additional detail to the embankment founding conditions, especially on the upper right flank. Findings of this investigation are reported in *Lower Coerney Dam Supplementary Geotechnical Survey* (Report no P WMA 07/N40/00/2619/3), where further details can be found. Note that this report is a stand-alone geotechnical report as it incorporates all data and findings from the first geotechnical report. The following sub-sections mainly summarise the relevant findings of the geotechnical investigation.

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 8. The investigations on the right abutment are thus limited to some test pits, and no core drilling was done there. Should the detail design confirm 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.

5.2 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 Peak Ground Acceleration (PGA) values are less than 0.02g, with a 10% probability of being exceeded in a 50-year period.

5.3 Dam wall foundation

The foundation of the embankment comprises of two main components, namely the core cut-off trench and the shell zone. Considering 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) and 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 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, in order to remove the potentially organic-rich, and potentially compressible topsoil stratum. The latter value of stripping of 0.5 m is used further, such as for embankment volumes.

5.4 Foundation of spillway

The geology in the vicinity of the spillway and its discharge channel comprise of 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 of 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.

5.5 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 allowances have 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.

5.6 Materials

5.6.1 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%. This applies to all the material types encountered. The percentages passing the

0.425 mm sieves 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, where, for impervious materials, occasional samples 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 indicate 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 5-1) then the general compliance of the soils within the Coerney 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 5-1: 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

5.6.2 Sand filter and concrete aggregates

Other materials, such as coarse aggregate for concrete and sands for filters, and fine aggregate were not found in the basin 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 distances away from 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.

- 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 Coerney site, also in the Uitenhage area; and produces both sand and 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 at about 65 km from site.

5.7 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.

6 Embankment

6.1 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 5), 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 the drawings in Appendix B, 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.

6.2 Embankment layout

The proposed embankment design has typical dimensions for a dam of its size. The 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 112546-0000-DRG-CC-001 for a layout drawing.

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 6-1** showing the zones and various elements, which are discussed below. This is shown in more detail in Drawing 112546-0000-DRG-CC-003 found in **Appendix B**.

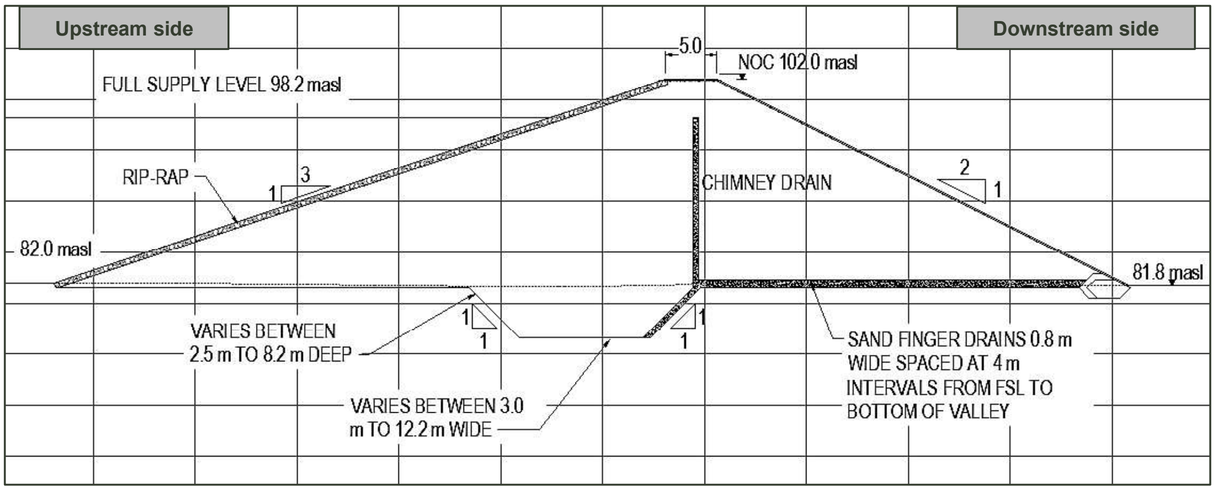


Figure 6-1: Illustrative cross section through the proposed embankment

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, 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 5.7, but also to limit the effect of arching of placed fill, which could occur if the slopes were steeper.

7 Flood hydrology

7.1 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).

Based on the size of the study catchments 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 7-1**.

Table 7-1: 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, 8am to 8am (which is a wholly artificial time resolution). But intense rainstorms might have a duration that straddle the artificial 8am cut-off for a "daily" measurement. Consequently, the "daily" values in the rainfall record, representing such "straddling" storms on either side of the artificial 8am cut-offs, cannot reflect the maximum 24-hour values which do reflect those intense rainstorms. 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 8am to 8am 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 are given in **Table 7-2** below.

Table 7-2: 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

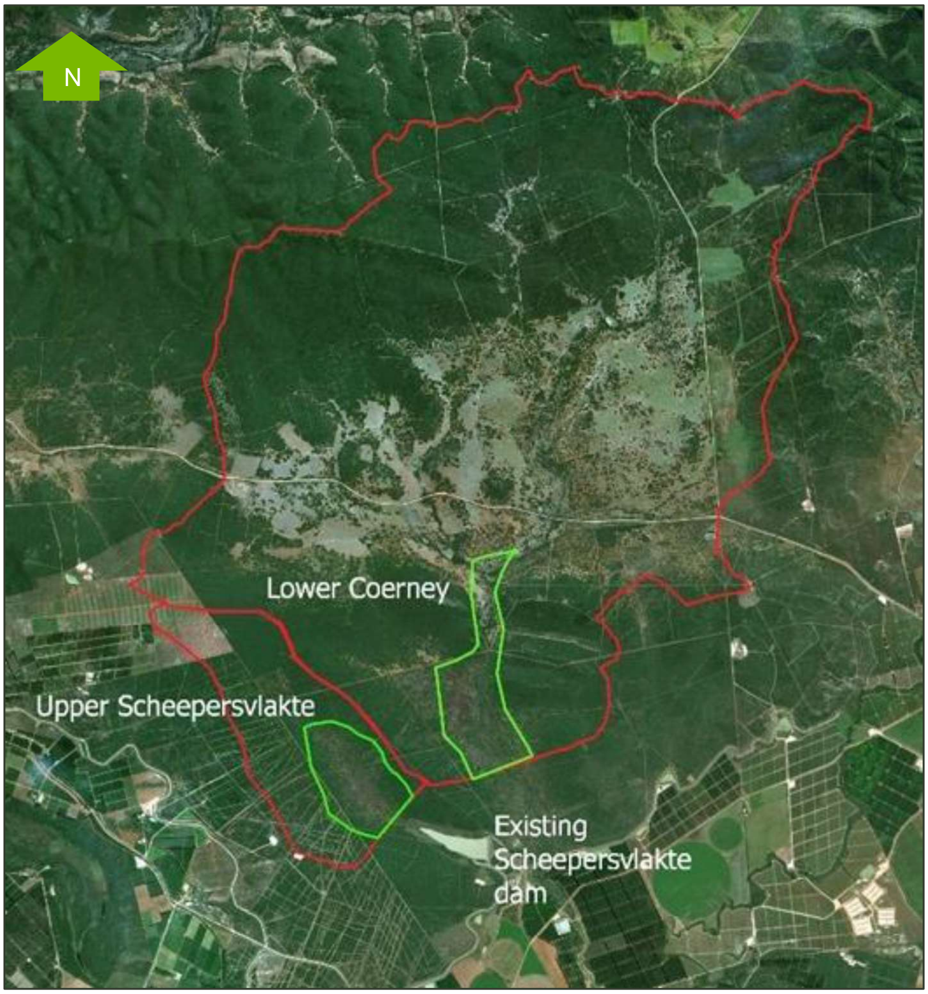


Figure 7-1: Dam catchment

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 7-3**.

Table 7-3: 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**

7.2 Design floods

Based on the height, storage capacity and expected hazard potential downstream of the dam, as discussed in Section 4, the expected classification for the dam is Category 3.

By that standard, the SANCOLD Guidelines in Relation to Floods (SANCOLD, 1991) recommend that:

- Recommended Design Flood (RDF) equals the 1:200 year recurrence interval (0.5% AEP) 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.

8 Spillway

8.1 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 8-1** below. Further details can be found in **Appendix B** Drawing 112546-0000-DRG-CC-001.

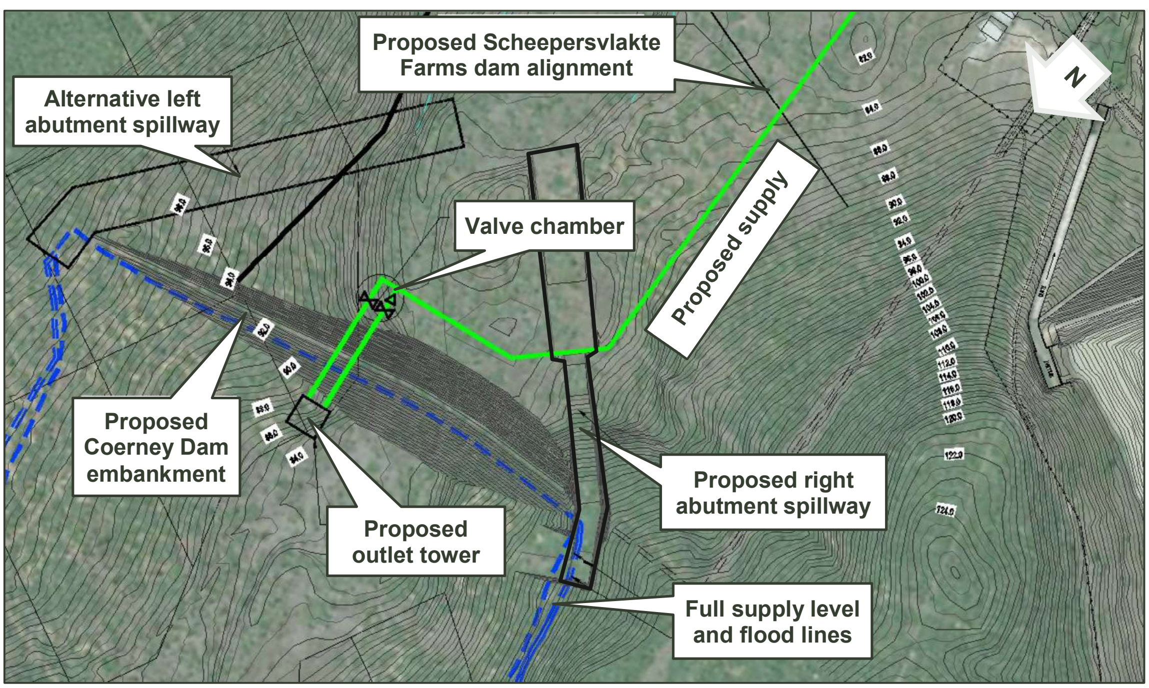


Figure 8-1: Plan view of the embankment showing two spillway location options

8.1.1 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.

