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TABLE OF CONTENTS

APPENDICES

List of Tables

LIST OF FIGURES

ABBREVIATIONS

EPCM	EPCM Consultants SA
SoW	Scope of Work
WUL	Water Use Licence
WULA	Water Use Licence Application
Eskom	Eskom Holdings SOC Ltd
ADF	Ash Disposal Facility
PCD	Pollution Control Dam
CWD	Clean Water Dam
DWS	Department of Water and Sanitation
DEA	Department of Environment
DWAF	Department of Water Affairs (now DWS)
ROD	Record of Decision
MAP	Mean Annual Precipitation
SMP	Stormwater Management Plan
ECSA	Engineering Council of South Africa
GCL	Geosynthetic Clay Layer
GM	Geomembrane
GT	Geotextile

GRI	Geosynthetic Research Institute
HDPE	High density polyethylene
TLB	Tractor Loader Backhoe

EXECUTIVE SUMMARY

Kusile Power Station, located approximately 40 km from Bronkhorstspruit in the Mpumalanga Province, is a coal fired power station, owned and operated by Eskom Holdings SOC (Ltd). Construction of the power station began in 2008, and the first power production started in 2017. Coal is burned in large boilers, and the heat generated in the burning of the coal drives the electricity generation process. The residual material from the coal-burning is ash.

The power station will utilise dry ashing facilities for the disposal of its ash. Studies have indicated that the current ash/gypsum co-disposal facility is inadequate to accommodate the expected volume of ash to be generated during the life of the station. It’s carrying capacity will be limited to less than 10 years if both gypsum and ash are disposed on the facility.

An additional facility is therefore required to accommodate ash disposal for the 60-year design life of the station. Once the first phase of the ADF has been completed, the two waste streams of ash and gypsum will be permanently separated. Thus, the current co-disposal facility will be utilised exclusively for the storage of gypsum, for the 60-year design life of the station.

Eskom Holdings SOC Ltd. (Eskom) appointed EPCM to provide professional services for Basic and Detail Design of the 60 Year Ash Disposal Facility (ADF) at the Kusile Power Station. Under the current EPCM contract, and described in this report, is the detailed design for the construction of Phase 1

It is envisaged that the facility will include the following infrastructure:

- A conveyor belt system for the transportation of ash from the power station to the ash disposal site/facility;
- A barrier containment system;
- Contaminated and clean water management systems;
- Stormwater management including stream diversion pipelines and attenuation dams;
- Site services for the facility;

- Office facilities for a contractor to operate and maintain the facility;
- Auxiliary supporting systems (Dust Suppression, Irrigation, Electrical, Control and Instrumentation).
- Pump houses and pipe reticulations.

The new facility has been designed in compliance with the new ash area's Integrated Environmental Authorisation issued by the Department of Environmental Affairs (DEA) and the Water Use Licence requirements, with amendment to the latter to be issued by the Department of Water and Sanitation (DWS) and supported by the information contained in this Design Report.

Catchment Details

The Kusile Power Station and ADF facilities fall within the Olifants Water Management Area. Within this area, the facilities fall within the **B20F** quaternary catchment. The site is characterised by slightly undulating topography, with elevations ranging from 1441 m.a.m.s.l. in the north to 1515 m.a.m.s.l. in the south. The average slopes are about 3% with a dominant slope toward the Klipfonteinspruit.

Concept Design

A conceptual design was prepared by Jones & Wagener Consulting Civil Engineers, to support the Environmental Authorisation application for the project. The dump geometry and ancillary works were developed based on a set of design criteria such as ash production figures, site constraints, operational requirements and storm water management.

A Zitholele JV project team then developed the basic and detailed engineering designs based on the detailed concept design, which was deemed to be an approved concept. Under the current EPCM contract, and detailed in this report, a Detailed Design for the construction of Phase 1 has been documented.

Waste Classification

The ash the waste is a **Type 3** waste (low hazard waste) and therefore requires a **Class C Barrier**, in accordance with DWS' National Norms and Standards for Disposal of Waste to Landfill NEM:WA R636.

ADF and Ancillary Works

The growth plan has been developed based on the stacker system. Discussions were held with the Eskom Bulk Materials Handling (BMH) team to optimise the conveyor configuration. This report forms the basis of the engineering design for Eskom's CoE teams.

The growth plan is based on pre-deposition works for Phase 1 to commence in early 2026. The design anticipates a 3 year construction period, however, the facility is required to receive first ash by October 2026. Thus the construction of the pre-deposition works within Phase 1 Stage 1a and Stage 1b areas (as shown in drawing 366-511845) would need to be prioritised prior to the receiving of first ash. It would take approximately 1 year to construct the bottom stacker platform. During the commissioning of the bottom stacker and conveyor system the initial top stacker platform would be constructed.

The bottom stacker conveyor system commences with parallel shifting at which stage the top stacker will be assembled. Once the top stacker initial platform is completed the assembly of the stacker machine commences. The platform would be extended over the next 20 years, after which, the shifts for the Shiftable Conveyor commence. The ADF airspace volume is designed for 60 years life at full ash production with a 5 year decommissioning period. The operational life of the facility ends around the year 2090.

The ADF footprint extensions are prepared in phases varying between 4 and 10 years to align with conveyor shift positions. The pre-deposition works consists of top soil stripping, base preparation, lining and related infrastructure.

Stormwater Management

Stormwater management will include attenuation dams, diversion drains, lined concrete trapezoidal drains of ADF perimeter, and a stream diversion pipeline, together with various erosion protection measures and stilling basins. Contaminated water flows to PCDs fitted with sil raps, and clean water is discharged to the environment or stored in clean water dams.

Water Balance

An integrated water balance model has been developed in the concept phase and was used to size the dams required for storm water management. There are four main pollution control dams, PCD 1A, PCD 1B, Road PCD and PCD 2, which by regulation are not allowed to spill to the environment more than once in a 50-year period. The rest of the dams are designed to overflow to the other dams within the system. The results of the water balance simulation indicate no spillages over the simulated period if transfer systems between all dams are incorporated under the conditions as defined in this report. The system is therefore in compliance with the requirements of the governing regulation, GN 704, for the control of storm water at the facility.

The system is however dependent on operational conditions in the form of water abstraction parameters, dam operating levels, water transfer capacity, etc. These parameters have been taken into consideration in the design process with Eskom's LPS CoE and they will be clearly defined in the Operations and Maintenance manual.

Geotechnical

A geotechnical investigation was conducted during the basic design stage of the project and it indicated that the ground conditions are variable but the following general soil profile can be expected across the majority of the site:

- Topsoil to a depth of about 0.3m
- Overlying colluvium over the majority of the site that is not suitable engineered fill,
- Depth to weathered rock is about 2m, within the central one-third of the ADF footprint,
- Beyond this central portion, rock has been encountered at depths generally between 3m and 7.5m (i.e. rock was commonly not encountered within the excavation depth of the test pits), with various rock types being encountered.

The analysis of the field observations made during the investigation indicates that the site is suitable for the construction of the proposed ADF provided that certain design considerations are adhered to.

Integrated Environmental Authorisation

On the 17th of July 2015, DEA granted an Integrated Environmental Authorisation. The authorisation number is: 12/12/20/2412. The authorisation also encompasses the approval of the Environmental Management Programme (EMPr). The Water Use License (WUL) was granted by the Department of Water and Sanitation. The updated design included in this report will be submitted to the Department in support of an Amended Water Use Licence.

Procurement Strategy

Eskom normally utilises its subsidiary company, Eskom Rotek Industries (ERI), for construction, maintenance, and transportation services in support of its operations. During Basic Design stage, it was indicated that ERI would execute the construction works and eventually operate the facility. At that time, the costing for the project was based on ERI's construction prices on similar ash disposal facility projects for Eskom.

It is now understood that the procurement strategy has shifted to an open and competitive tender process. It should be noted that prices from the competitive open tender could vary significantly. A Bill of Quantities and Engineer's Estimate have been prepared.

Project Investment

An estimate of the expected construction value of the project was compiled based detailed bill of quantities for Phase 1 and attached in Appendix G.

1. TERMS OF REFERENCE

EPCM Consultants SA (Pty) Ltd has been appointed by Eskom Holdings (SOC) Ltd to review existing design, develop Detailed Design for Phase 1 and consolidate all required information for the 60-year Ash Disposal Facility Design Report, to support the amended Water Use Licence Application.

2. BACKGROUND INFORMATION

2.1 Project Background

Kusile is a coal fired power station, owned and operated by Eskom Holdings SOC (Ltd). Construction of the power station began in 2008, and the first power production started in 2017. Currently 4 units of the planned 6 units have been constructed and have been connected to the national grid. Coal is burned in large boilers, and the heat generated in the burning of the coal drives the electricity generation process. The residual material from the coal-burning is ash.

The power station will utilise dry ashing facilities for the disposal of its ash. Studies have indicated that the current ash/gypsum co-disposal facility, which received Environmental Authorisation with the power station in March 2008, is inadequate to accommodate the expected volume of ash to be generated during the life of the station. It's carrying capacity would be limited to less than 10 years if both gypsum and ash are disposed on the facility.

An additional facility is therefore required to accommodate ash disposal for the 60-year design life of the station. Once the first phase of the ADF has been completed, the two waste streams of ash and gypsum will be permanently separated. Thus, the current co-disposal facility will be utilised exclusively for the storage of gypsum, for the 60-year design life of the station.

Eskom Holdings SOC Ltd. (Eskom) appointed EPCM to provide professional services for Detail Design of the 60 Year Ash Disposal Facility (ADF) at the Kusile Power Station: Under the current EPCM contract, and described in this report, is the detailed design for the construction of Phase 1.

The new ADF facility has been designed in compliance with the facility's Integrated Environmental Authorisation issued by the Department of Environmental Affairs (DEA) and the Water Use Licence requirements, to support an amended Water Use Licence Application (WULA) that would be issued by the Department of Water and Sanitation (DWS). The construction works and operations of the facility are to follow suit and also comply to the licence requirements.

The scope of new infrastructure that will be required includes the following:

- Conveyor system for the transportation of ash from the power station to the new ADF.
- Barrier containment / liner system for the ADF.
- Stormwater, contaminated water, and clean water management systems.
- Site services.
- Office facilities for a contractor to operate and maintain the facility.
- Support systems for dust suppression, irrigation, electrical, control and instrumentation.

The ADF is to be developed in phases, with the extension of the ADF at approximately five-year intervals. Eskom's objective is to construct and commission the proposed ADF so that ash produced by the power station over its design life of 60 years can be disposed in a safe and responsible manner.

An authorisation for the Ash Disposal Facility (ADF) was issued in 2015. In order to commence with the development, Eskom Holdings SOC Limited applied for a Water Use Licence for the applicable water uses, in terms of the National Water Act, 36 of 1998, as amended.

2.2 Scope of report

The scope of this report for Phase 1 Design includes the following:

- Background information on site conditions,
- ADF design, including barrier design and drainage design,
- Design of PCDs and CWDs with silt traps,

- Stomrwater management,
- Stream diversion pipeline,
- Catchment analyses for runoff volumes and peak flows,
- Upslope runoff management including Attenuation Dams,
- Geotechnical considerations,
- Proposed stream diversion pipeline specification,
- Construction Quality Assurance Documentation,
- Preliminary Operating Plan,
- Water Balance for the ADF,
- Preliminary cost estimates.

Note that aspects of the design take into account Phases 2 to 12 of the ADF facility, where this influences the tie-in points and capacities of Phase 1 infrastructure. The report excludes design of roads and bridges, as well as mechanical and electrical engineering aspects.

2.3 Objectives of this Report

The objective of this Design Report is to document the design criteria and details for Phase 1 of the ADF providing sufficient information to support both the IWULA amendment and Phase 1 Tender Engineer’s cost estimate.

Previous comments from the Record of Decision (ROD), together with comments from the EPCM reviews will be taken into account.

The National Department of Water and Sanitation (DWS) issued a revised document in November 2020 pertaining to waste disposal applications, namely, “*National Environmental Management Waste Act Regulations 2013: Basal Barrier System Checklist for the Lead authority (National or Provincial Government) in Advance of Document Submission to Commenting Authority*”. Information in the Design Report will seek to fulfil these checklist requirements.

2.4 Design Engineer, Peer Review

The project Design Engineer will be Mr. Kris Matulovich, Professional Engineer with ECSA Registration number 20190729.

Contact Details: 011 425-2810, Email: kris@envitech.co.za

An internal peer review was undertaken for the final design report.

2.5 Approved Professional Person (APP)

Where dam wall heights exceed 5m or where capacity exceeds 50 000m^3, the design needs to be approved by a registered APP. This applies to dams PCD-1A, PCD-2, Attenuation Dams 1, 2, 3 and 5.

The final appointment of an APP is underway.

The Developer's Representative

The Developer's Representative will be Mr Samuel Mahlangu of Eskom Holding SOC (Ltd).

Contact Details: Cell: -- Tel: 013 699 7271 Email: MahlaSam@eskom.co.za

2.6 Project History to Date

A conceptual design of the ADF was prepared by Jones and Wagener Consulting Engineers in 2013 in support of the Environmental Authorisation application for the ADF. Thereafter a Zitholele JV project team developed the basic detailed design in 2018, which is deemed to be an approved concept. An Environmental Authorisation was granted by the DEA in July 2015.

EPCM have been tasked to review the design and oversee the ADF Detailed Design, which will also support an amended Water Use Licence Application. Phase 1 of the Detailed Design is submitted in this report.

The upslope stormwater cut-off drain is currently under construction under the "Early Works" Package.

3. ENGINEER'S ESTIMATE

3.1 Procurement Strategy

Eskom normally utilises its subsidiary company, Eskom Rotek Industries (ERI), for construction, maintenance, and transportation services in support of its operations. During Basic Design stage, it was indicated that ERI would execute the construction works and eventually operate the facility. At that time, the costing for the project was based on ERI's construction prices on similar ash disposal facility projects for Eskom.

If the procurement strategy shifts to an open and competitive tender process then it should be noted that prices from the competitive open tender could vary significantly. A current Bill of Quantities (BoQ) has been prepared, based on general market rates for large scale projects, as well as an Engineer's Estimate. The BoQ is attached in **Appendix G**.

Whilst the BoQ would be issued to tenderers to assist in pricing and measurement, the Contractor is to be paid at set milestones i.e. based on the Activity Schedule (as definited and set out by the contractor during pricing) rather than on a re-measurable BoQ basis.