8.1.2 Left abutment option

The original position earmarked for the spillway was on the left abutment. The main advantage of 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 and from 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 shallow wide and long 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.

8.1.3 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 which 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.

8.2 Spillway type and arrangement

Considering the general lack of bedrock at 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, comparing them both on the left and right abutments.

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 Lower Coerney Dam site.

8.3 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 and side channel is shown below in **Figure 8-2**. Further details can be found in **Appendix B** Drawing 112546-0000-DRG-CC-004.

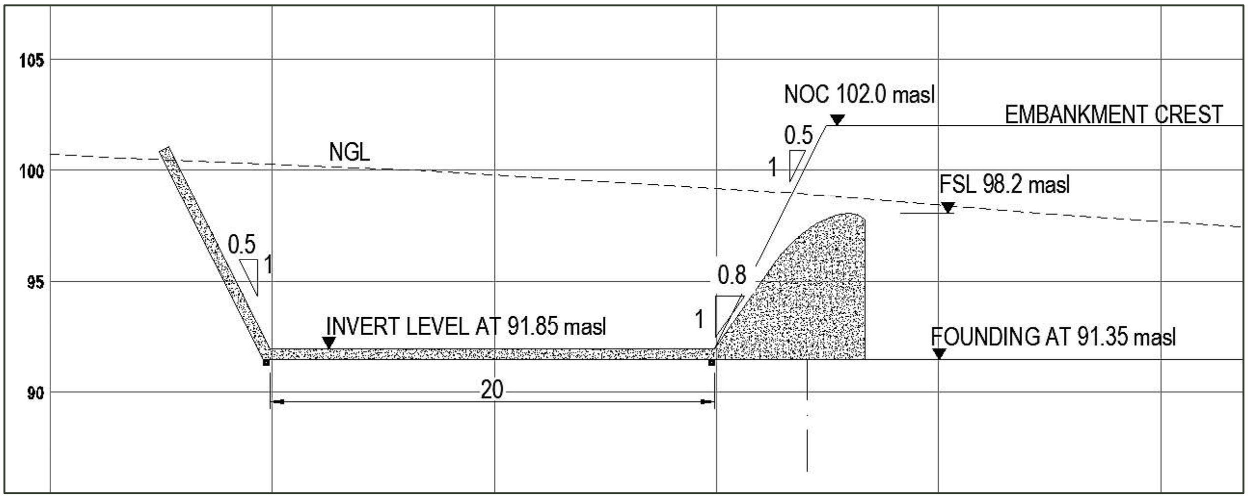


Figure 8-2: Cross section through spillway overflow and side channel

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 8-1**).

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 A and the discharge rating curve is shown in **Figure 8-3** below. The discharge head and maximum stages for the routed and un-routed RDF and SEF are given in **Table 8-1**.

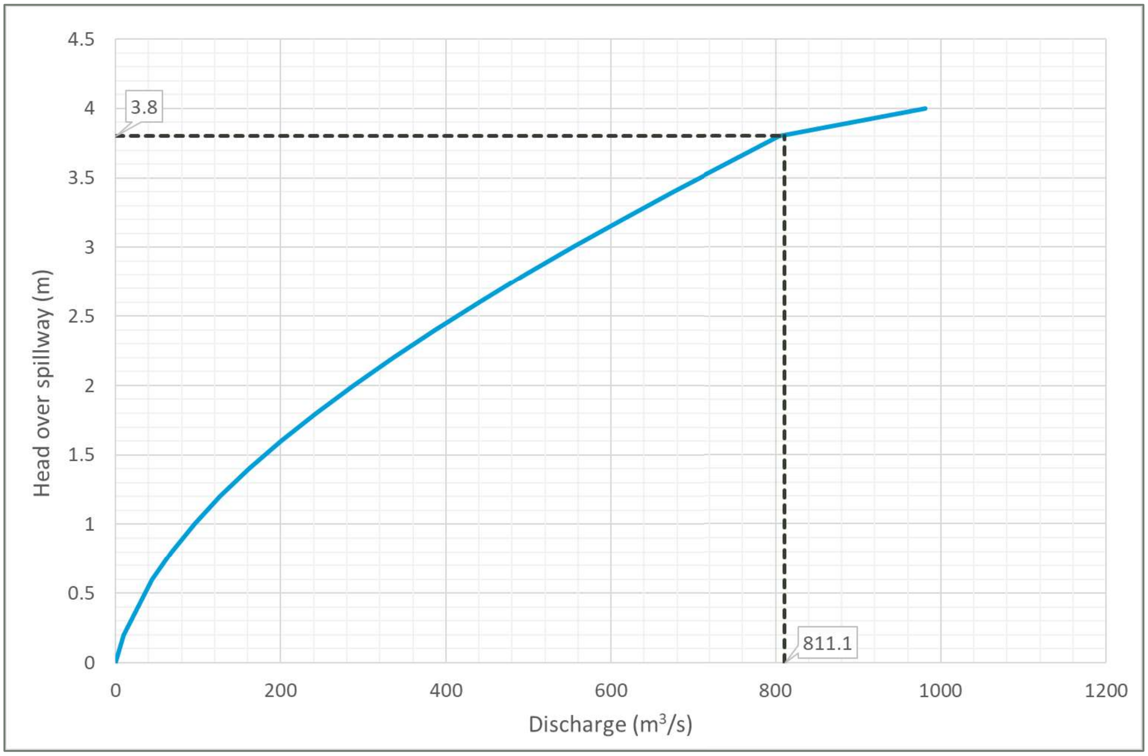


Figure 8-3: Spillway rating curve (showing discharge at NOC)

8.4 Side channel

The ogee 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.

8.5 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 8-4**. Further details can be found in **Appendix B** Drawing 112546-0000-DRG-CC-004.

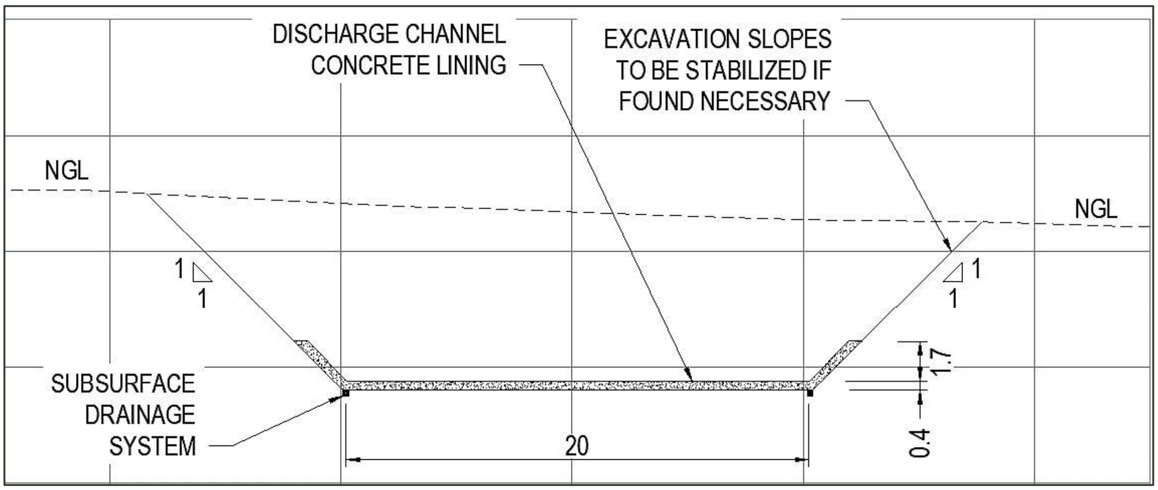


Figure 8-4: Cross section through the spillway discharge channel

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.

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.

8.6 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.

8.7 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 8-1** below with more details provided in **Appendix A**.

Table 8-1: 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

9 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 9-1**, with detailed calculations and input parameters in **Appendix A**.

Table 9-1: Freeboard combinations

Aspect	Value
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
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**

10 Outlet works

10.1 Inlet/Outlet works configuration

The proposed Coerney Dam will be connected to the existing water supply scheme via a 1 400 mm diameter steel pipe. 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 dia to 1200 mm and then 1000 mm dia after it bifurcates into a separate inlet and outlet branch at the outlet chamber at the downstream toe of the embankment. It is proposed that the encased pipework through the embankment be made of stainless steel.

The layout is shown in **Figure** 10-1. Further details can be found in **Appendix B** Drawing 112546-0000-DRG-CC-001.

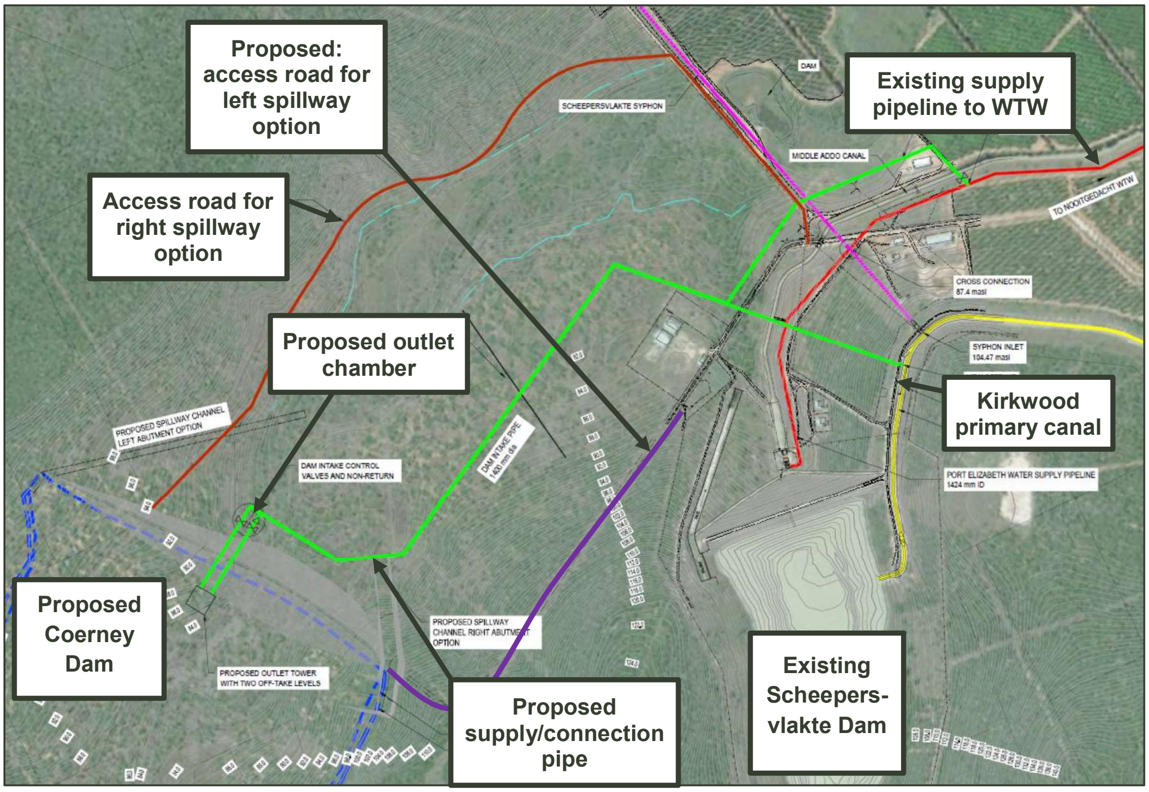


Figure 10-1: Dam and supply/connection pipe layout

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 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 dia inlet/outlet pipes will be encased in reinforced concrete through the embankment. This is shown in **Figure 10-2** and **Figure 10-3** below.