3.2 Bill of Quantities

An Engineer's Estimate of the development cost will be undertaken and cognisance will be taken of material specifications and methodologies that allow for competitive sourcing and costing of materials and methods. The Estimate will be included in the final Design Report **Appendix G**, and is considered confidential.

3.3 Activity Schedule

A summary of the proposed activity schedule for Phase 1 can be found in **Appendix H**. It is important to note the required completion date of an accepted lined, facility to receive first ash from the power station.

3.4 Declaration of Interest

There is no affiliation between Design Engineer and any material suppliers.

4. LEGISLATIVE AND OTHER REFERENCES

4.1 Integrated Environmental Authorisation

Zitholele Consulting (Pty) Ltd undertook the Integrated Environmental Authorisation, which included an Environmental Impact Assessment (EIA), a Waste Management Licence (WML) application and the Water Use Licence Application (WULA) as required for the proposed construction, operation and decommissioning of the project.

On the 17th of July 2015, DEA granted an Integrated Environmental Authorisation. The authorisation number is: 12/12/20/2412, and it is attached in **Appendix A**. The authorisation also encompassed the approval of the Environmental Management Programme (EMPr). The updated and detailed design included in this report will be submitted to the Department in support of an Amended Water Use Licence.

4.2 Applicable legislation

The applicable legislation includes, but is not limited to, the following:

- National Water Act, 1998 (ACT NO. 36 OF 1998);
- Regulations on Use of Water for Mining and Related Activities Aimed at the Protection of Water Resources, Government Notice No. 704, Government Gazette, 4 June 1999 (Vol. 408, No. 20119);
- Dam Safety Regulations, Government Notice No. R 139, Government Gazette, 24 February 2012;
- Minimum Requirements for Waste Disposal by Landfill, Department of Water Affairs and Forestry, Second Edition, 1998;
- National Environmental Management Act, 1998 (Act No. 107 of 1998);
- National Norms and Standards as published in Government Gazette Notices 634, 635 and 636 of 2013
- The National Department of Water and Sanitation (DWS) document “National Environmental Management Waste Act Regulations 2013: Basal Barrier System Checklist for the Lead authority (National or Provincial Government) in Advance of Document Submission to Commenting Authority”, November 2020.

- Mineral and Petroleum Resources Development Act, 2002 (Act No. 28 of 2002);
- A4 Best Practice Guidelines for Water Resources Protection in South African Mining Industry;
- Best Practice Guidelines for Water Resource Protection in the South African Mining Industry –A4: Pollution Control Dams, Department of Water Affairs and Forestry, August 2007;
- The South African National Roads Limited (SANRAL), 2006, “Drainage Manual”, 5th Edition.

4.3 Eskom Standards and Other References

Various Eskom Standards have reference:

- ESKOM Zero Liquid Effluent Discharge (ZLED) philosophy.
- Eskom Standard - 240-57127951: Standard for the Execution of Site Investigations;
- Eskom Standard - 240-57127953: Execution of Site Preparation and Earthworks, February 2013;
- Eskom Standard - 240-57127955: Geotechnical and Foundation Engineering Standard, April 2015;
- Eskom Standard - 240-55864300: Dam Design, January 2013;
- Eskom Standard - 240-56364545 - Structural Design and Engineering, March 2015;
- Eskom Standard - 240-85549846: Design of Drainage and Sewerage Infrastructure, March 2015;
- Eskom Standard - 240-76368574: High Security Mesh Fencing Standard (Reg), April 2014;
- Eskom Standard – 203-770: Kusile Specification for Structural Concrete October 2009.

The following documents form part of the detail design and are referenced in this report for more information.

- SANS 1200 – Standard Specifications for Civil Engineering Construction, various;
- SANS 10409 – 2020 Design, Selection and installation of Geomembranes;
- SANS 10160-Parts 1 to 8: Basis of structural design and actions for buildings and industrial structures;
- SANS 10400: The application of the National Building Regulations (as applicable);
- SANS 10100-1: The structural use of concrete;
- SANS 10161: The design of foundations for buildings;
- SANS 10162-Parts 1 and 2: The structural use of steel;
- SANS 10164-1: The structural use of masonry;
- SANS 2001-CS1: Construction works – Structural steelwork;
- GRI Standards for lining components. (GM3, GCL3, GT13)

5. SITE OWNERSHIP

The site of the Kusile Power Station is located over several portions of the farm Klipfontein 566 JR and is owned by Eskom Holdings Ltd.

6. SITE DESCRIPTION

6.1 Location and access

The Kusile Power Station is located 40 km from Bronkhorstspruit and 18 km north of Ogies in Mpumalanga Province of South Africa. Access is off the R686 regional road that runs to the west and then north of the Site, which in turn can be accessed from the N4 via the R545. The ADF site is south of the Kusile Power station on open land that was under crop farming and animal grazing until relatively recently. The location co-ordinates of the centre of the ADF are tabled below. The ADF covers approximately 740 hectares.

Table 1: Site Location Co-ordinates

POINT	LATITUDE	LONGITUDE
Centre of ADF	25^0 57′ 39″ South	28^0 54′47″ East

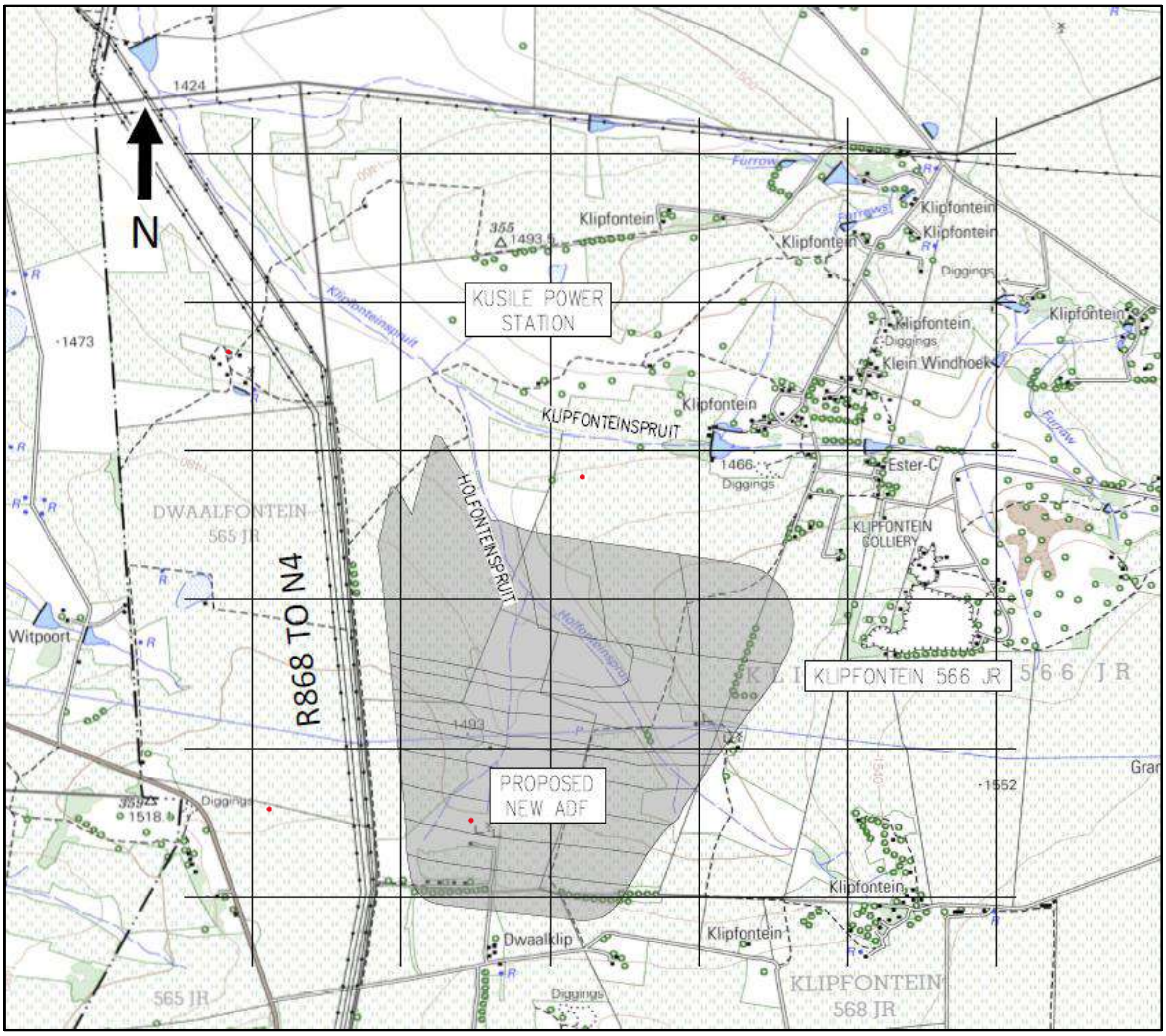


Figure 1: Locality Plan, Extract of 1:50000 Topographocal Map No.2528DD

6.2 Topography

The site is characterised by slightly undulating topography, with elevations ranging from 1441 m.a.m.s.l. in the north to 1515 m.a.m.s.l. in the south. The main drainage channel on the site is the Klipfonteinspruit, running from east to west to the north of the ADF and dams complex. This stream is fed by the Holfonteinspruit, a non-perennial stream from the south-west, and it's tributary from the south that run diagonally across the proposed ADF footprint. The average slopes are about 3% with a dominant slope toward the Klipfonteinspruit.

6.3 Quaternary catchment

The Kusile Power Station and ADF facilities fall within the Olifants Water Management Area. Within this area, the facilities fall within the **B20F** quaternary catchment, shown in **Figure 2** below. The Klipfonteinspruit passes between the Power Station and the ADF, flowing in a westerly direction, ultimately joining the Wilge River system.

The Holfonteinspruit and a tributary flow northwards to join the Klipfonteinspruit. The proposed ADF will be positioned over the Holfonteinspruit and it's tributary, as shown in **Figure 3** and drawings.