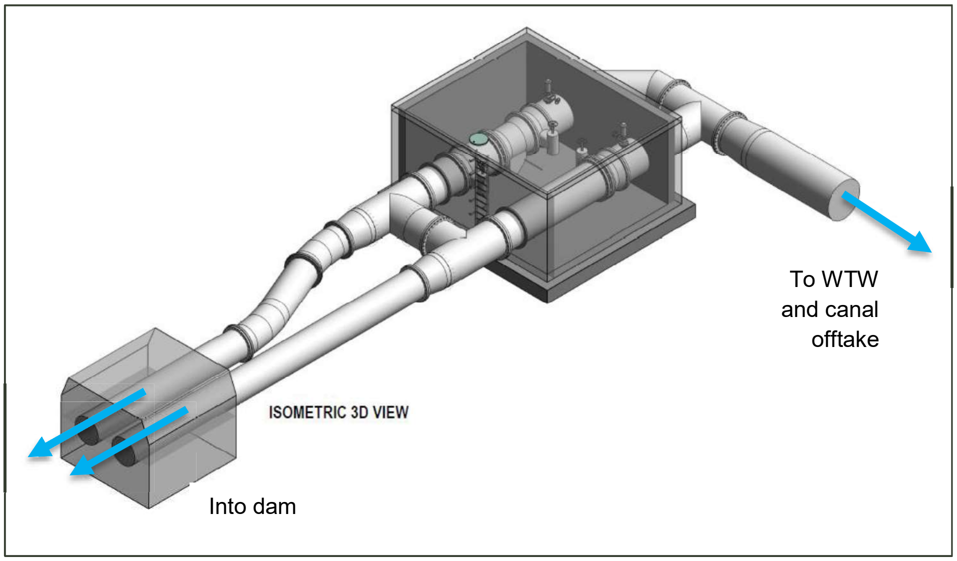


Figure 10-2: Isometric view of the dam's downstream outlet pipe chamber arrangement

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.

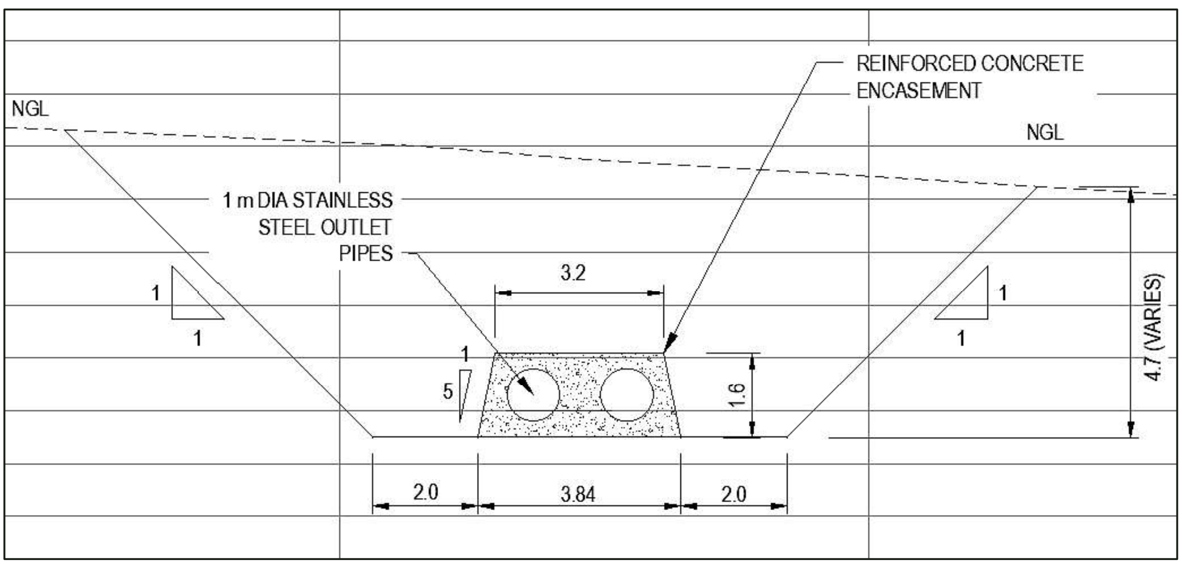


Figure 10-3: Cross section of the outlet pipe encasement through the embankment.

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. It has been recommended in the *Affected Land and Infrastructure* report (DWS, 2019) that the EWR be investigated in more detail. This investigation can form part of the EIA process.

10.2 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 is shown in **Figure 10-4** below. Further details can be found in **Appendix B** Drawing 112546-0000-DRG-CC-005.

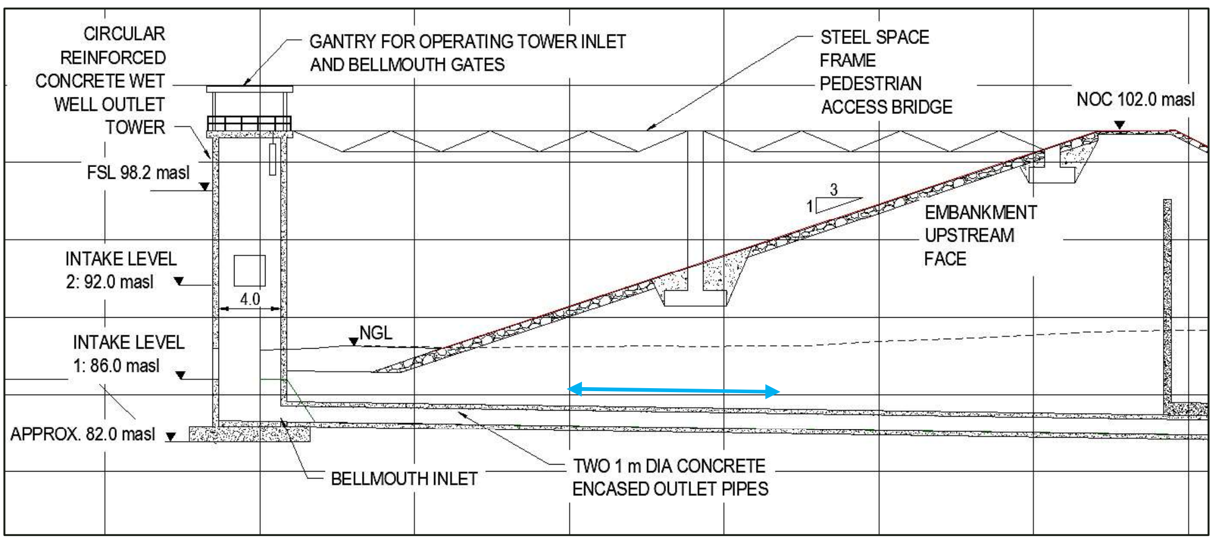


Figure 10-4: View of the outlet tower and access bridge

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 top.

11 Associated infrastructure

11.1 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.

11.2 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.

11.3 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.

11.4 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.

12 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 7-3**. 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 12-1** below as an indication of the order of magnitude of the river diversion.

Table 12-1: 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 7, as well as the acceptable risk and size of the diversion flood to accommodate, 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).

13 Legislative considerations and authorisations

13.1 Environmental Impact Assessment

The environmental impact assessment (EIA) process is expected to commence in 2020, as soon as DWS has appointed an Environmental Assessment Practitioner. This will be further dealt with in the *Implementation Support Report*.

13.2 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), of the National Water Act, 1998). This licence application is included in the scope of work for the EIA study (refer also the *Implementation Support Report)*.

13.3 Ecological Water Requirement

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

A Reserve determination is required to ascertain the releases required from the dam for ecological purposes during different times of the year.

This aspect is discussed further in the *Implementation Support Report*.

13.4 Dam Safety Licence Requirements

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:

- Apply for classification of the dam with the Dam Safety Office (DSO) (as part of the Department of Water and Sanitation). The dam is expected to be classified as a Category III dam. This requires the services of an approved professional person/engineer (APP) supported by an approved professional team.

- The APP will be responsible for the design work as well as submitting an application for a Licence to Construct from the DSO, which comprises an application form, design report, engineering drawings and construction specifications.
- A Water Use Licence or written authorisation must be obtained from the Regional Director of the relevant region before a Licence to Construct can be issued.
- During construction, the APP must submit quarterly reports to the DSO on progress of the construction of the dam.
- Before the construction completion and impoundment is set to commence, the APP must apply for a Licence to Impound from the DSO. This involves the compilation and submission of an operation and maintenance manual and emergency preparedness plan.
- After completion of all construction work, the APP must register the dam, submit a completion report, completion drawings and a completion certificate stating that the work has been completed according to his/her specifications.

13.5 Land ownership

The portion of land upon which the dam is to be located is known as Portion 7 of Scheepers Vlakte of the Scheepersvlakte Farms Pty Ltd. The affected land and infrastructure are discussed in further detail in the Affected Land and Infrastructure Report (DWS, 2019).

14 Conclusions

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

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 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 along with the expected *High* hazard rating, results in a Category III dam.
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, gravel and rock are not available on site for the filter zones, embankment protection or concrete aggregates 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 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.
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 via a 1400 mm dia supply pipeline.
23) At the outlet chamber at the toe of the dam, the supply pipeline will bifurcate into an inlet and outlet branch, and then reduce to 1200 mm dia and then 1000 mm dia before entering the dam. These two pipes through the embankment will be of stainless steel and will be concrete encased.
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 Lower 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.

A number of recommendations emanate from the feasibility design, which are discussed in the following section.

15 Recommendations

The design and construction of the dam at the Lower Coerney site has been shown to be the most favourable of the sites investigated during previous study phases and the proposed design is considered feasible. However, a number of design inputs and elements should be refined in the detailed design phase.

a) 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.

b) Filter sand, gravel 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.

c) 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.

d) 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 with further targeted geotechnical investigations on the founding conditions for a spillway on the right abutment. The two sites can then be properly comparted and the best spillway location confirmed.

e) 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.