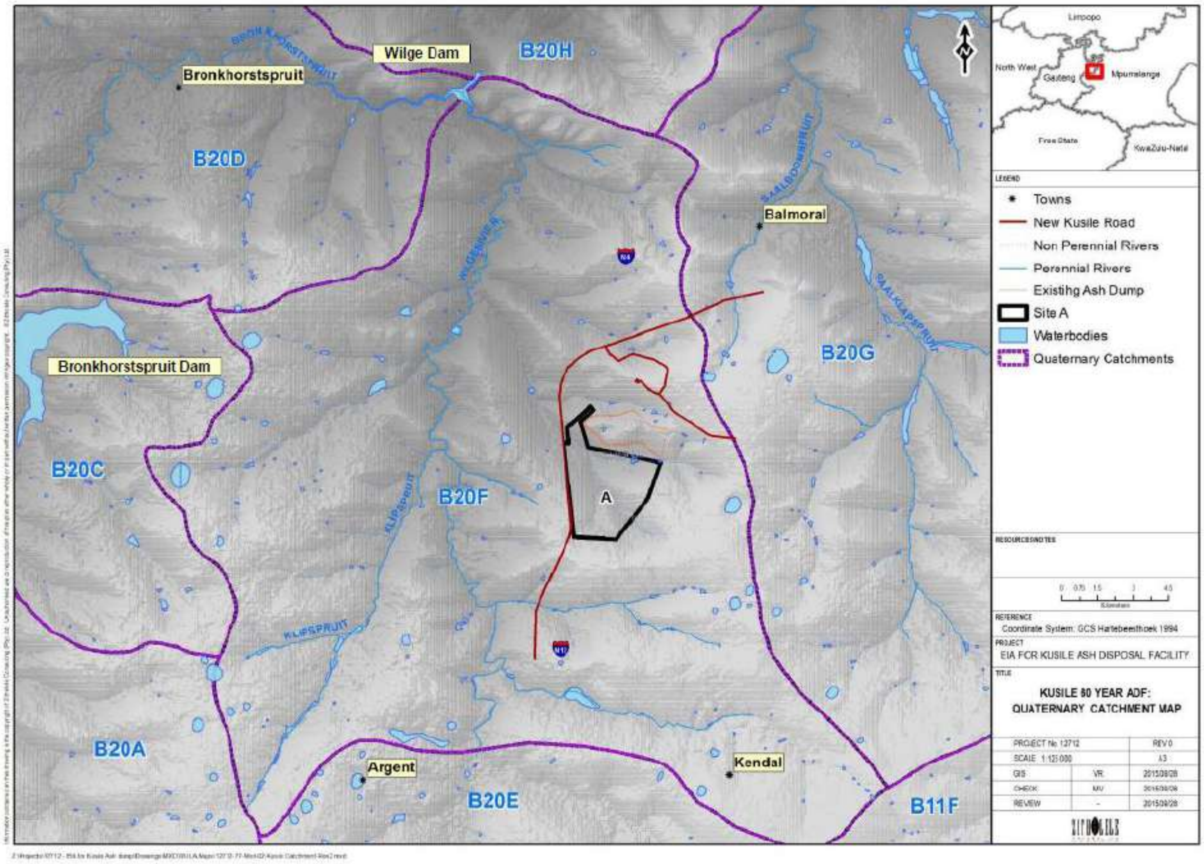


Figure 2: Quaternary Catchment, Zitholele 2014

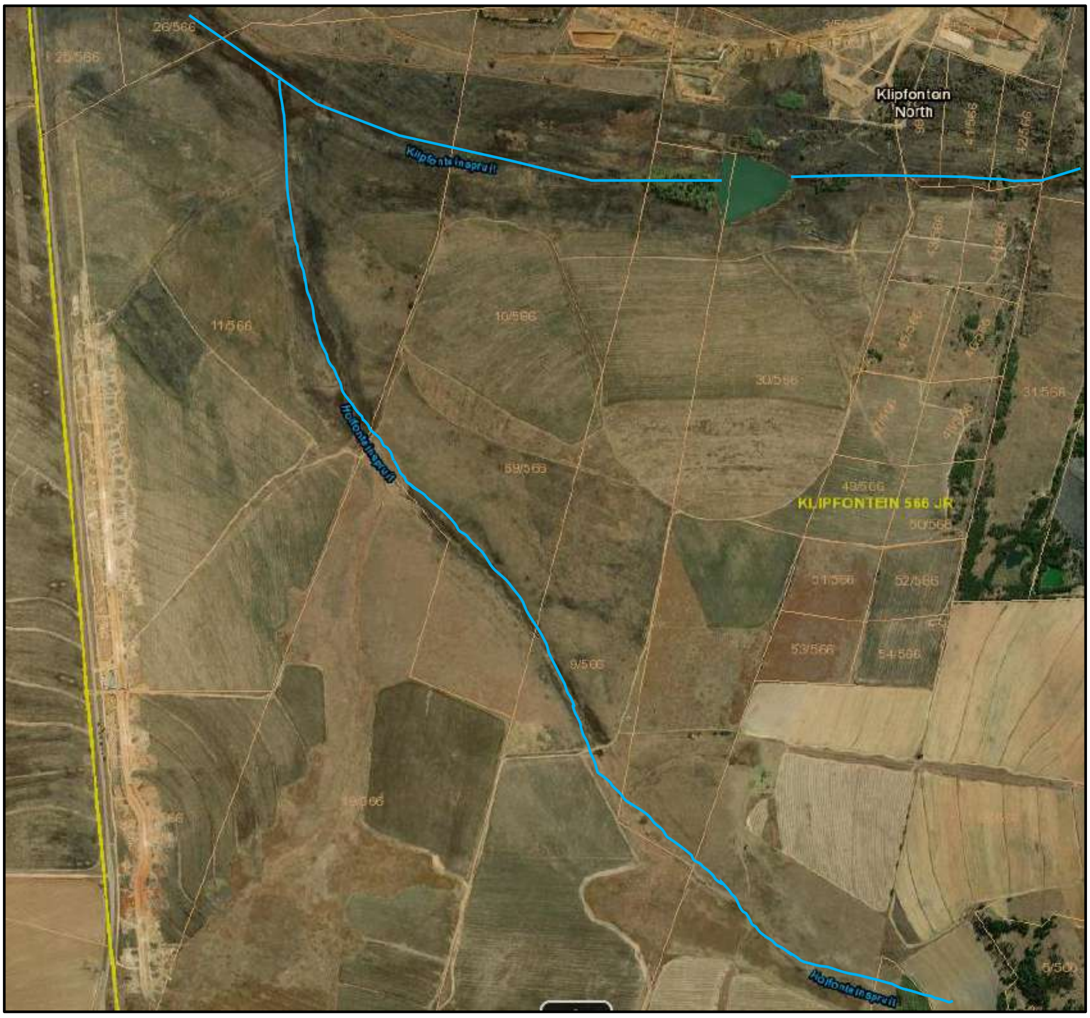


Figure 3 - Rivers within the ADF site footprint

6.4 Regional Geology & Hydrogeology

According to the 1:250 000 geological map 2528 Pretoria the regional lithostratigraphy of the site comprises of the Ecca and Dwyka Groups of the Karoo Supergroup. The constituents of these sedimentary formations are predominantly the tillites and shales of Permian age. The site is underlain by shale, shale sandstone, grit sandstone, conglomerate with coal in places near the base and top from the Ecca Group with tillite and shale from the Dwyka Group of the Karoo Super Group, with diabase intrusions that is of Vaalian and post-Mogolian age. Underlying these formations is the Silverton

Formation shale's that are carbonaceous in places with hornfels and chert from the Pretoria Group. The region is not underlain by dolomite.

6.5 Geotechnical investigations undertaken to date.

The available reports used for the analysis of the *Kusile 60 Year Ash Disposal Facility*, *Geotechnical Summary Report, Document no.: 366-513520 by EPCM, November 2023*, include the following:

- *Jones & Wagener/ Zitholele JW140/13/D121 rev2; Kusile Power station 60 year ADF: Engineering Detailed Concept Design Report, 2013&14*
- *Jones & Wagener/ Zitholele JW195/15/D121-07; Geotechnical Investigation for the proposed 60 yr ADF at Kusile. Geotechnical Investigation Report, 2015*
- *EPCM 366-490112 REVB; Kusile 60 Year Ash Disposal Facility, Geotechnical Analysis Report*, Document no.: 22020-TO1-CE-RPT-04-001 Rev. B 19/06/2023.

In last report data from WSP Golder, EPCM Bonisana, and Soiltecnix were also used and summarised with the existing soil profiles and laboratory data.

6.6 ADF Site Geology

An area-specific description of the geology is given below. This is a quite generalised summary due to the variability of the tillite composition. The Geotechnical Summary Report is attached in Appendix B.

<u>Stream bed where pipeline to be laid:</u>

0.8 – 2.4m	Clayey Silty Sand - Colluvium & Alluvium
3.0 – 4.9m	Silty Clayey Sandy Gravel but varies - Residual Tillite/Shale

Groundwater seepage between 0.7 to 3.4m below NGL.

Excavatability – Soft to Intermediate

<u>The PCDs/ CWDs area close to the Klipfonteinspruit:</u>

0.5 – 1.6m	Sandy Gravel - Colluvium & Alluvium
3.0 – 4.9m	Silty Clayey Sandy Gravel but varies - Residual Tillite/Shale

Groundwater seepage between 0.7 to 3.4m below NGL.

Excavatability – Soft to Intermediate, with some hard material expected in the Road Dam footprint.

The ADF Phase 1 area:

0.8 – 3.0m	Clayey Silty Sand - Colluvium & Alluvium
2.0 – 5.2m	Clayey Sandy gravelly Silt but varies - Residual Tillite/Shale

Seepage is evident in the form of springs in places.

Excavatability – Soft to Intermediate

6.6.1 Ground Water Levels

The water table on-site is relatively shallow and saturated conditions can be expected in low-lying areas. Groundwater seepage was recorded in various testpits over the site, at a depths varying from 0,7m to about 3,5m, which is generally the depth of weathered rock upper surface. The presence of ferruginisation in portions of the site is indicative of seasonal fluctuations in ground water seepage levels, which should be taken into account where sub-soil drains may be required. Groundwater flow is found to generally follow surface topography.

6.6.2 Topsoil

A loose structured topsoil layer of between 100mm and 300mm was observed on the site. This material is to be set aside for final rehabilitation in the progressive closure of the ADF stages. The area south of Phase 1 has been considered for a topsoil stockpile area.

6.7 Seismicity

According to the SANS Standard SANS 10160-4:2017 the Kusile site falls within Seismic Risk Zone 1 with a value of 0.100 m/s^2.

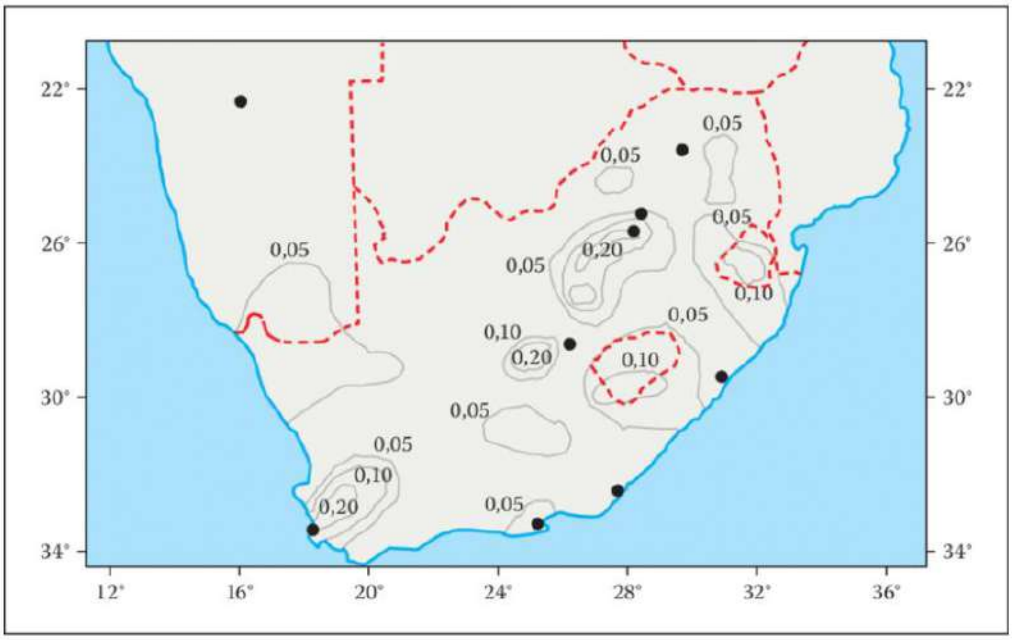


Figure 4: Seismic Risk Values

6.8 Regional Climate

The region falls withing the Highveld sub-tropical climate zone. The summers are characterised by hot, humid and wet conditions whilst the winters are characterised by lower tempeartures and Monthly precipitation levels are highest in January at approximately 124mm, whilst June is the driest month with about 7mm average precipitation.

The region expreiences frequent frosts in winter, and the area does receive hailstorms, usually in summer. The prevailing wind direction is north-westerly in summer and easterly in winter. Winds are usually light to moderate.

6.8.1 Mean Annual Precipitation

The Wilgerivier (SAR) 0514618_W weather station is the closest station to the study area that has reliable (95%) historic daily rainfall data, with records ranging from 1903 – 2000. The Mean Annual Precipitation (MAP) is reported as 697 mm per annum. Daily rainfall depths were extracted using the daily rainfall data extraction utility developed by Richard Kunz, from the Institute for Commercial Forestry Research (ICFR), in conjunction with the School of Bioresources Engineering and Environmental Hydrology (BEEH) at the University of KwaZulu-Natal, Pietermaritzburg, South Africa.

6.8.2 Rainfall and Evaporation Figures

The evaporation far exceeds precipitation and is expected to be around 1500 mm per annum. Mean monthly figures are provided below.

Table 2: Monthly Rainfall and Evaporation

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEPT	OCT	NOV	DEC	TOTAL
RAIN (mm)	**127,8**	**98,8**	**88,3**	**41,7**	**17,7**	**7,5**	**8,0**	**9,1**	**18,5**	**69,5**	**110,9**	**114,7**	**697**
S-Pan Evaporation	**166,1**	**143,5**	**135,4**	**105**	**85,3**	**67,4**	**74,6**	**102,4**	**139,5**	**163,1**	**160,3**	**174,9**	**1524**

6.8.3 Weather stations

The closest South African Weather Service (SAWS) weather stations are as follows in Table 3 below. The data from these stations are sourced automatically by the hydrology software and are also input into manual calculations such as the Rational Method, for catchment run-off analysis.

Table 3: Weather Station Data

Station	Year of Records	Co-ordinates	Distance from ADF
Wilgerivier	94	25^{O} 49′ S; 28^{O} 51′	< 1 km
Bronkhorstspruit	92	25^{O} 48′ S; 28^{O} 44′	12.7 km
Blesbokfontein	35	25^{O} 57′ S; 28^{O} 48′	15.4 km
Kleinwater	39	25^{O} 48′ S; 29^{O} 2′	19.9 km
Waaikraal	49	25^{O} 59′ S; 28^{O} 40′	26.8 km
Hartebeestspruit	59	25^{O} 46′ S; 29^{O} 7′	29.3 km

6.9 Existing Services and Structures

There are a few remnants of farm buildings, but no significant buildings on the property. There are old stock fences here and there on the ADF site and alongside the R686 road running along the western boundary. Security fences surround the Power Station on the northern bank of the Klipfontein Spruit. A boundary fence runs on most the eastern side of the ADF property, on the boundary with New Largo Mine and the Phola conveyor.

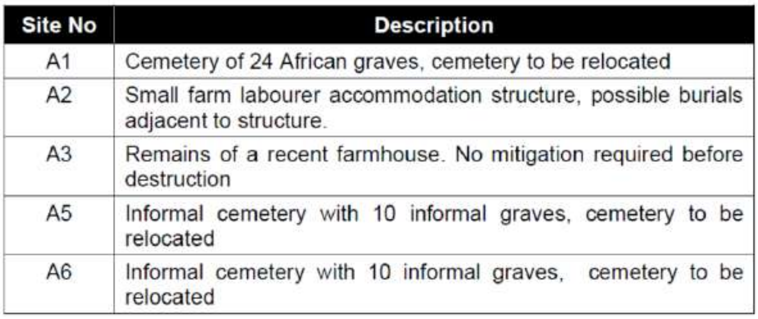
A 400 kV powerline runs alongside the R686 road to the west and a 22 kV overhead powerline traverses the site east-to-west about midway across the ADF site, which will have to be re-routed. The planned new route powerline route is shown on Drawing 366-511891.

There are a few gravel farm roads across the site and around the cut-off drain construction works.

6.10 Historical Graves

Graves have been found on site, shown in **Figure 5** below.

Table 4: Graves and Farm Structures



Site No	Description
A1	Cemetery of 24 African graves, cemetery to be relocated
A2	Small farm labourer accommodation structure, possible burials adjacent to structure.
A3	Remains of a recent farmhouse. No mitigation required before destruction
A5	Informal cemetery with 10 informal graves, cemetery to be relocated
A6	Informal cemetery with 10 informal graves, cemetery to be relocated

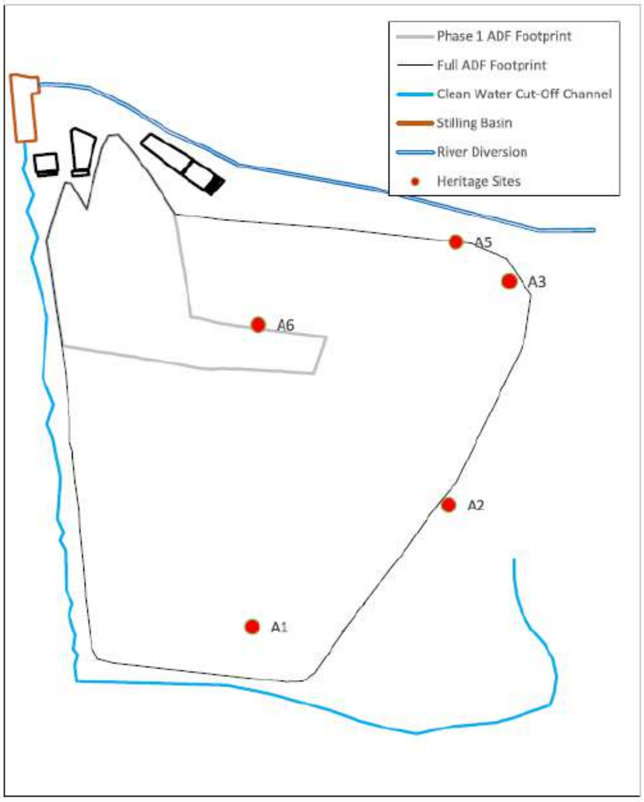


Figure 5: Location of Graves

7. CONCEPTUAL AND BASIC ENGINEERING DESIGNS

The conceptual design was prepared in August 2013 by Jones & Wagener Consulting Civil Engineers (Jones & Wagener) for the EIA submission Phase 1 of the project. The report is titled *Kusile Power Station 60 Year Ash Disposal Facility: Engineering Detailed Concept Design* and the report number is JW140/13/D121 – Rev A.