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

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

References

Badenhorst, D.B. 1988. *The design of earth- and rockfill dam structures*. Unpublished Master's Degree Thesis. (in Afrikaans). University of Pretoria.

Chief Directorate Surveys and Mapping (DLA-CDSM 2006). 1:50 000 inland water features dataset.

Department of Water and Sanitation, South Africa 2017, Environmental Constraints Analysis. Prepared by Aurecon South Africa (Pty) Ltd as part of the Support of the Water Reconciliation Strategy for the Algoa Water Supply System.

Department of Water and Sanitation (DWS), 2016. Report on the Reliability of Scheepersvlakte Dam Domestic Water Supply. Second Draft Report. Prepared by Naidu Consulting (Pty) Ltd. As part of the Scheepersvlakte Dam Remedial Works project.

Department of Water and Sanitation, South Africa 2017, Identification of Options for Balancing Storage. Prepared by Aurecon South Africa (Pty) Ltd with AfriCoast Consulting Engineer's assistance as part of the Support of the Water Reconciliation Strategy for the Algoa Water Supply System.

Department of Water and Sanitation, South Africa 2018, Desktop Assessment of Short-Listed Options. Prepared by Aurecon South Africa (Pty) Ltd as part of the Support of the Water Reconciliation Strategy for the Algoa Water Supply System.

Department of Water and Sanitation, South Africa 2018. Coerney Dam Contour Survey (EC 003/2018). Survey Services: Southern Operations (National Water Resource Infrastructure).

Department of Water and Sanitation, South Africa 2018. Scheepersvlakte Contour and Detail Survey Report (EC026/2018). Survey Services: Southern Operations (National Water Resource Infrastructure).

Department of Water and Sanitation, South Africa 2019, Options Analysis. Report number P WMA 15/N440/00/2517/3. Prepared by Aurecon South Africa (Pty) Ltd as part of the Support of the Water Reconciliation Strategy for the Algoa Water Supply System.

Design of Small Dams, 1987. United States Department of the Interior, Bureau of Reclamation.

DWAF 2004. Farm Dams and rivers dataset.

“Guidelines on Safety in Relation to Floods” prepared by SANCOLD, December 1991.

“Lower Sundays River GWS, Scheepersvlakte Dam, Report on Dam Safety Evaluation Carried Out in Terms of Government Notice R 139 of February 2012” prepared by Department of Water and Sanitation (DWS), 2016.

SANCOLD, 1990. Interim Guidelines on Freeboard for Dams, South African National Committee on Large Dams, Report No 3, August 1990.

SANCOLD, 1991. Guidelines on Safety in Relation to Floods, South African Committee on Large Dams, Report No 4, December 1991.

“Second Draft Report on the Reliability of Scheepersvlakte Dam for Domestic Water Supply” prepared by Naidu Consulting for the Department of Water and Sanitation, 6/01/2016.

Ven te Chow, 1959. Open Channel Hydraulics, International Edition.

Appendix A: Design Calculations

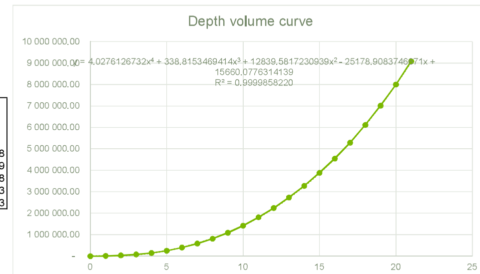
Coerney Dam
LEVEL POOL ROUTING

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

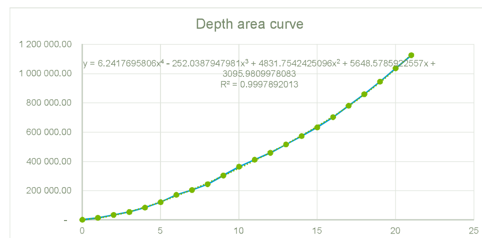
Volume		70000.00	m3
Height	3.077304812	70 000.00	m3
Elevation	85.07730481		masl

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

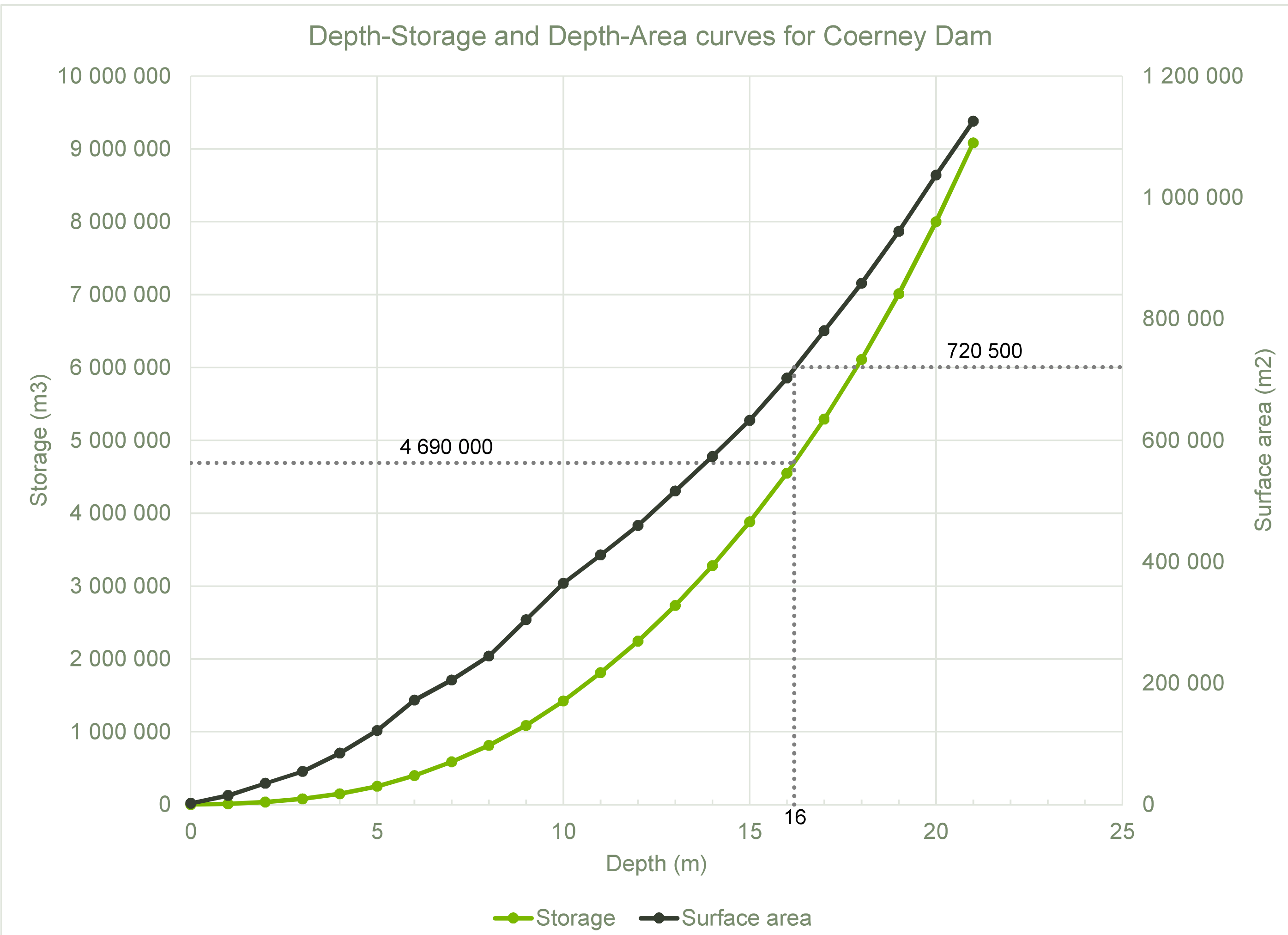
AREA

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
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
Storage (m3)
1 200 000
1 000 000
800 000
600 000
400 000
200 000
0
Surface area (m2)
720 500
4 690 000
16
0
5
10
15
20
25
Depth (m)
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

$$Q = 1.838L_eH^{1.5}$$

Ogee discharge parameters

Ho =	2.9	m
Co =	2.18	m

Ho/Hmax 0.75

Contraction due to piers and abutments

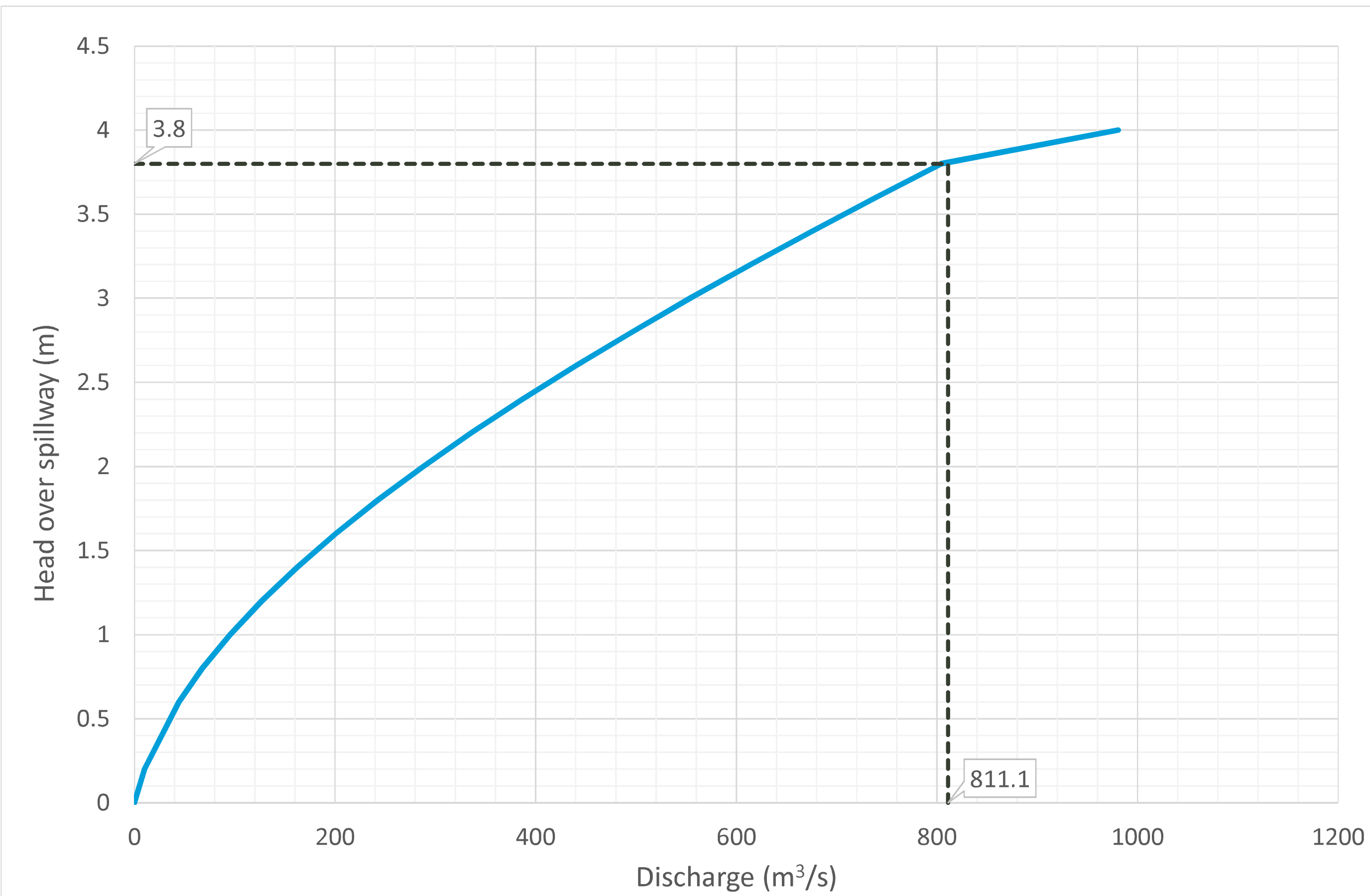
Number of piers, N =	0	No
Pier contraction coefficient, Kp =	0.02	
Abutment contraction coefficient, Ka =	0.2	