The design took into consideration the dry ashing operational philosophy which utilizes conveyor systems and ash stackers. The dump geometry and all required ancillary works were developed based on major criteria such as ash production figures, site constraints, operational requirements and stormwater management. The following key issues were addressed in the report:

- Site conditions and constraints,

- Footprint and lifespan of the facility,
- Height, source and volume of waste,
- Waste classification,
- The lining system,
- Clean and dirty water separation and containment infrastructure,
- River diversions,
- Pipelines
- Infrastructure relocations,
- Capping and rehabilitation,
- Access infrastructure,
- Development philosophy (phasing of the construction and operations works).

The basic design was approved by Eskom and report no. *15167-45-001-Kusile 60Yr ADF – Basic Design Report – Rev 2* was duly signed on the 30th of May 2017. Under basic design the concept design was further developed by, amongst other things, optimizing the utilization of the stacker systems, optimizing the geometry of the ADF and the pollution control dams.

8. DESIGN CONSTRAINTS

8.1 Existing Power lines

There are 22 kV and 220 kV powerlines that traverse across the proposed dump footprint from east to west. These powerlines need to be relocated and the project team is in the process of gathering information on the procedure for such relocations.

It has been indicated by Eskom's electrical representatives that the relocation of the powerlines will be undertaken by Eskom Distribution. The design team has provisionally indicated re-alignment routes based on the observed constraints. Drawing number 366-511891 has reference.

8.2 Main Kusile Access Road

A water pipeline traverses the western edge of the footprint along the main access road. It is located on the eastern side of the road which is the same side as the ash dump.

8.3 Planned Phola Conveyor

There are plans for a conveyor to be to be installed on the eastern boundary of the ADF site. The proposal is for the conveyor to transport coal from Phola coal processing plant to Kusile power station. The EIA process for the installation was initiated in the year 2010 but there have been no further developments.

The alignment of the conveyor would mostly affect the storm water management design on the south-eastern portion of the ADF.

8.4 Geological and hydrological setting

The test pits revealed the area to be highly variable by local region and by depth. It is doubtful that sufficient material suitable for engineered fill will be available. A Geotechnical Summary Report is attached in Appendix B.

The Klipfonteinspruit currently runs within the proposed footprint of the ADF and the diversion of this river and it’s Tributary is included in the scope for the engineering design.

8.5 Haul Distance

Due to the sheer size of the ADF footprint, cognisance of the haul distance to on-site stockpile areas should be noted and prospective tenderers should be informed of the haul distances.

8.6 Order of Works, Critical Path Items

8.6.1 Constructability

The order of works was discussed in detail in the *Stormwater Management Plan*, Report no. 355-511877 EPCM, October 2023. In summary, temporary and then permanent Attenuation Dams need to be in place before the pipelines for Phase 1 can be installed.

8.6.2 Pipe manufacture

The critical path for the stream diversion aspect of the project, however, would be the manufacturing lead time of the pipe sections. It is expected that the time to manufacture, the pipes could be in excess of six months, although delivery could commence from about three months. The supplier could possibly be engaged in discussions with Eskom about setting up a specially appointed manufacturing plant.

8.6.3 Rainfall season impact

The installation of the upslope attenuation dams should preferably be undertaken in the dry season. Likewise, the installation of the pipeline should take place when runoff and groundwater seepage are at their lowest.

The ADF footprint barrier layers should be installed in the dry season, in particular the preparation of the clayey soil base layer and the installation of the geosynthetic clay liner (GCL) and the geomembrane over it.

9. AVAILABLE SURVEY INFORMATION

A Lidar survey was carried out in August 2022, and contours of 1m interval were created. Only the contour file was available to the design team. For accurate earthworks quantities it was recommended that a physical survey of the valleys be carried out with definition lines for “top bank” and “bottom bank” and other feature lines, which are generally not picked up in a lidar type survey. Additional survey was duly received for the stream diversion valleys on 21 November 2023.

All survey data is based on the Hartebeesthoek 84 WGS Lo29 coordinate system which is currently the official geodetic datum for South Africa.

10. ELEMENTS OF THE PHASE 1 DEVELOPMENT DESIGN

The main elements in the Phase 1 development design are:

- Stormwater Management,
- Stream Diversions,
- Upslope Attenuation Dams,

- Ash Disposal Landfill Phase 1 (6 stages),
- Pollution Control Channels,
- Pollution Control Dams,
- Clean Water Channels,
- Clean Water Dams,
- Pump houses and pipe reticulations.

Also included in Phase 1 but excluded from this Design Report, are the internal roads network and the conveyor and access bridges crossing the Klipfontein Spruit from the Power Station to the ADF.

Mechanical infrastructure such as the pump stations and pipework at the Dams complex will be designed and installed by Eskom. Tie-in points have been formalised in the EPCM Interface Report for Package 1 / Package 2 and tables of significant tie-in points have been included in the sections following.

11. LATEST PROPOSED DESIGN OPTIMISATION

A few design optimisations have been proposed at the time of this report, which have been discussed in recent meetings with EPCM and Eskom representatives in October 2023.

- Fewer attenuation dams are proposed. It has been proposed to construct temporary attenuation dams higher up the valleys at the start of the project. This elevation allows for diversion of the streams, from the initial dams, into by-pass drains around the ADF Phase 1 footprint. Although the first construction works would now reach further up the valleys, and a small corner of the ash dump in Phase 1 would be sacrificed in the short term, fewer dams are required. This is both a cost saving and improves diversion of upslope runoff.
- All dams in the PCD and CWD complex are to be divided into two compartments, as requested by Eskom.
- PCDs 3 to 6 are planned to discharge into open channels rather than undergroudn pipes.
- A buttress wall is proposed for part of the ADF eastern flank, to be built up as the ash disposal progresses. The wall would be lined on the inside as per the basal barrier layers,

with special anchorage and stability measures placed with each progressive lift. Unlike the perimeter berm, the butress wall would rise about 25m at its highest point. The buttress wall would function as a contingency against the build-up of stormwater against the ash dump, in the event of stream diversion pipe blockage/ failure. During risk assessments it was previously proposed to build a dam on the eastern boundary with enough elevation on the dam wall to force the overspill into drains around the ADF. The required dam wall would have had a maximum height of 30m and a length of over 2.5 km. With the proposed optimisation, the ADF would now instead only require an inlet attenuation dam of 7m height, and should the dam or pipeline fail, the ADF would be protected. The buttress wall is not part of Phase 1 construction. However, the overall stormwater management does take note of the planned wall and dam.

- Previous designs called for a 3m to 3,2m pipe diameter based on maintenance equipment size, not flow. Considering advances in drone and mobile camera surveillance systems, and in remote control cleaning equipment, it is believed that large size inspection equipment should not be routinely necessary. The stream diversion pipelines are proposed to not exceed a diameter of 2,5m.

12. ASH DISPOSAL

12.1 Waste Classification

The waste stream consists of fine ash and coarser bottom ash. A waste classification report was drawn up by Jones and Wagener, Report number JW030/13/D121 Rev 2. The result of the analysis was that the waste is a **Type 3** waste (low hazard waste) and therefore requires a **Class C Barrier**, in accordance with DWA's Minimum Requirements for Handling, Classification and Disposal of Hazardous Waste and the then draft waste classification regulations, now referred to as DWS' National Norms and Standards for Disposal of Waste to Landfill NEM:WA R636.

12.2 Design Criteria

The design criteria are listed in Table 5 below, and form the basis for the design of the Kusile ADF.

Table 5 - Basis of Design

Design parameter description	Value of design parameter	Source of information
Ash Characteristics		
Ash grading		
4.75mm	100%	WSP Golder Lab Testing - 2022
2.00mm	100-99%	WSP Golder Lab Testing - 2022
0.425mm	98-96%	WSP Golder Lab Testing - 2022
0.075mm	87-78%	WSP Golder Lab Testing - 2022
0.002mm	5-6%	WSP Golder Lab Testing - 2022
Grading Modulus	0.15 – 0.27	WSP Golder Lab Testing - 2022
Plastic Index	-	WSP Golder Lab Testing - 2022
Ash dry density	800 kg/m^3	J&W Detailed concept design
Ash moisture content	20-33%	Kendal Operations manual
Ash Bulk Density	1193.156 kg/m^3	WSP Golder Lab Testing - 2022
Specific Gravity	2.202 – 2.254	WSP Golder Lab Testing - 2022
Ash angle of repose:		
Upper third	40 degrees	Kendal Operations manual
Middle third	33.7 degrees	Kendal Operations manual
Lower third	22.6 degrees	Kendal Operations manual
Average slope for geometric modelling	30.5 degrees	Determined from survey
Ash permeability	2.1x10^{-5} cm/s to 1.4x10^{-5} cm/s	Results from 10-year co-disposal testing and Civilab Results (2024)
Ash Production		
Annual tonnage/ unit	1 182 600 t/year	J&W detailed concept design
Number of units in total	6	J&W detailed concept design
Total tonnage/ year	7 095 600 t/year/6units	J&W detailed concept design
Phase 1 pre-deposition works construction date		TBD
Operating life of facility	63 years (at 100% production full time)	J&W detailed concept design
Annual airspace required per unit?	1 004 499 m3/year	Quantity estimation based on dry density

Design parameter description	Value of design parameter	Source of information
Annual airspace required at full production	6 026 997 m3/year	Quantity estimation based on dry density
Total airspace estimated for life of facility	377 757 251 m3	Based on available space
Geometric characteristics		
Overall maximum height	138m	Geometric modelling
Modelled operational side slopes	1V:2H	
Platform ramp slopes	1V:10H	
Modelled rehabilitated side slopes	1V:5H	
Maximum bottom stacker back stack	12m	Geometric modelling
Maximum top stacker back stack	12m	Geometric modelling
Conveyor Parameters		
Target system utilisation (Top:Bottom)	50:50 Split	TBC
Top Extendable Conveyor:		
Maximum length	2768m	TBC
Maximum allowable grade	1V:10H	TBC
Bottom Extendable Conveyor:		
Maximum length	2672m	TBC
Maximum allowable grade	1V:10H	TBC
Top Shiftable Conveyor		
Maximum length	1772m	Eskom GTE
Maximum grade	1V:20H	Operations manual
Shift distance	79.50m	Eskom GTE
Ashing reach	92.8m	TBC
Safe edge distance	15m	Stability analysis
Bottom Shiftable Conveyor		
Maximum length	2356m	Eskom GTE
Maximum allowable grade	1V:20H	Operations Manual
Shift distance	79.52m	Eskom GTE
Ashing reach	92.8m	TBC

Design parameter description	Value of design parameter	Source of information
Safe edge distance	15m	
Stacker machine parameters		
Link conveyor length	50m	Communication between ZJV and Eskom
Stacker boom length	35m	Communication between ZJV and Eskom
Minimum angle between link conveyor and shiftable conveyor	25°	Based on Majuba Stacker System
Slope Stability Analysis		
Operational Static (FoS)	1.2	
Final Static (FoS)	1.5	

12.3 Overall ADF Growth Plan.

The ADF growth plan provides detail on ash platform development on a time and shift by shift basis to illustrate the development of the ADF for the duration of the operational life of the facility. The growth plan takes into account constraints such as:

- Stacker system utilisation:
- Mechanical constraints of the conveyors and stacker systems;
- Initial pre-deposition works;
- Ramp up production of the power station;
- Ultimate ash production; and
- Decommissioning time.

The ash is conveyed from the power station to the ADF via two (2) transverse conveyor systems (TAC 1&2) in a south western direction and discharges onto two (2) new overland conveyor systems (OC 1&2) at existing transfer house 8. The new overland conveyor sections will be installed from existing Transfer House 9 position as an extension of the existing overland link conveyor system (OLC1&2). The overland conveyors discharge to the top and bottom extendable conveyors (EC 1&2) at Transfer Houses 11 and 12 respectively. The new overland conveyor sections to be installed are approximately 1 377m in length between existing Transfer House 9 and 12 at the conveyor head end, with Transfer House 11 located at chainage 1004m from Transfer House 9.

The bottom extendable conveyor (EC 2) receives ash from the overland conveyors at Transfer House 12 and conveys the ash up onto the bottom stacker platform in order to discharge onto the bottom shiftable conveyor (SC 2). The bottom extendable conveyor is extended on the top of the stacker platform as per the requirement of the bottom shiftable conveyor shift position.

Similarly for the top stacker extendable conveyor (EC 1), which receives ash from the overland conveyors at Transfer House 11, the ash is conveyed up onto the top stacker platform in order to discharge onto the top shiftable conveyor (SC 1). The top extendable conveyor is also extended on the top of the stacker platform as per the requirement of the top shiftable conveyor shift position. The bottom and top stacker shiftable conveyors both discharge onto a crawler mounted stacker machine utilizing a traveling tripper car and link conveyor.

For Phase 1, a total of 58,348,377m^3 of ash will be deposited on the Phase 1 footprint, with a maximum height of 105m. At 100% full production of all 6 units, this will serve Kusile Power Station for 10 years.

The Growth Plan model and time-stage tables are included in the Operating Plan In **Appendix E.**

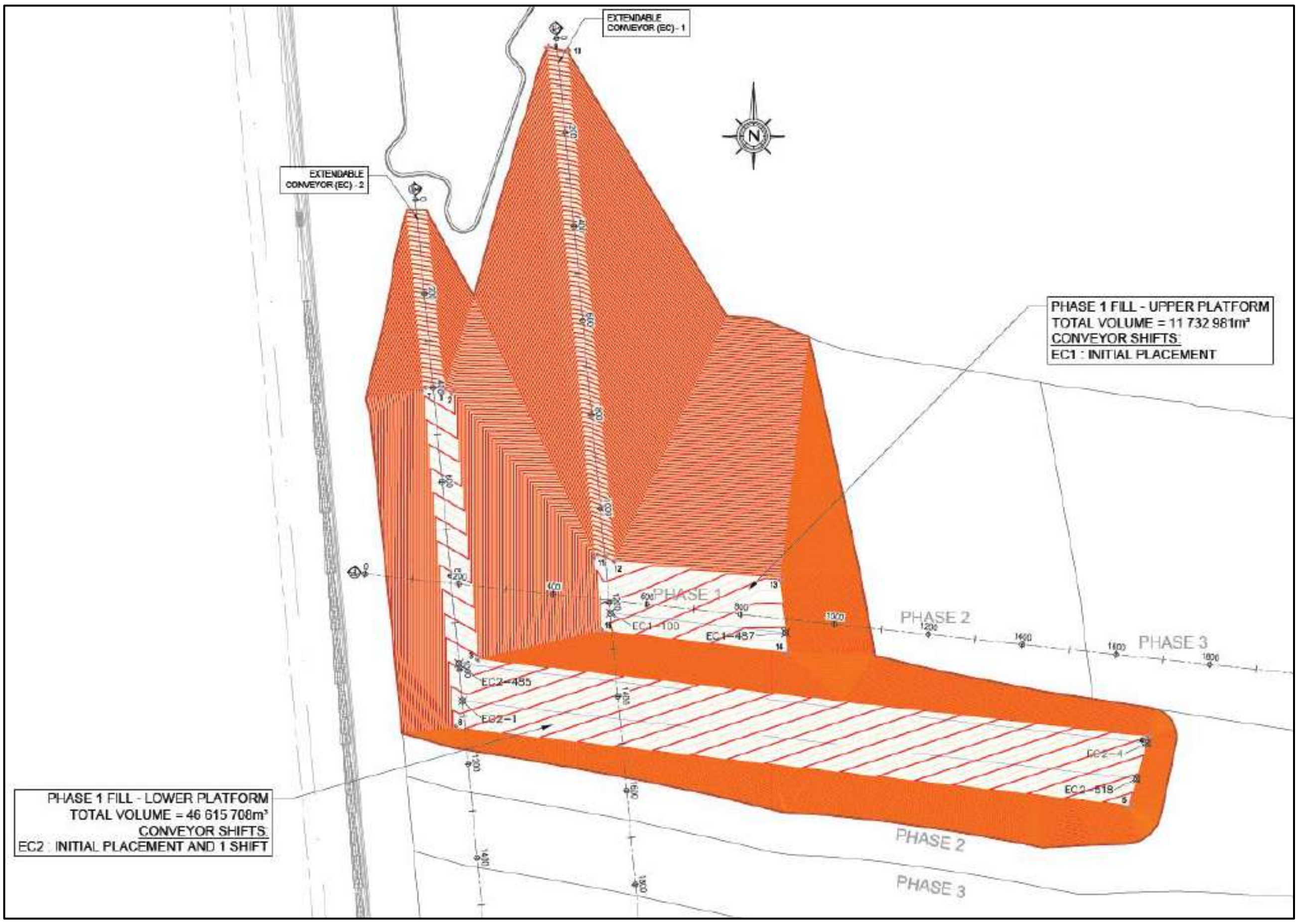


Figure 6: Ash Conveyor Positions

12.3.1 Pre-shift dump development

The pre-shift deposition works essentially consists of the construction and operational works that takes place before parallel shifting of the shiftable conveyor systems. The first phase of pre-deposition is for the first seven (7) years of ash deposition and consists of:

- ADF Pre-Deposition construction activities such as topsoil stripping, starter embankment construction, base and liner preparation, geomembrane placement and drainage layer installation;
- Conveyor corridor civil works and Stacker Erection Platform 2 (SEP 2) construction; and

- Conveyor and stacker assembly in order to convey ash to the ADF.

12.4 Pre-deposition Works

The pre-deposition infrastructures are the elements required to safely dispose of ash in a technically sound and environmentally friendly manner while ensuring a stable ash dump. The pre-deposition infrastructure will be implemented in a phased manner in stages of approximately four-year intervals. Pre-deposition works required for the ADF are discussed in the sub-sections below.

12.4.1 Topsoil stripping

The footprint of the ADF is covered with topsoil / mineral rich transported soils. There is a requirement from the Environmental Impact Assessment (EIA) to strip the top soil from the footprint and to stock pile it. An average thickness of 300mm of topsoil should be stripped and used as cover material to rehabilitate the final landform crest and slopes (1V:3H and flatter).

The topsoil from Phase 1 shall be stripped and stockpiled on the area demarcated for Phase 4. This should ensure that the excess topsoil stockpile does not impede the continuous development of the ADF pre-deposition works while still being in close proximety to reduce haulage distances during rehabilitation. As from Phase 2, the topsoil stripped from its footprint shall be used for rehabilitation of the previous phases or moved to the stockpile. The Post-Deposition work is delayed by one phase and will only start once Pre-Deposition Phase 2 commences. Once the ADF has developed over the Phase 1 footprint, rehabilitation of the ADF should commence.

It should be noted that the material balance may change depending on the as-built construction plan and must be updated accordingly.

12.4.2 The footprint excavation and footprint preparation

The footprint excavation and preparation, consists of shaping the footprint according to the lines, levels and grades as indicated on the detailed design drawings. The footprint excavation will mainly consist of removing material unsuitable for the footprint in terms of strength and permeability. The fill material for the majority of the starter and intermediate walls will also be sourced from the footprint as detailed in the material balance. If additional material is required a borrow pit should be sourced for suitable material or approved (by the site engineer) selected on-site material shall be allowed, provided it qualifies as a G5 construction material. The in-situ materials are highly variable and it is imperative that a qualified geotechnical engineer and soil laboratory are on site during earthworks activities.

12.4.3 Engineered Fill

The north-eastern extent of the Phase 1 footprint slopes fairly steeply down the valley. An area at the toe of the facility is to be raised with engineered fill, with a surface sloping westwards, against the natural gradient. The engineered fill will provide stability to the north-eastern ash slopes, and raise the perimeter channel inverts with enough height to facilitate gravity flow to the PCDs and CWDs.

The engineered fill embankment is to be formed with selected and imported fill conforming to at least G5 material sepcification, and compacted in layers not exceeding 150mm to a minimum of 95% Mod AASHTO. The footprint of the engineered fill is to be stripped of topsoil and box cut in steps of 450mm to provide a keying in of the imported layers.

12.4.4 Perimeter and Intermediate Embankments

The starter and intermediate embankments are constructed to define the actual extent of the ADF aswell as to anchor-in the geomembrane and protection geotextile of the liner system. A nominal 1.5m high starter embankment will be provided. The intermediate embankments will make up the inter-stage walls on the inside footprint of the facility. The embankments act as an ash containment boundary line and as a tie-in perimeter for the liner works. The starter embankment (outer and intermediate) properties can be summarised as follows:

- Minimum Height of 1.5m
- Crest Width of 3m
- Inside Slopes to be 1 (vertical): 3 (horizontal)
- Outer Slope to be 1 (vertical): 3 (horizontal)
- Compaction of berms to be 150mm layers to 95% Mod AAHTO

The leachate collection pipes will need to pass through the embankments in order to allow for gravitational, controlled flow. As a result, a geomembrane pipe boot will need to be placed over the collection pipes passing through the embankments and welded to the geomembrane liner covering the ADF footprint.

The leachate collection pipes exitting the embankments to the exterior of the ADF footprint will tie in with the dirty water channels.

The summary of the Materials Balance for Phase 1 of the Kusile can be found in **Appendix C**.

12.4.5 Anchor Trench

The anchor trench for the barrier layers is designed so that the geomembrane would rather pull-out of the anchor trench than forming a wide width tear at the crest of the embankment. Therefore, the maximum tensile strength with an appropriate safety factor is used to size the anchor trenches. The anchor trench is 600mm wide and 700mm deep, with a run-out length on the crest of 1m.

The dimensions of the crest anchor trenches and an anchorage ratio has been calculated. The anchorage ratio is the ratio between the allowable tensile force and the resistance force provided by the anchor trench. The anchorage ratio should be as close to one as possible to ensure the geomembrane does not fail by means of a wide width tear.

The trench is to be backfilled with selected fill material of G5 quality, compacted in 150mm layers to 95% Mod AASHTO. The crest corners should be rounded to ensure that no stress concentrations form at these points that may lead to stress cracking as well as to ease the hot wedge welding of the seams and to minimise burnouts.

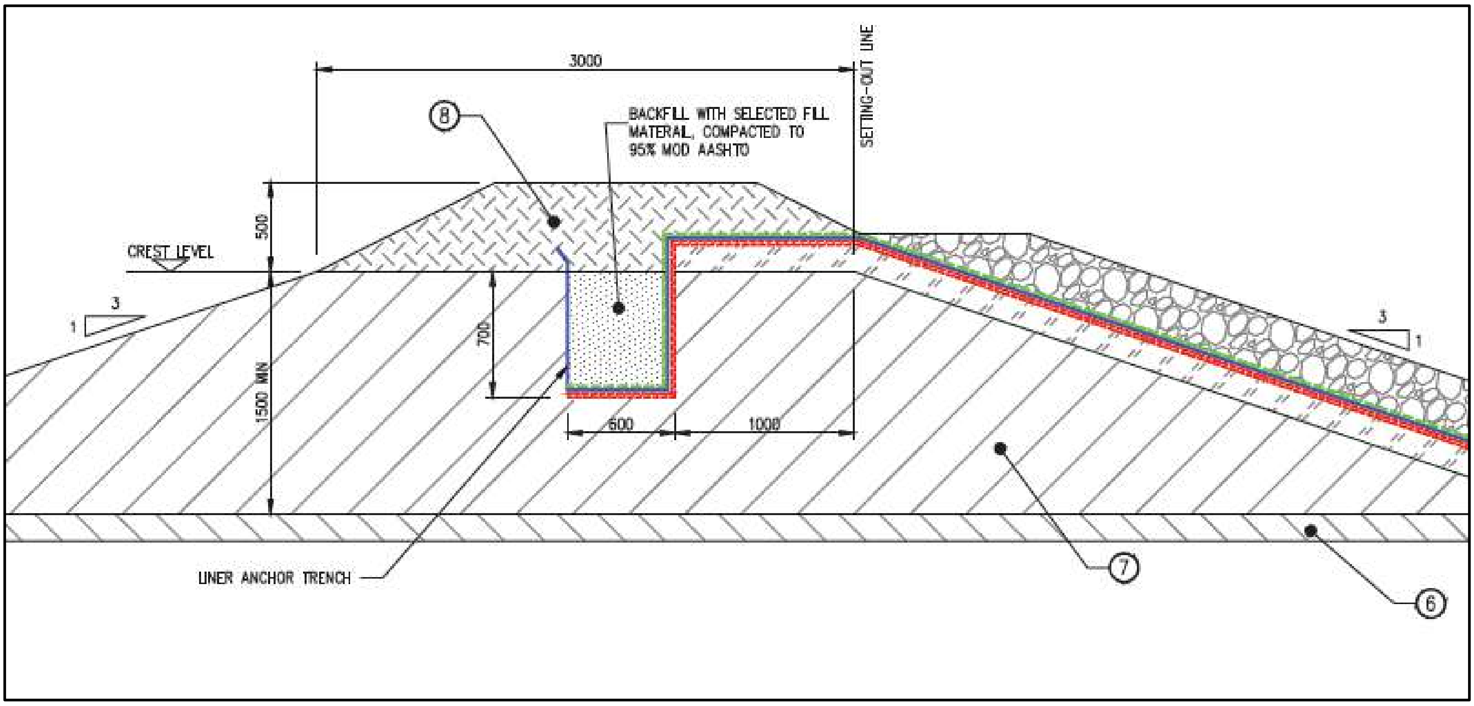


Figure 7: Edge Berm Anchor Trench

12.4.6 Subsoil Drainage System

The subsoil drainage system, which serves to prevent shallow groundwater from saturating the layer works and for leakage detection purposes, is provided underneath the composite liner. The subsoil drainage system consists of a network of pipes with the same layout as for the leachate collection pipes, except at lower elevations, and at roughly double the spacing. The subsoil drains consist of a 400 mm wide by 500 mm deep trench with a separation geotextile, backfill layers, and HDPE pipes . The subsoil drain outlet pipes for each phase shall report to a separate manhole outside the basin footprint to enable monitoring of each section of the ADF where possible. Due to the valley within the Phase 1 footprint, some sub-soil drains are directed to the stream diversion pipeline where they will penetrate and drain into the pipeline junction boxes.

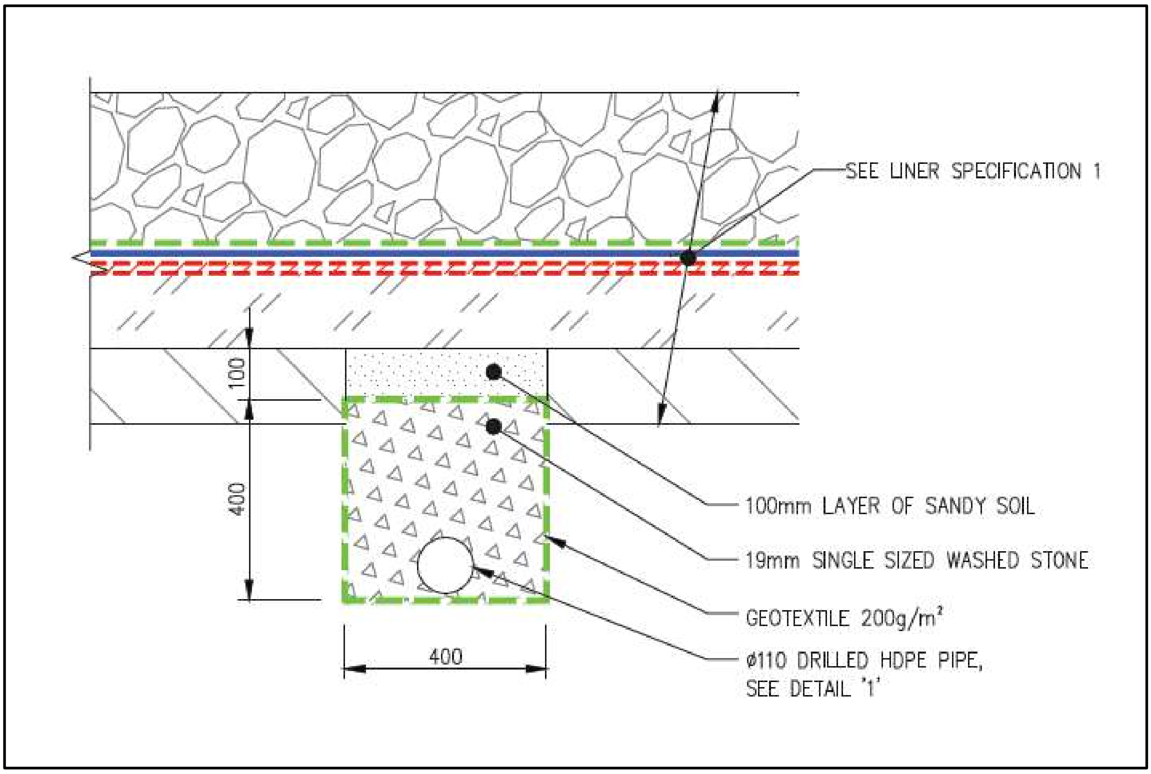


Figure 8: Sub-soil drain in ADF

12.4.7 Leachate Drainage System

The leachate drainage system drains water that percolates through the ash dump to the base of the ADF. The leachate system lies above the barrier layers, within a drainage layer of coarse material. The system is made up of HDPE pipes drilled with 15mm holes over the top of the pipe, laid within a finger drain of stone aggregate. The perforations allow for the ingress of water which then drains through the pipes by gravity, to the lowest point in the network.