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

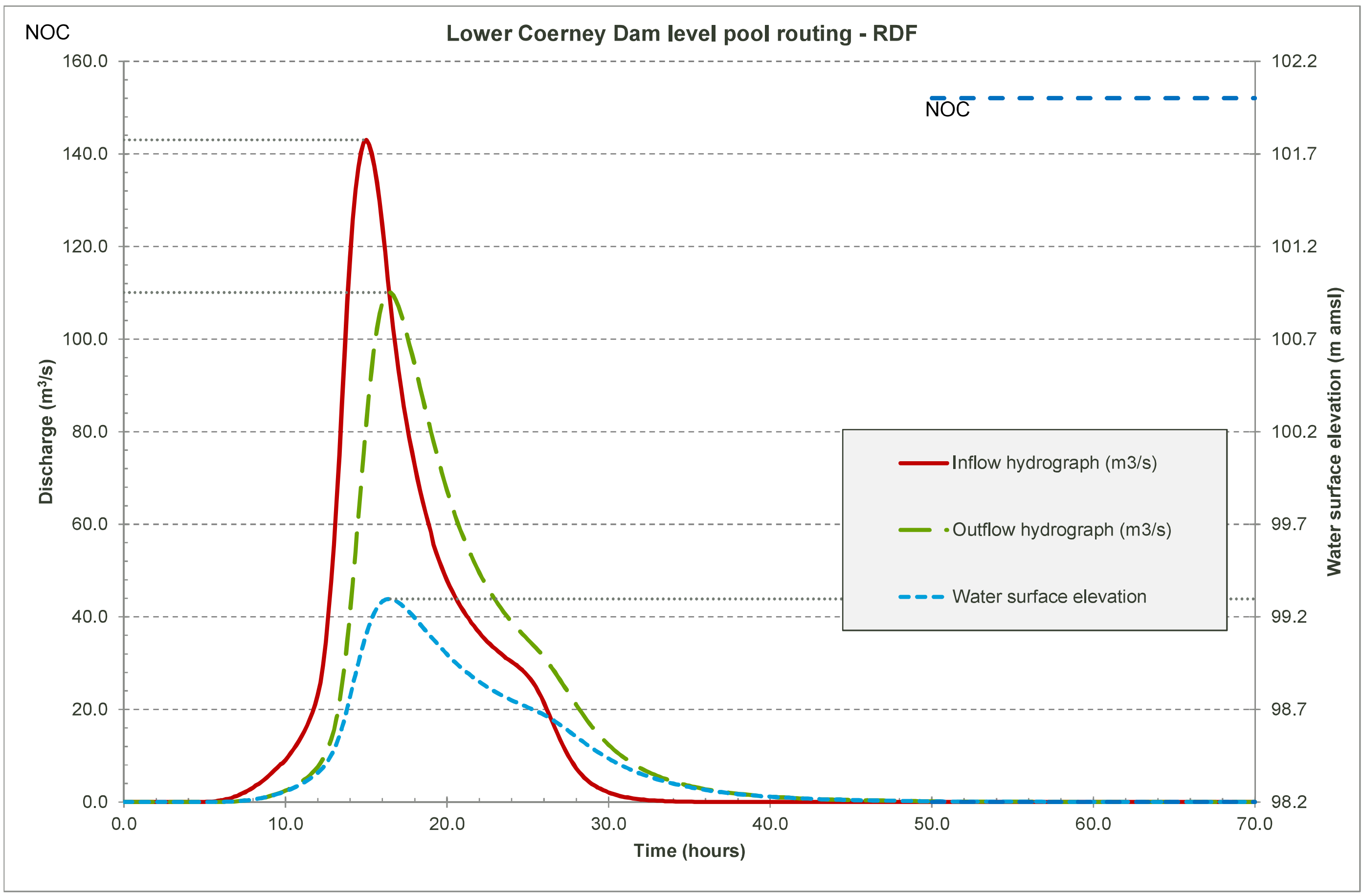
L' = Net length of crest
He = Head on crest

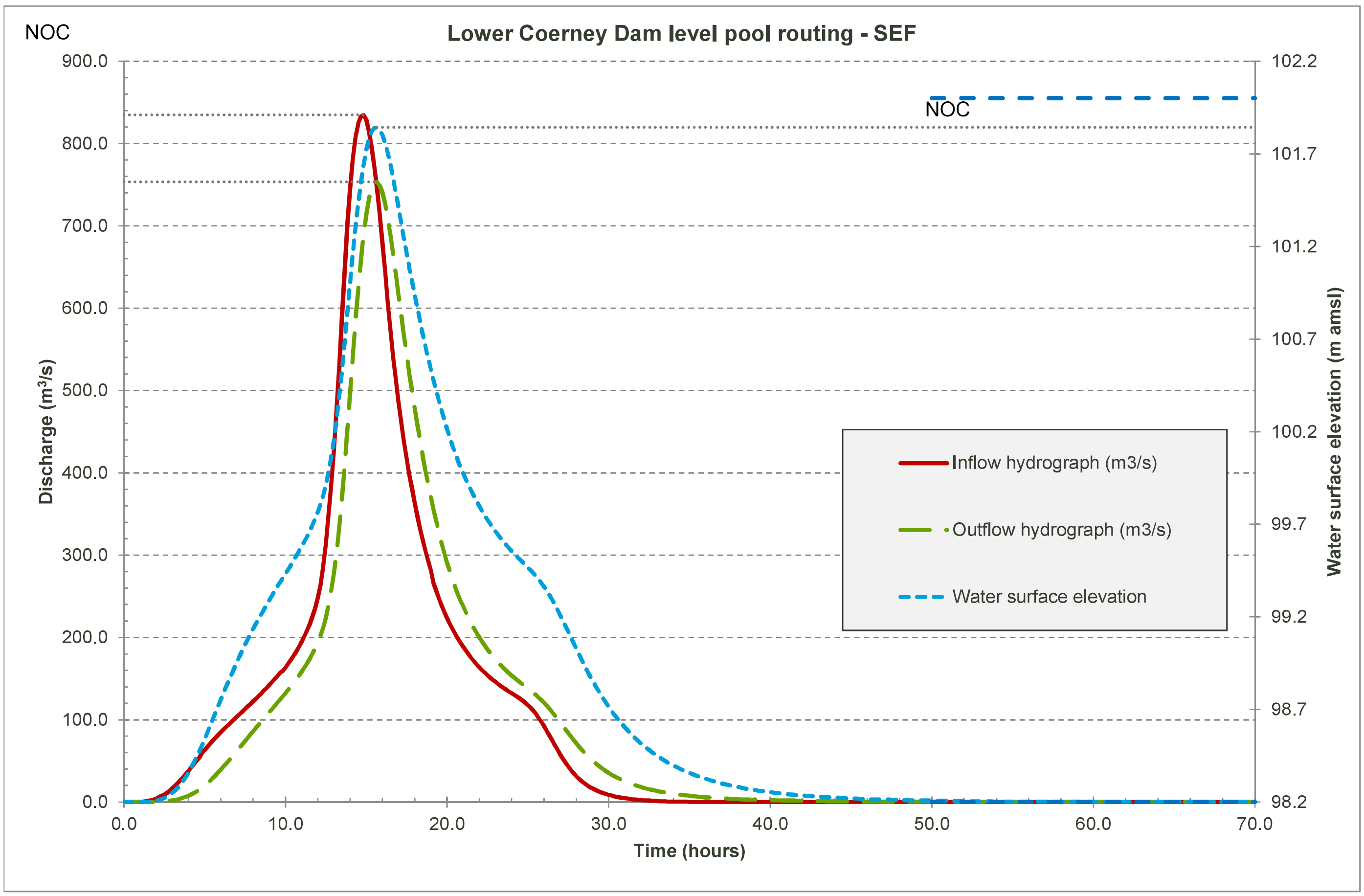
Water level (m)	Head above FSL (m)		Discharge coefficient, C		Effective crest length, L' = Le (m)	Spillway crest discharge (m^3/s)	NOC discharge (m^3/s)	TOTAL	
	Ogee Spillway	NOC (m)	Ogee Spillway	NOC, Cd (m)	Ogee Spillway				
98.20	0.00	0.00	0	1.45	50.00	0.00	0.00	0	
98.30	0.10	0.00	1.75	1.45	49.96	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



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


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


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

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 protector	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.