At the outlet point, a solid wall pipe penetrates through the liner to discharge into the dirty water channel outside the ash dump perimeter. At the penetration, a HDPE geomembrane pre-fabricated boot seal is welded to the barrier geomembrane to create a sealed outlet. The channels convey the leachate to the PCDs.The detail of the leachate finger drains is shown below.

PIPE SPECIFICATION:

1. LEACHATE PIPES TO BE ø160 OR ø200 HDPE, PN100 PE16, DRILLED AS PER DETAIL WHERE PERFORATED PIPES ARE REQUIRED.
2. SUBSOIL / LEAKAGE DETECTION PIPES TO BE ø110 HDPE, PN100 PE16, DRILLED AS PER DETAIL WHERE PERFORATED PIPES ARE REQUIRED.

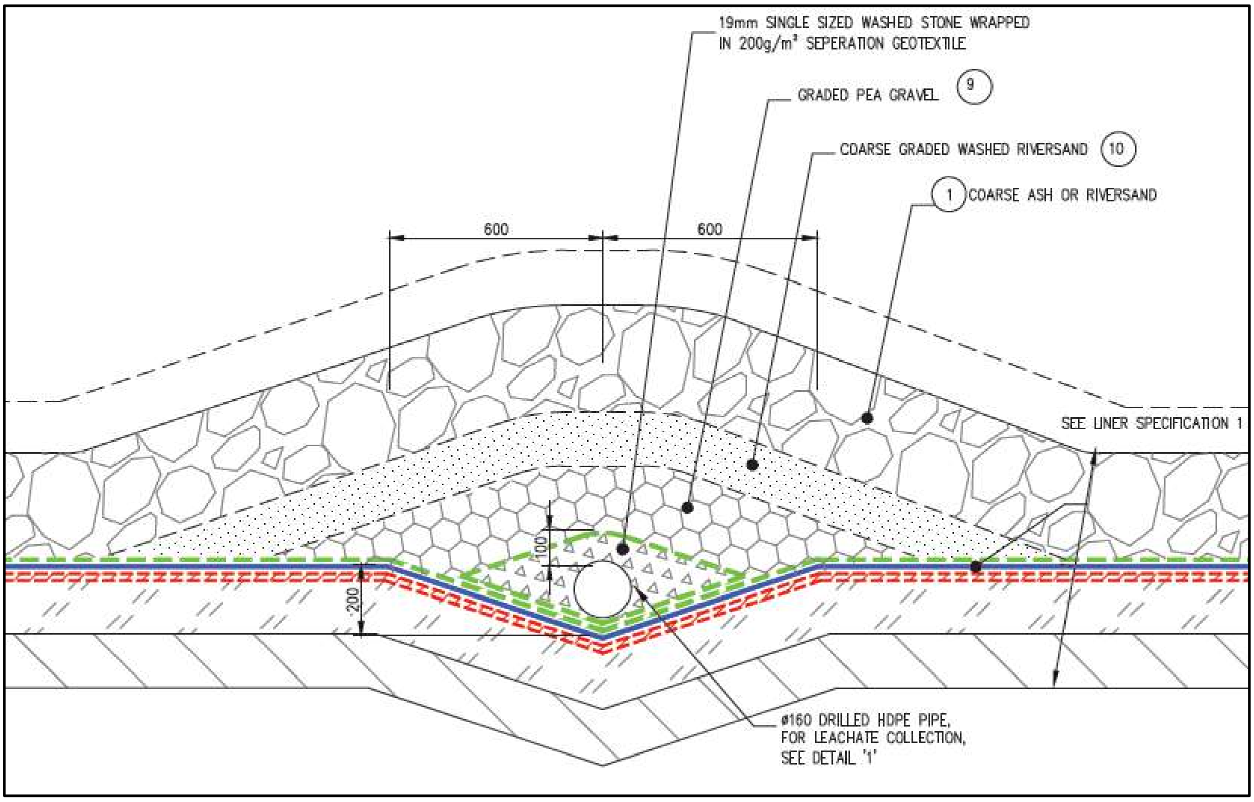


Figure 9: Typical Leachate Herringbone Drain, extracted from Drawing 366-511892

12.4.8 Barrier System

The ash of Kusile is classified as a Type 3 waste in terms of Regulation 636 of NEM:WA (2008) which requires a Class C Landfill barrier.

The National Norms and Standards for Disposal of Waste to Landfill NEM:WA R636 prescribes the following generic barrier design (from top to bottom):-

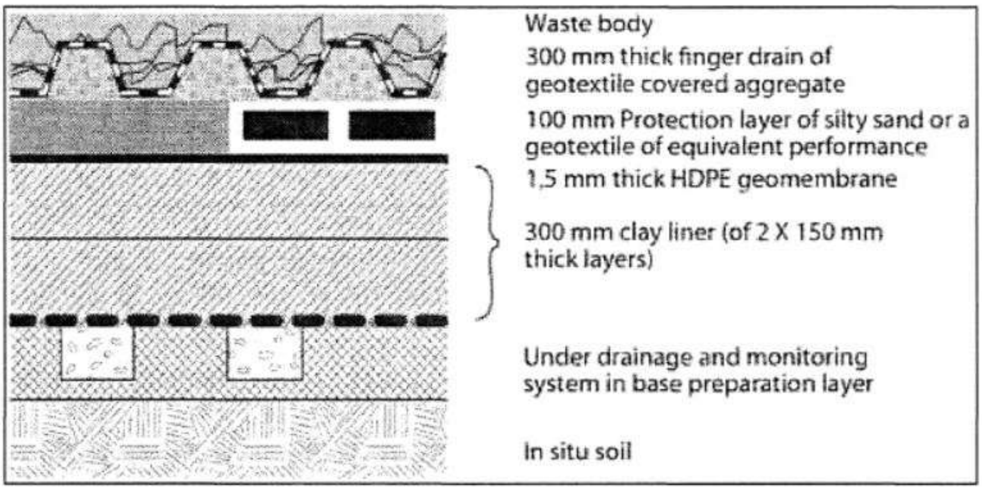


Figure 10: Norms and Standards Class C Barrier

Based on the above, the following site-specific equivalent barrier is proposed, listed from top to bottom:

- Waste body
- 300mm thick finger drains of geotextile covered aggregate within coarse ash layer
- 1000g/m^2 Protection geotextile
- 2mm thick mono-textured HDPE geomembrane
- 3,7 kg/m^2 Geosynthetic Clay Layer (GCL)
- 1 x 150mm thick clayey soil levelling layer / not required should the in-situ material be suitable to have a GCL layer placed upon it.
- Rip and Recompact in-situ material as base preparation or imported compacted fill

LINER SPECIFICATION	
LAYER	DESCRIPTION
1	300mm THICK LAYER OF COARSE MATERIAL AS DRAINAGE AND PROTECTION LAYER
2	1000g/m² PROTECTION GEOTEXTILE TO GRI– GT12 SPECIFICATION
3	2 mm MONO–TEXTURED GEOMEMBRANE TO GRI–GM3 SPECIFICATION, WITH TEXTURE FACING DOWNWARDS
4	3,7kg/m² GCL TO GRI–GCL 3 SPECIFICATION
5	150mm LAYER OF CLAYEY SOIL, SELECTED ON SITE UNDER ENGINEER'S DIRECTION, COMPACTED TO 98% MOD PROCTOR AT OMC TO +2%
6	BASE PREPARATION LAYER: RIP AND RE–COMPACT TO 95% MOD AASHTO
7	SELECTED FILL MATERIAL COMPACTED IN LAYERS NOT EXCEEDING 150mm THICK TO 95% MOD AASHTO
8	10% STABILISED SANDY SOIL COPACTED TO 95% MOD AASHTO
9	GRADED PEA GRAVEL
10	COARSE GRADED WASHED RIVERSAND
11	150mm SACRIFICIAL LAYER OF SOIL TYPE G7 OVER DRAINAGE LAYER OF ASH ONLY (NOT OVER RIVERSAND), AS DIRECTED, WHERE AREA IS TO REMAIN OPEN FOR AN EXTENDED PERIOD.

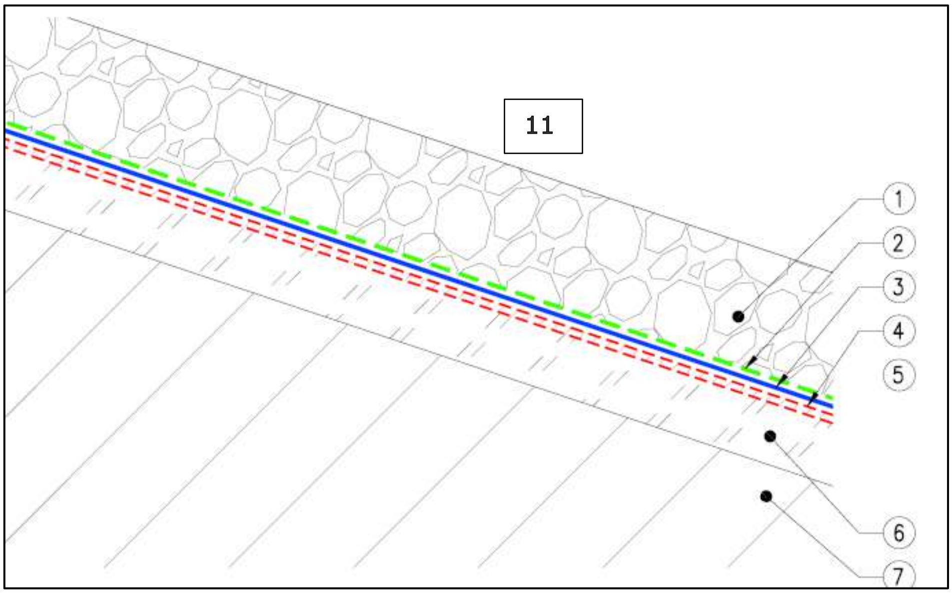
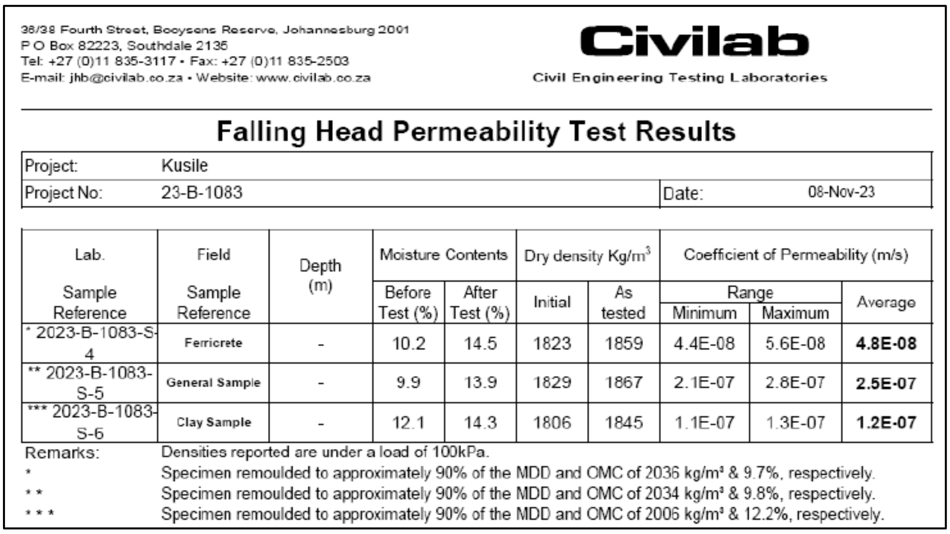


Figure 11: ADF Liner Design

It is to be noted that the specifying of a Geosynthetic Clay Layer (GCL) in place of the 2 x 150mm compacted clay layers as set out in the Norms and Standards standard liner composition has been done following several suitability tests on the in-situ clay material available on site.

Due to the clay samples taken from stockpiles on site not meeting the minimum requirements of maintaining a hydraulic conductivity of less than or equal to 1 x 10-7cm/s as set out in the Norms and Standards (see results below), a geosynthetic clay liner has been proposed to replace the compacted clay liner as detailed.



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Civilab
Civil Engineering Testing Laboratories

Falling Head Permeability Test Results

Project:	Kusile		
Project No:	23-B-1083	Date:	08-Nov-23

Lab. Sample Reference	Field Sample Reference	Depth (m)	Moisture Contents		Dry density Kg/m^3		Coefficient of Permeability (m/s)		
			Before Test (%)	After Test (%)	Initial	As tested	Range		Average
							Minimum	Maximum	
* 2023-B-1083-S-4	Ferricrete	-	10.2	14.5	1823	1859	4.4E-08	5.6E-08	**4.8E-08**
** 2023-B-1083-S-5	General Sample	-	9.9	13.9	1829	1867	2.1E-07	2.8E-07	**2.5E-07**
*** 2023-B-1083-S-6	Clay Sample	-	12.1	14.3	1806	1845	1.1E-07	1.3E-07	**1.2E-07**

Remarks: Densities reported are under a load of 100kPa.
* Specimen remoulded to approximately 90% of the MDD and OMC of 2036 kg/m^3 & 9.7%, respectively.
* * Specimen remoulded to approximately 90% of the MDD and OMC of 2034 kg/m^3 & 9.8%, respectively.
* * * Specimen remoulded to approximately 90% of the MDD and OMC of 2006 kg/m^3 & 12.2%, respectively.