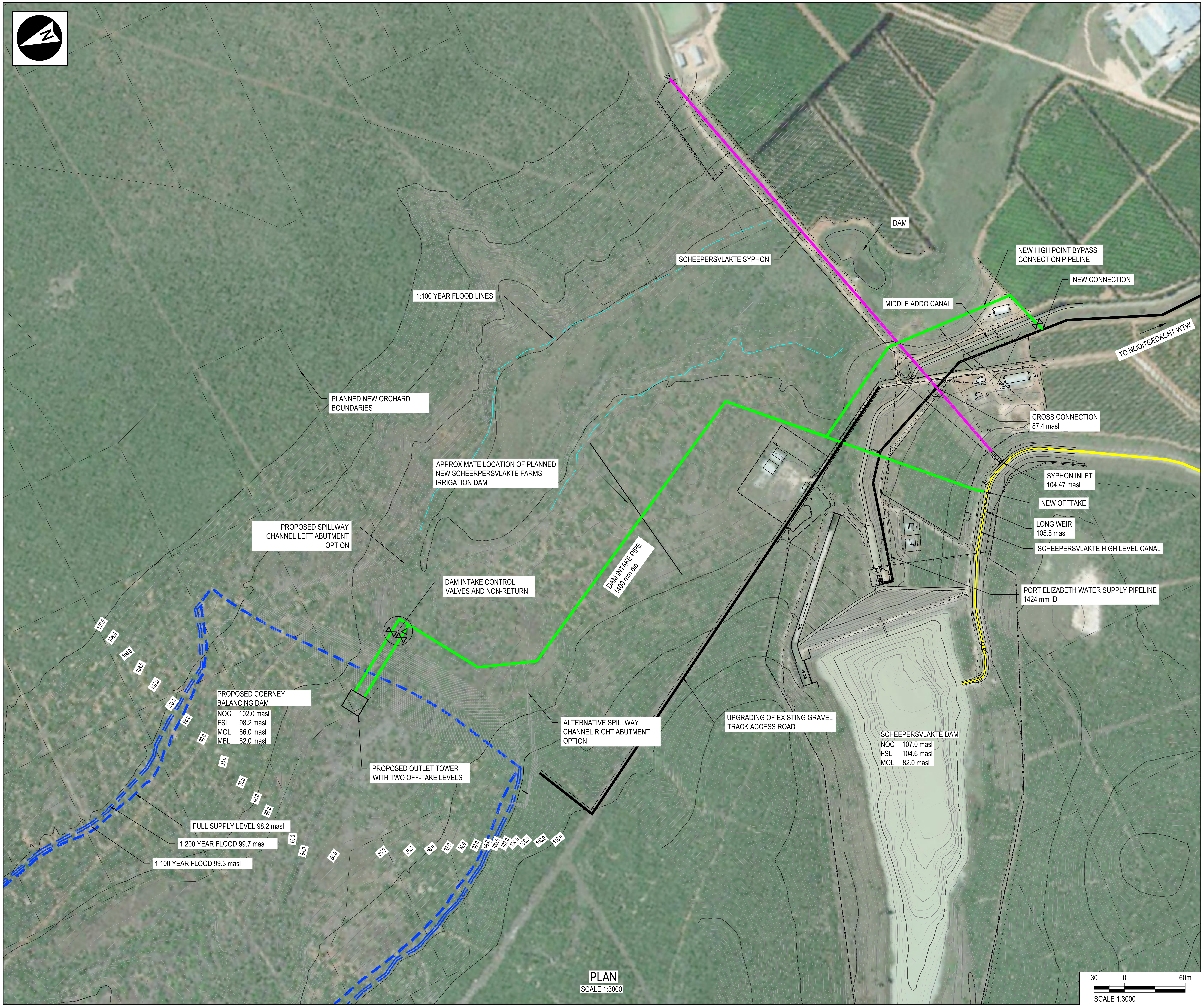
Type of dam	Minimum total freeboard (m)	Minimum difference in level between stillwater RDF surcharge level and non-overspill crest (m)
Earthfill (Category I)	0.8	0.5
Earthfill (Categories II & III)	0.0	1.5
Rockfill (Categories II & III)	0.0	1.5
Concrete (Categories II & III)	1.5	1

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

Appendix B: Drawings



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

aurecon
www.aurecongroup.com

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: AS SHOWN | SIZE: 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

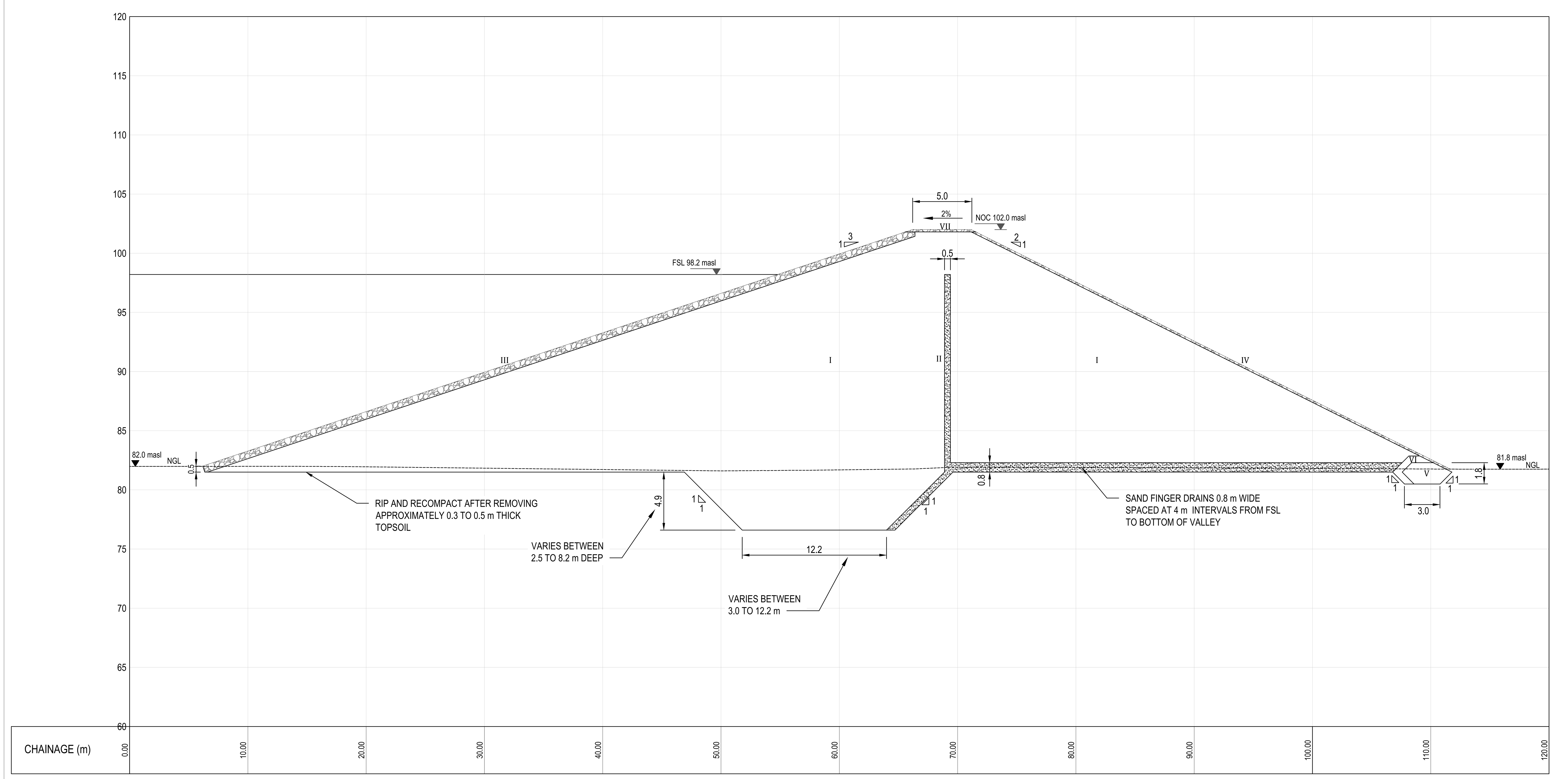
TITLE

COERNEY BALANCING DAM LAYOUT PLAN

DRAWING NUMBER

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

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

aurecon
www.aurecongroup.com

CLIENT

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

SCALE	SIZE
AS SHOWN	A1

FOR INFORMATION

DRAWN
A. KNOX

DESIGNED
O. HUMAN

CHECKED
F. DENEYS

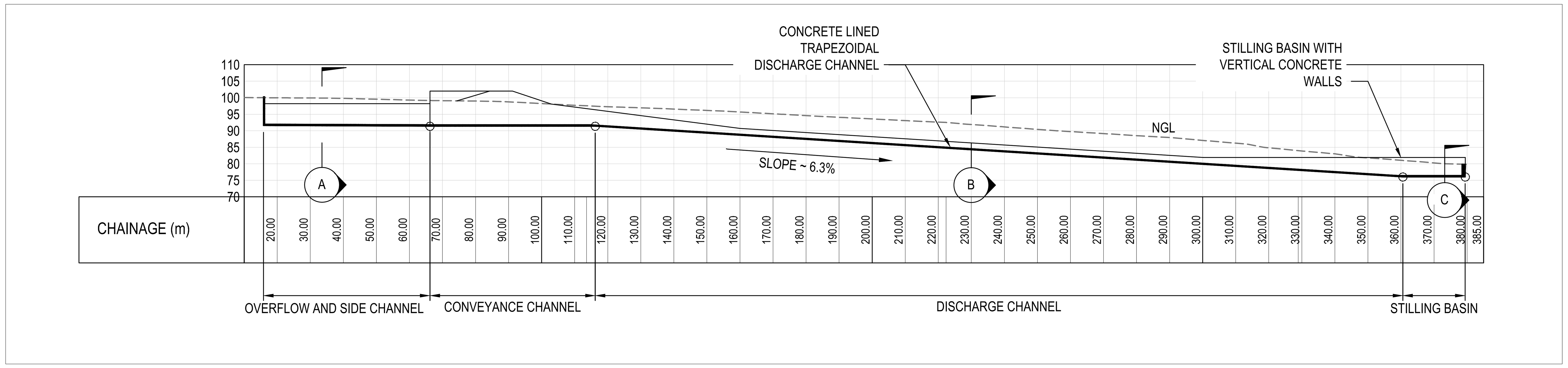
APPROVED
DATE
E. VAN DER BERG
ECSA
920455

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

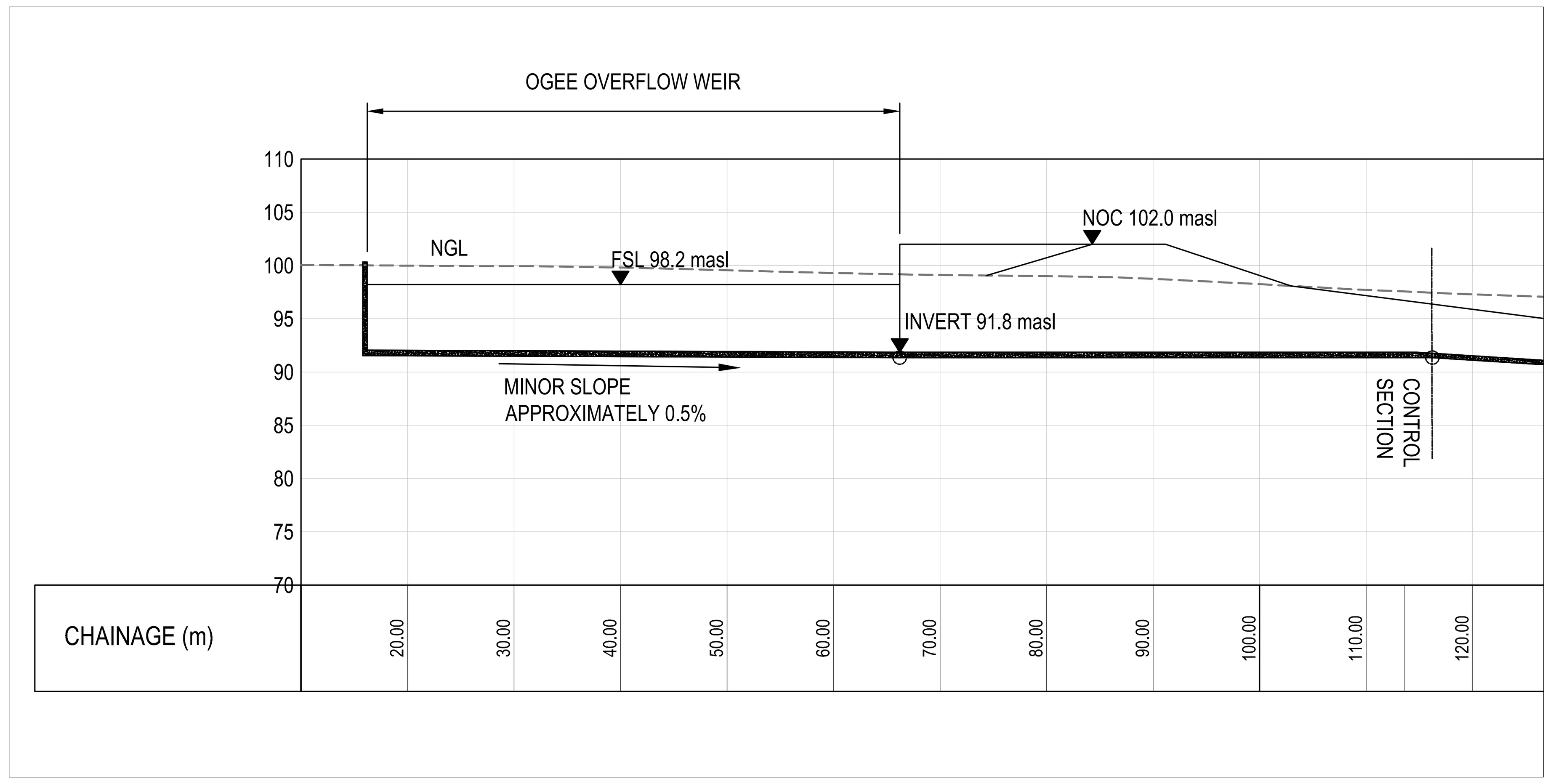
TITLE
LOWER COERNEY BALANCING DAM EMBANKMENT SECTION

DRAWING NUMBER

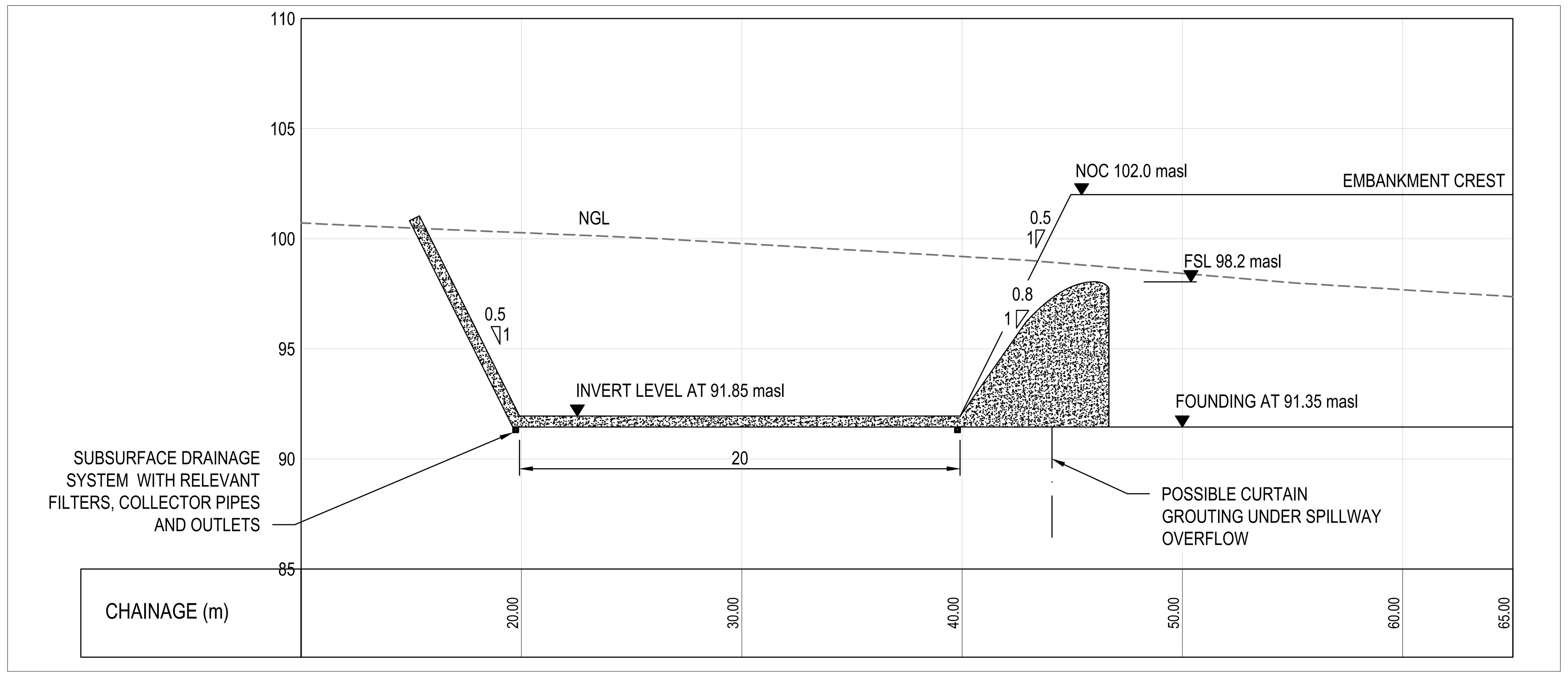
PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
112546	0000	DRG	CC	003	A