A GCL generally contains a woven carrier layer, a powder bentonite layer and a geotextile cover layer. The equivalence of a GCL compared to a 60 cm thick CCL can be gauged based on the CCL's typical minimum hydraulic conductivity of 1x10-7 cm/s whereas, when saturated, the hydraulic conductivity of bentonite typically drops to less than 1 x 10-9 cm/s.

With stringent MQC's being undertaken during manufacturing, GCL's allow for a consistent quality throughout the lining layer without the need for uniform selection, compaction and testing of a CCL. Furthermore, the self-healing capability even after dry-wet cycles as well as the easy and quick installation are benefits for the quality of the construction as well as for the construction process.

12.5 Slope Stability Analysis

Global Stability

The stability assessment of two sections as indicated on **Figure 12** were carried out. The layout indicates a valley to the north of the ADF which falls outside of the ADF footprint and is not considered. The valley indicated on the drawing runs underneath the ADF and is the valley referred to in the sections below. The top stacker platform extension is towards the east from its initial position on the western side of the site.

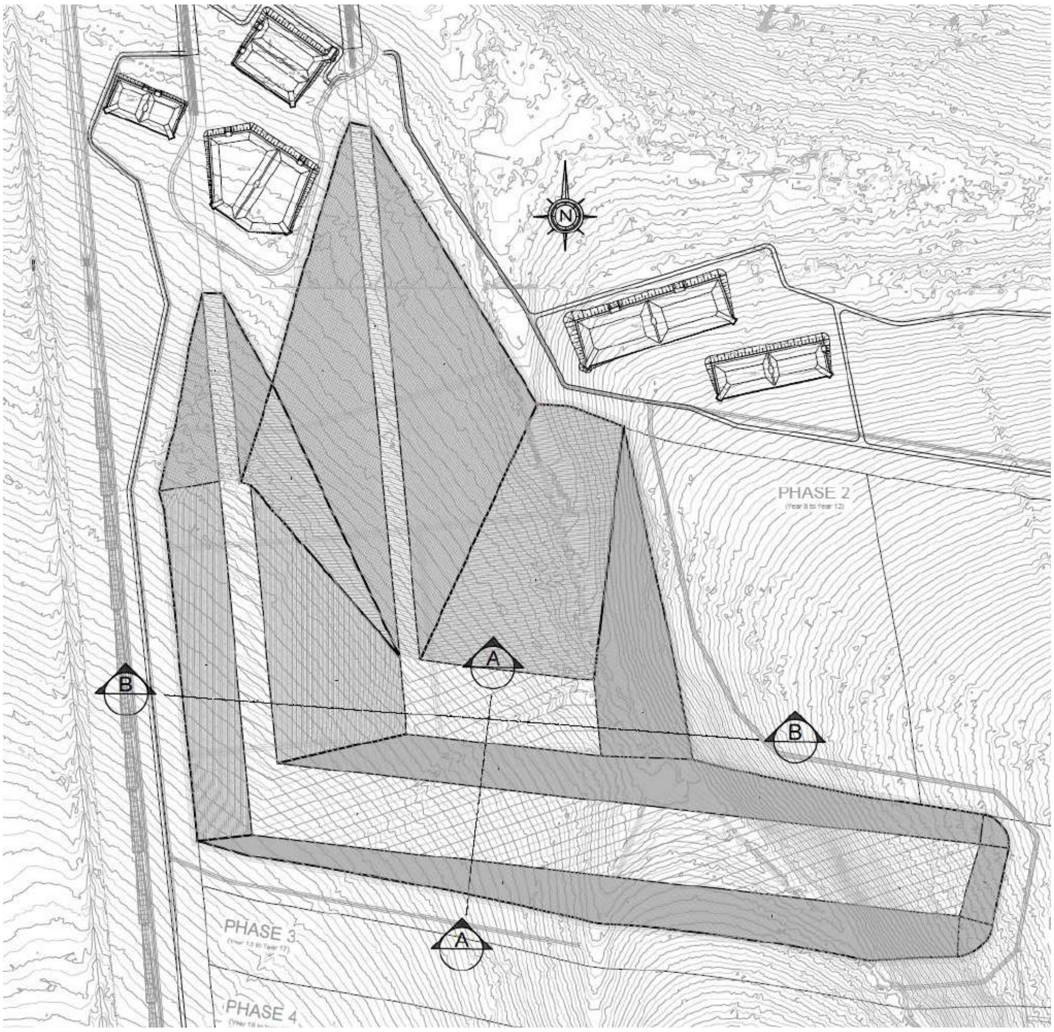


Figure 12 - Slope Stability Sections

The sections analysed were deemed to be representative of the critical scenarios during operation and final shaping of the ADF with:

- Section AA: Typical steep working face at a slope of 1:2 with stacking surcharge on edge
- Section BB: Longest steep slope from top stack platform to base at valley area.

Other operational sections have been analysed, but only the above, critical sections have been reported on as these are representative of the overall stability.

Available information

The following information was reviewed as part of the stability assessment:

- Final geotechnical investigation report prepared Envitech and EPCM which included:
  - Laboratory testing results on in-situ material;
  - Logs of in-situ material.
- Proposed ADF geometry (as per basic engineering level design); and
- Reference paper on the properties of dry dumped fly ash (Fourie et. al, 1997).

Assumptions

This section summarises the critical assumptions made as part of the stability assessment:

Water Levels

- Current and post-closure static water tables were modelled at 300mm above the lined basal level throughout the ADF body.

Material Properties and Founding Conditions

- The material properties defined in terms of a Mohr Coulomb material model are provided in the table below. The material properties are based on laboratory testing carried out by EPCM (2022) and ZJV (Report No.: 1651909-307604-1, dated 16 January 2017) and values taken from literature as discussed below. These values are reflected in **Table 6**.

Table 6 - Material Properties used in the Stability Assessment

Material	Unit Weight γ (kN/m^3)	Cohesion c' (kPa)	Friction Angle ϕ' (degrees)
Ash	8	0	35
HDPE Liner	10	0	15.8
Clay	16.5	0	21.3
Soft Rock	22	500	0
Bedrock	22	1200	0

- The Mohr-Coulomb strength parameters of the ash have been adjusted (with reference to Fourie et. al (1997)) such that a slip surface with a factor of safety of 1.0 is apparent near to the surface of the top two-thirds of the advancing face, which conforms to the method by which the ash is placed (i.e. at or near angle of repose).
- The Mohr-Coulomb strength parameters for the liner material were assessed based on previous testing carried out on identical lining compositions with the friction angle displayed representing the critical friction angle between the HDPE and the protection geotextile above it.
- The founding conditions, based on test pit profiles, were taken as a 2 m thick clayey layer succeeded by a 3 m soft rock layer. A bedrock layer is present below the soft rock layer. It should be noted that these delineations are considered to be conservative, as stripping during construction will reduce the clayey layer depth.
- Ash dumped within the ash dump for the power station’s lifecycle will be similar to that of the characteristics of the ash samples tested.
- The angles at which the ash will naturally settle after dry placement were taken as:
    - 22.6° for the bottom third of the face
    - 33.7° for the middle third of the face
    - 40° for the top third of the face

Analysis

Some notes on the analysis performed:

- The development of water-filled surface cracks was omitted.
- Surcharge loading from plant and equipment other than the conveyors and stackers were ignored, a distributed load of 85kPa (over 20m) is used at the location of the stacker (loading is based on Majuba power station stacker load).
- A half-sine ratio between the horizontal and vertical inter-slice forces was specified for the Morgenstern-Price method of slices.
- A safe edge distance of 15m was used for the ash stacked and a distributed load of 85kN was used over 20m for the stacker weight on the ADF body.

Methodology

Stability analyses were carried out using the following software:

GEO5 2023, a slope stability program which computes the stability of slopes and embankments with circular or polygonal slip surfaces. In terms of polygonal or non-circular slip surfaces, the programme can perform the Sarma, Spencer, Janbu or Morgenstern-Price, Shahunyants and ITF methods. We have used the Sarma and Morgenstern-Price methods.

The Sarma (1979) method falls within a category of general sliced methods of limit states. It is based on fulfilling the force and moment equilibrium conditions on individual blocks. The blocks are created by dividing the region above the potential slip surface by planes, which in general have a different inclination.

Morgenstern-Price (1967) is a general method of slices developed on the basis of limit equilibrium. It requires satisfying equilibrium of forces and moments acting on individual blocks. The blocks are created by dividing the soil above the slip surface by vertical dividing planes.

The stability is expressed in terms of a Factor of Safety (FoS) against failure, which can be defined as follows for static loading conditions (taken from the SAICE Code of Practice on Lateral Support in Surface Excavations, 1989):

- FoS < 1,0 - The stability is inadequate, and failure is imminent.
- 1,0 < FoS < 1,3 - Stability is marginal.
- FoS > 1,3 - The short term / operational stability is acceptable.
- FoS > 1,5 - The long term / closure stability is acceptable.

Stability Analyses Results

The results of the stability analyses are given in **Table 7** below.

Table 7 - Stability Analysis Results

Analysis number	**Operational**	**Global**	**Figure Reference***
1.	Section AA Initial – Block failure upper working failure	1.43	Figure 1
2.	Section AA Initial – Composite failure surface	1.45	Figure 2
3.	Section BB East – Working face block failure	1.29	Figure 3
	Post-closure		
4.	Section BB Closure – Lower Platform	2.50	Figure 4
5.	Section BB Closure – Upper Platform	3.41	Figure 5

* The figures presenting the failure surfaces are attached in Appendix C.

Discussion

The FoS of slip surfaces intersecting the ash stacking machinery are within the upper limit of the marginal stability category (close to 1.5) or satisfy operational stability (Figures 1 to 3). The stability is, therefore, considered to be marginal and will require continuous monitoring. The dump profile has been shown to work in practice at Kendal Power Station and it is expected that the FoS of the dump should increase as time progresses, due to the cementation phenomenon engendered by the free lime content in the ash. Additional to this, some shear strength gain will be realised through matric suction caused by the partially saturated conditions experienced in the ash.

The FoS post-closure is well above 1.5 (Figures 4 & 5) and the rehabilitated profile is appropriate for closure.

For the conditions analysed in this analysis, the ADF is considered to be safe, although only marginally in certain scenarios (with stacker at safe edge distance and slip surface intersecting distributed load). It is essential that safe edge distances are adhered to by the conveyor stackers.

Should any additional information become available on any pertinent aspects of the ADF (machinery changes, additional data on the ash properties, etc.) the stability analysis should be repeated with consideration of these aspects.

Veneer Stability Analysis

A veneer stability analysis was carried out on the cell slope liner system in accordance with the methodology of Soong and Koerner (2005). This methodology is described in “Designing with Geosynthetics, 6th Edition, by Robert M. Koerner.

The following input parameters were used in the calculations:
Thickness of cover soil : 0,30m
Slope angle (1v:3h) : 18,4º
Length of slope : 6,3m
Unit weight of soil` layer : 18,6 kN/m3
Friction angle of cover soil : 36°
Cohesion of cover soil : 0 kN/m2
Critical interface friction angle
(geotextile/geomembrane - TRI) : 8,7°
Critical interface adhesion
(geotextile/geomembrane - TRI) : 1kN/m2

The following Factors of Safety were obtained from the analyses:

6,3m slope : 1,31

The FoS of 1,31 appears to be satisfactory for a temporary slope against the separation berms but poses no structural stability issues should a slippage occur.

12.6 Compatibility of Liner Materials with the Waste Stream

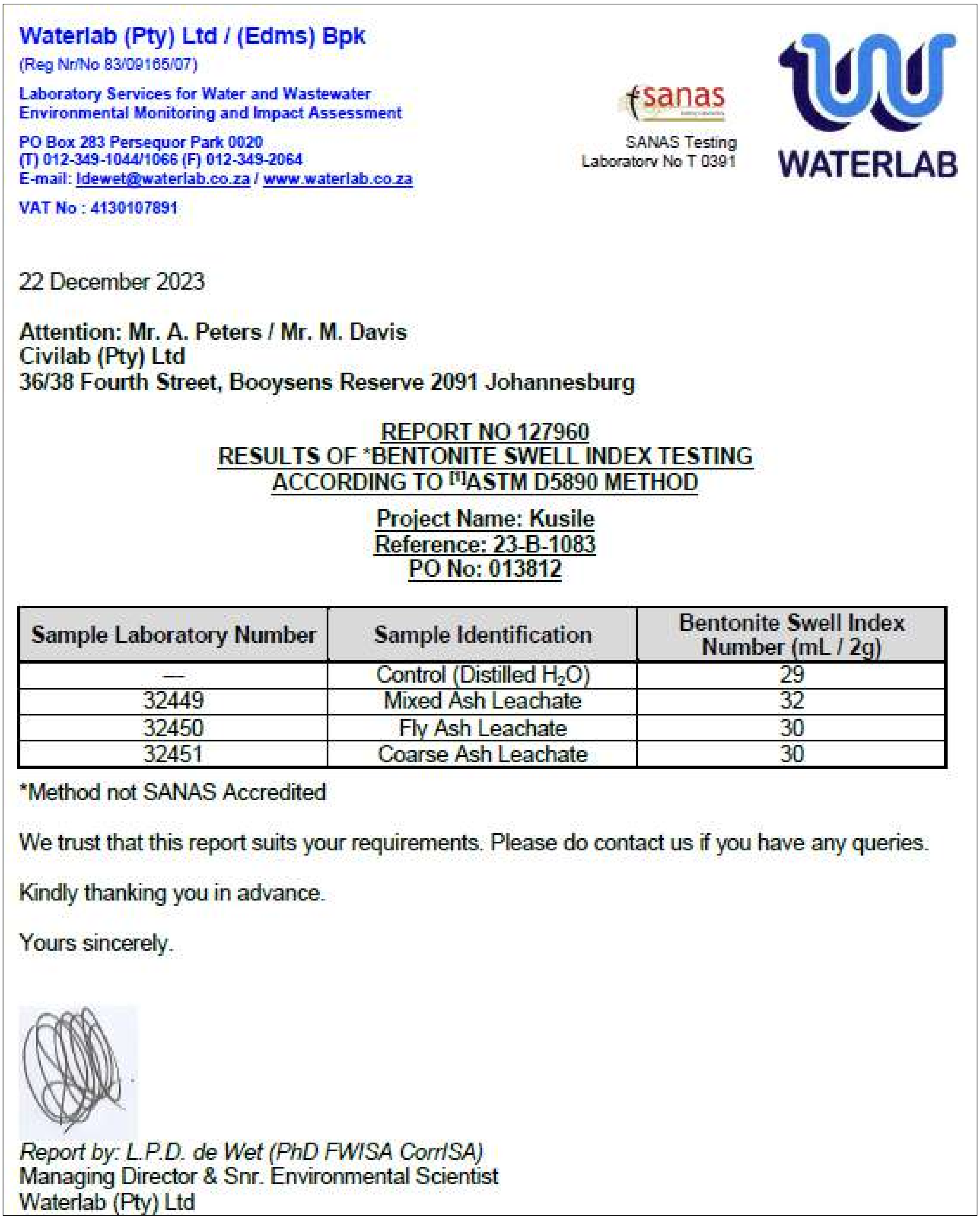
In the experience of EPCM Engineers, HDPE geomembranes manufactured in accordance with GRI GM13 specifications are generally accepted to be compatible with coal ash products.