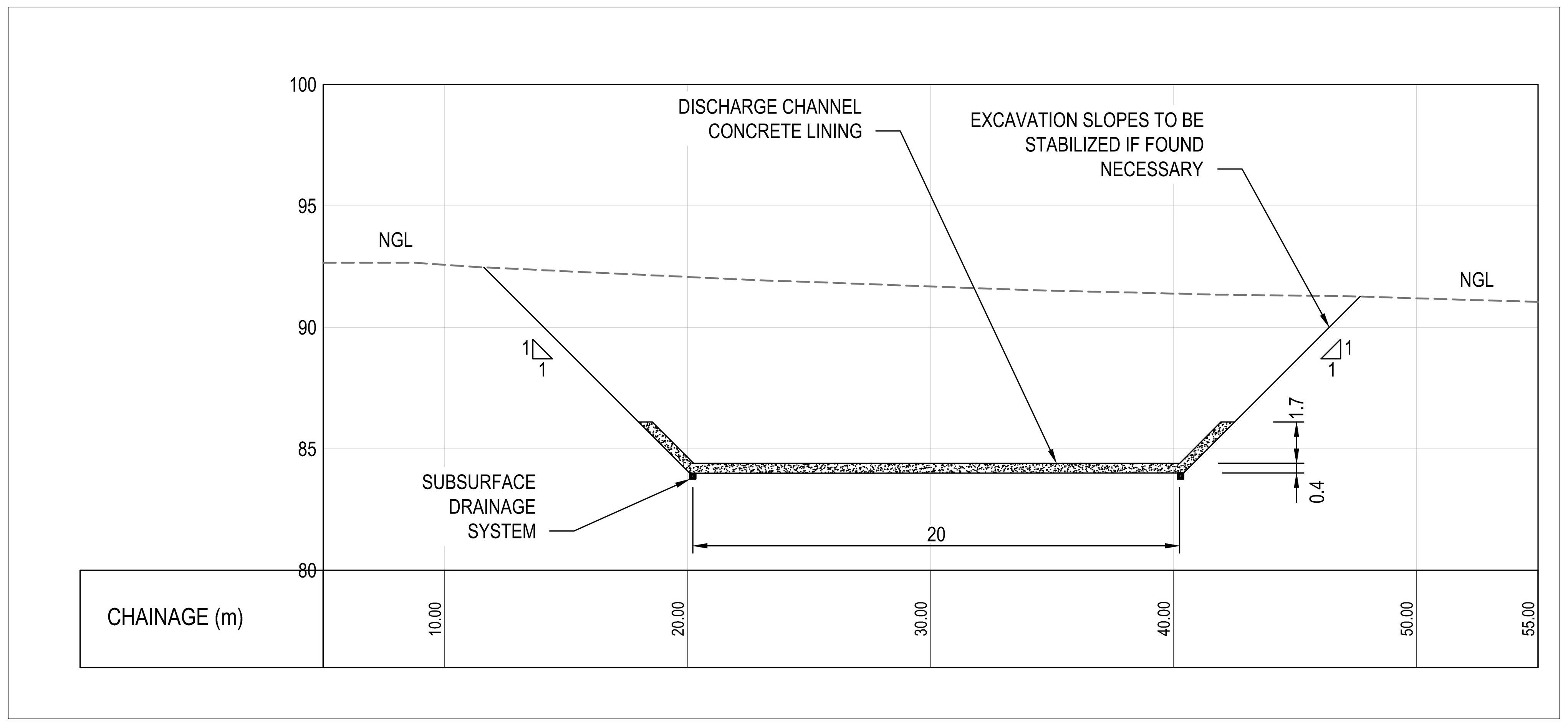
LONG SECTION THROUGH SPILLWAY
1:1000



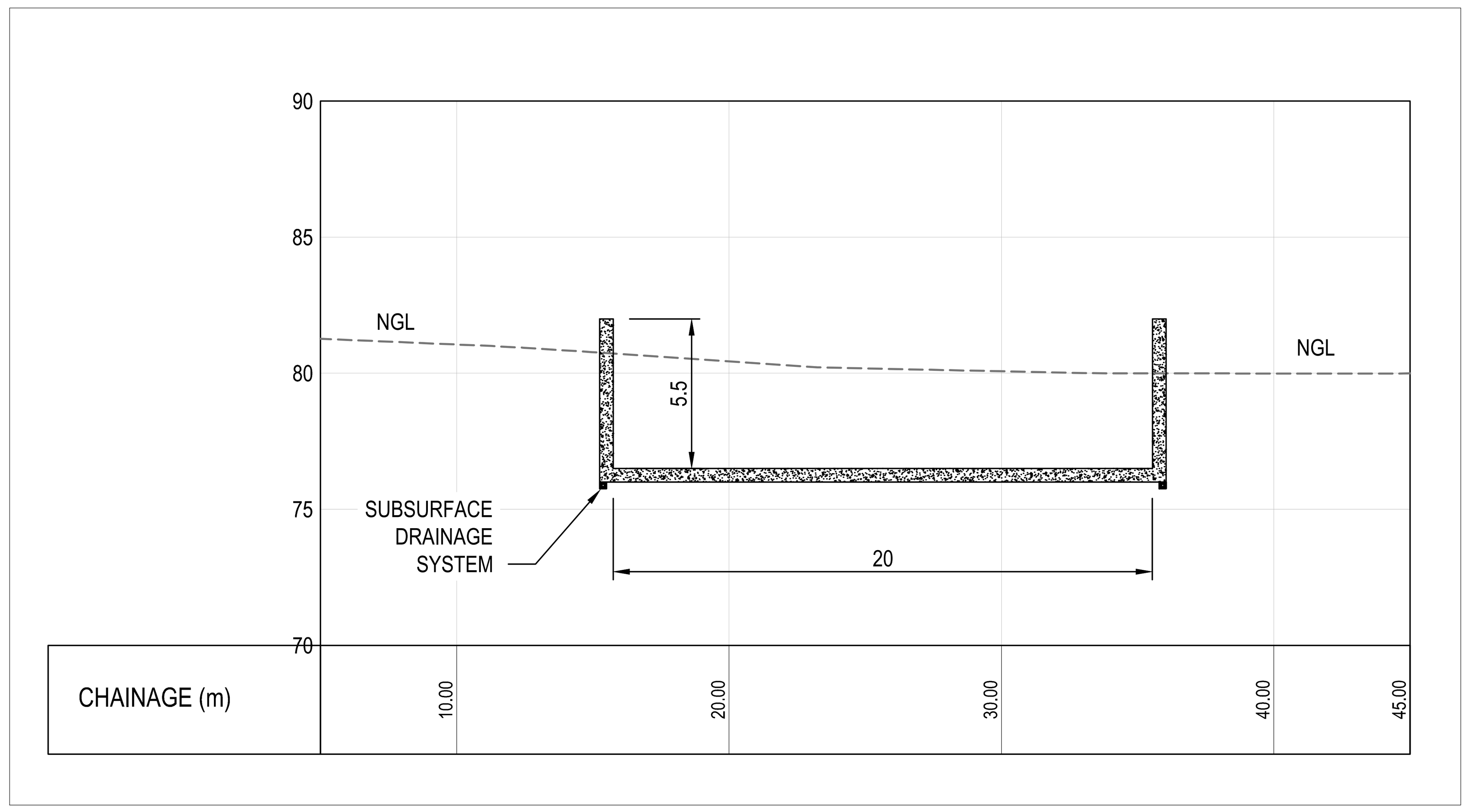
SECTION AT SPILLWAY OVERFLOW
1:500



DETAIL A 1:200 **SPILLWAY SIDE CHANNEL SECTION**



DETAIL B 1:200 **SPILLWAY DISCHARGE CHANNEL SECTION**



DETAIL C 1:200 **SPILLWAY STILLING BASIN SECTION**

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

www.aurecongroup.com

CLIENT

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

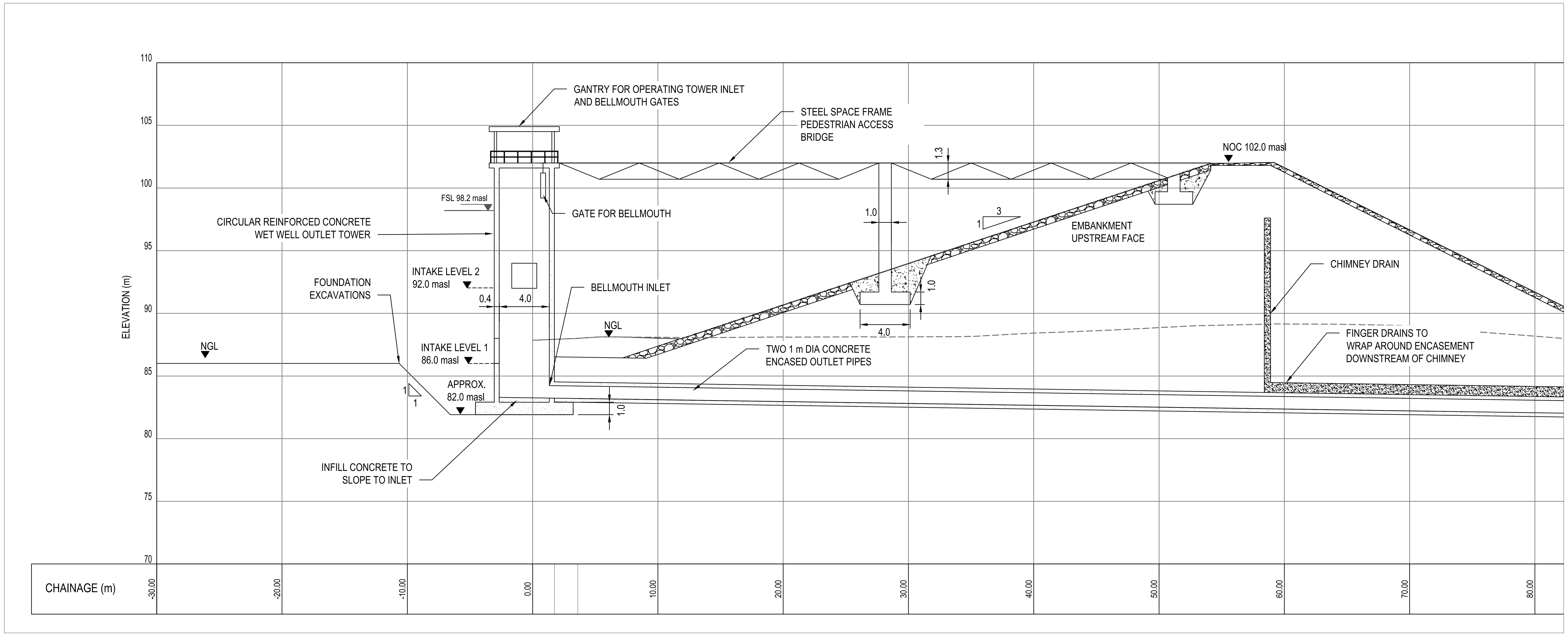
SCALE: AS SHOWN
SIZE: A1
FOR INFORMATION
DRAWN: A. KNOX
DESIGNED: O. HUMAN
CHECKED: F. DENEYS
APPROVED: E. VAN DER BERG
DATE
ECSA 920455

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

TITLE
LOWER COERNEY BALANCING DAM SPILLWAY SECTION

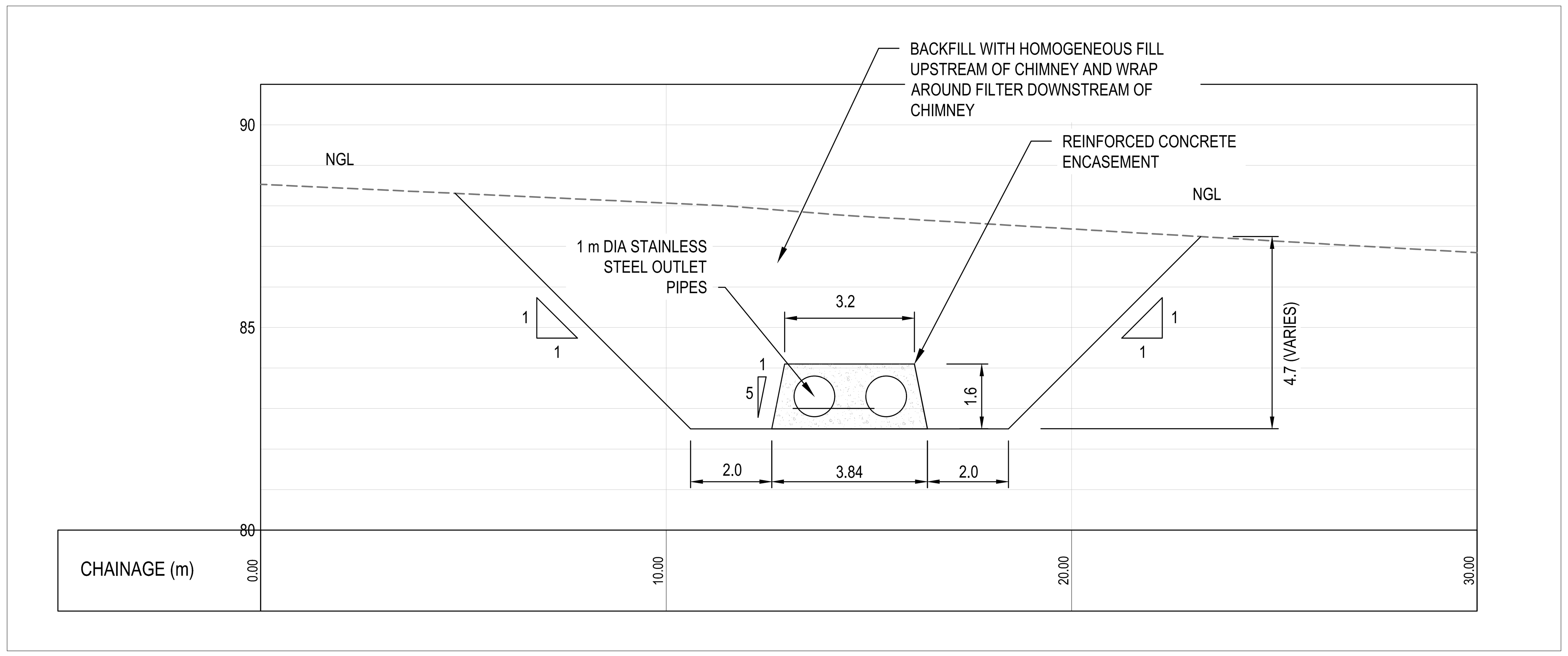
DRAWING NUMBER

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

aurecon
www.aurecongroup.com

CLIENT

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

Aurecon South Africa (Pty) Ltd
1977/003711/07
Aurecon Centre
1 Century City Drive
Waterford Precinct
Century City
Cape Town
7441
PO Box 494
Cape Town
8000
South Africa
T +27 21 526 9400
F +27 21 526 9500
E capetown@aurecongroup.com
W aurecongroup.com