Geotextiles, both woven and non-woven have been found to be compatible with ash leachate historically, provided that the geotextiles are not left exposed to ultraviolet light. The geotextiles installed must comply with GRI-GT12 and GRI GT13 as applicable with additional specifications as detailed in the report by GMJ Consulting which can be found in **Appendix C**.

Testing has been undertaken with site-specific ash (Kusile Power Station Ash) and bentonite powder taken from a commonly used, locally produced GCL. These tests would need to be re-done once a contractor is appointed for the construction, and their lining supplier is made known, to ensure the compatibility of the proposed materials.

The recent test results, shown below, indicate that the site specific ash's permeant solution (leachate) did not restrict the bentonite powder swell. This indicates that a GCL is suitable for use in the barrier system.

Additionally, similar swell testing was undertaken with site specific soil permeant and bentonite powder. The results indicate that the site specific soils do not restrict the bentonite powder swell and a GCL is fully compatible for the lining of this 60yr ADF Facility.



Waterlab (Pty) Ltd / (Edms) Bpk
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SANAS Testing Laboratory No T 0391

WATERLAB

22 December 2023

Attention: Mr. A. Peters / Mr. M. Davis
Civilab (Pty) Ltd
36/38 Fourth Street, Booysens Reserve 2091 Johannesburg

REPORT NO 127960
RESULTS OF *BENTONITE SWELL INDEX TESTING ACCORDING TO [1]ASTM D5890 METHOD

Project Name: Kusile
Reference: 23-B-1083
PO No: 013812

Sample Laboratory Number	Sample Identification	Bentonite Swell Index Number (mL / 2g)
—	Control (Distilled H_2O)	29
32449	Mixed Ash Leachate	32
32450	Fly Ash Leachate	30
32451	Coarse Ash Leachate	30

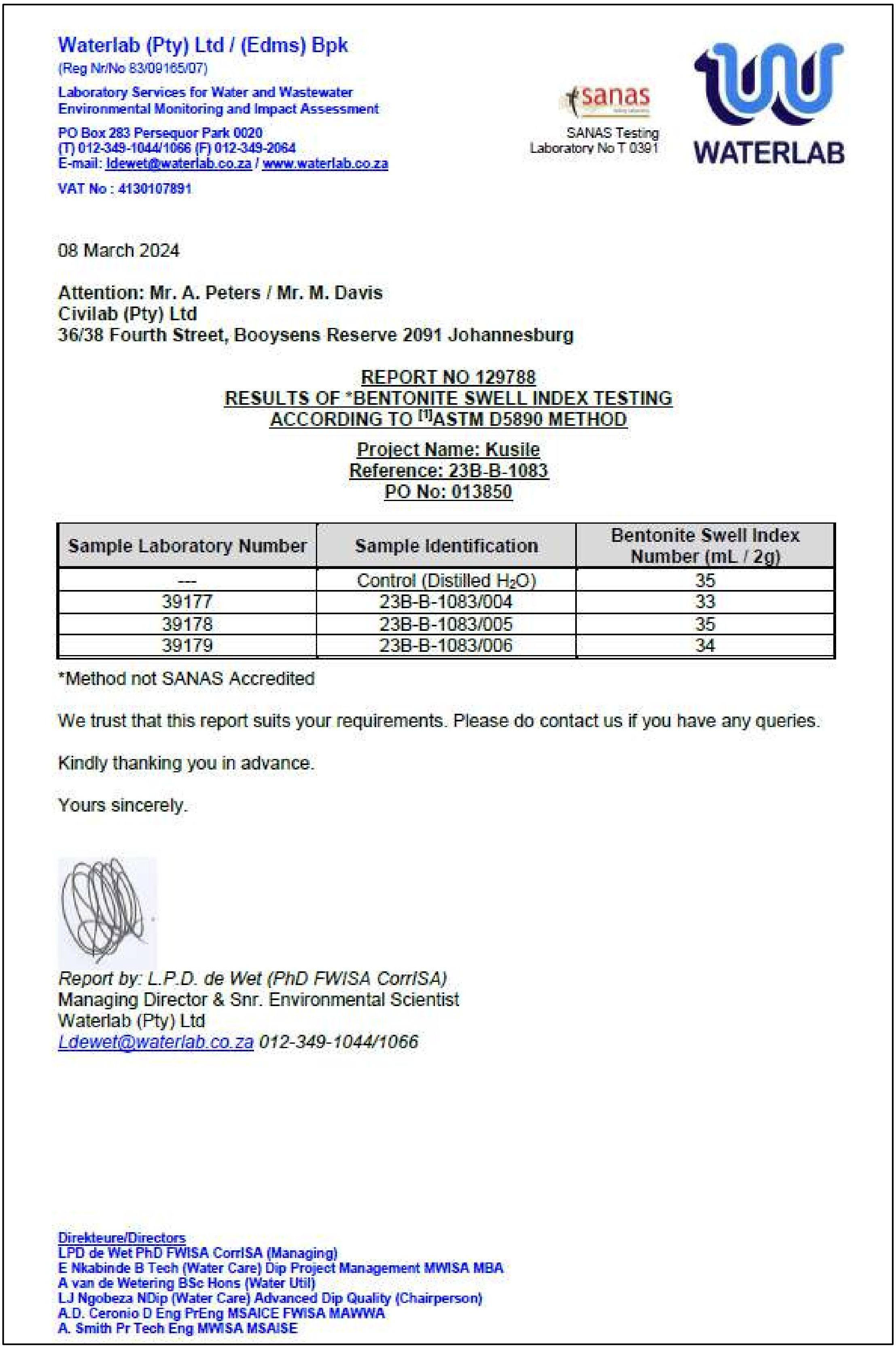
*Method not SANAS Accredited

We trust that this report suits your requirements. Please do contact us if you have any queries.

Kindly thanking you in advance.

Yours sincerely.

Report by: L.P.D. de Wet (PhD FWISA CorrISA)
Managing Director & Snr. Environmental Scientist
Waterlab (Pty) Ltd



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VAT No : 4130107891

SANAS Testing
Laboratory No T 0391

WATERLAB

08 March 2024

Attention: Mr. A. Peters / Mr. M. Davis
Civilab (Pty) Ltd
36/38 Fourth Street, Booysens Reserve 2091 Johannesburg

REPORT NO 129788
RESULTS OF *BENTONITE SWELL INDEX TESTING ACCORDING TO [1]ASTM D5890 METHOD

Project Name: Kusile
Reference: 23B-B-1083
PO No: 013850

Sample Laboratory Number	Sample Identification	Bentonite Swell Index Number (mL / 2g)
---	Control (Distilled H_2O)	35
39177	23B-B-1083/004	33
39178	23B-B-1083/005	35
39179	23B-B-1083/006	34

*Method not SANAS Accredited

We trust that this report suits your requirements. Please do contact us if you have any queries.

Kindly thanking you in advance.

Yours sincerely.

Report by: L.P.D. de Wet (PhD FWISA CorrISA)
Managing Director & Snr. Environmental Scientist
Waterlab (Pty) Ltd
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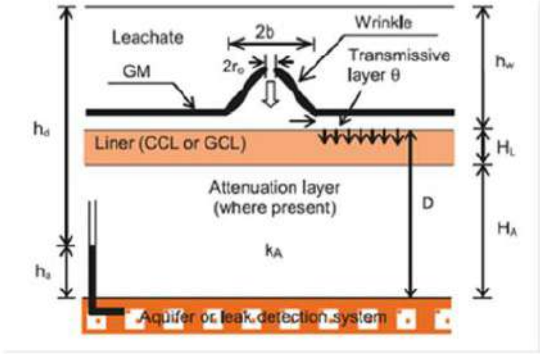
12.7 Total Expected Seepage through the Landfill Barrier System

The leakage through liners is usually interpreted in terms of the actual advective flow through the liner. The combination of a geomembrane and GCL substantially reduces leakage relative to a geomembrane alone and in addition geomembrane/GCL composite liners can be constructed to give lower leakage than a geomembrane/CCL composite (Rowe 2005).

Various methodologies may be used to calculate leakage rates i.e. Giroud and Bonaparte (1989), Rowe and Booker (1998), Rowe (2005) and Rowe (2012) however, all methodologies have various variables, assumptions and specific conditions (site and laboratory). Historic studies have also shown a significant discrepancy between theoretical and measured leakage rates. The major variables to consider are the size of the hole/s, the hydraulic conductivity of the GCL, the head difference across the liner, the transmissivity of the interface between the geomembrane and GCL (dependent on good or poor contact) and CQA. Using Rowe (1998), Rowe (2005) and Rowe (2012), the following equation was used to estimate the potential leakage rate for the ADF.

$$[6] \qquad Q = 2L[kb + (kD\theta)^{0.5}]h_d/D \qquad \text{(Rowe, 2012)}$$



Schematic showing leakage through a wrinkle of length *L* and width *2b* with a hole of radius r_o (adapted from Rowe 1998)

The estimated leakage rate based on the landfill liner system, with stringent CQA and assuming 5 No. wrinkles with holes per hectare equals to approximately **535 lphd**. This leakage rate is in line with the norms as indicated and documented theoretically for a GCL.

12.8 Service Life of a Landfill Liner System

To ensure a long service life of a HDPE geomembrane exposed to leachate in a landfill it is necessary to limit the tensile strains/ stresses in the geomembrane to an acceptable low level (Rowe, R.K. and Yu, Y., 2018). In addition, the service life of a containment barrier system is highly dependent on the exposure temperatures on the liner system and more specifically on the HDPE geomembrane. The service life of the HDPE geomembrane is normally evaluated using a three-stage degradation model which consists of Stage A (depletion of antioxidants through volatilization, diffusion or oxidation), Stage B (induction or start of polymer degradation) and Stage C (polymer degradation and decrease in key physical properties). The boundaries between these stages are not so distinct in practice, and the end of Stages B and C may vary depending on the parameters being considered, e.g. tensile break strength, tensile break strain and stress crack resistance.

Because stress cracking is the mode of final failure, stress cracking is considered to be the most appropriate determinant of end of life when data is available, as stated by Jafari, N.H., Stark, T.D. and Rowe, R.K. (2014). “Service Life of HDPE Geomembranes Subjected to Elevated Temperatures.” Journal of Hazardous, Toxic and Radioactive Waste.

12.9 Anticipated Leachate Temperature Range and Estimated Service Life

The combustion of coal for electricity generation results in a coal ash waste product, known as a coal cumbustion product (CCP). CCPs include :

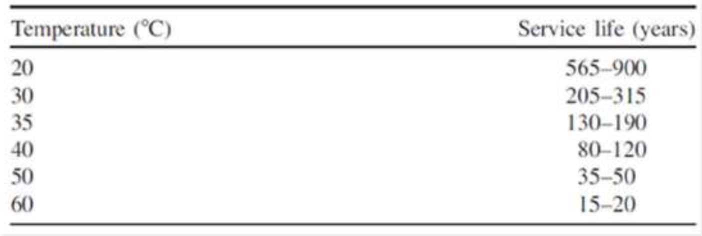
- Flue gas
- Boiler slag
- Bottom ash, and
- Fly ash

The ASTM C618 specification defines two classes of Fly Ashes- Class C and Class F. Class C ashes typically have calcium in the range of 10%- 30%, as CaO (free lime). Class F ashes are generally low Calcium ash with CaO less than 10%. The ash therefore presents a lower potential for the hydration of pozzolans within the ash and subsequently a lower temperature build-up is expected in the ADF than typical coal ash dumps.

Cognisnace will be taken of heat generation within the waste body, in the design of the barrier system. Temperatures are not envisaged to exceed 60 deg Celcius at the time of placing fresh ash due to the initial pozalinic action, which over time will no longer occur resulting in lower continuous thermal readings. Further to this, the presence of a protection geotextile and a 300mm layer of pioneering sand / coarse ash will thermally insulate the liner from the temperatures within the ADF body. Thermal monitoring probes will be installed on the liner surface to confirm assumptions made during the design process.

From the table below, the anticipated temperature will be 35 deg Celcius resulting in a service life of at least 130 years.

Table 8: Estimated HDPE Geomembrane Service Life Based on 50% Reduction in Tensile Strength at Break for Different temperatures (Based on Rowe (2005))



Temperature (°C)	Service life (years)
20	565–900
30	205–315
35	130–190
40	80–120
50	35–50
60	15–20

12.10 Tensile strains in HDPE Geomembrane

The two key related tensile strains to be considered in a Geomembrane (GMB) are the following:

- Local indentations of GMBs induced by the overlying cover materials (minimal due to protection geotextile and coarse ash protection layer)
  and
- Down-drag load for GMBs on side slopes generated by waste settlement on steep side slopes (not applicable)

12.11 Tensile strain from indentations in basal liner

Point load peak strain tests (protection efficiency test) will be carried out prior to construction, using actual materials to be used for the liner system to confirm that the combination of materials does not result in liner strain exceeding 3%.

12.12 Monitoring Equipment

As a result of several assumptions made during the design process, it is a requirement to ensure these assumptions can be measured and confirmed during the operational phases of the project.

Thermocouple Temperature Monitoring Probes

Thermocouple probes will be places at strategic positions throughout the ADF body during the construction phases. These will be monitored in accordance with the proposed monitoring plan to confirm the temperatures to which the HDPE lining system is exposed.

An additional thermocouple will be placed above the sand / ash protection layer to determine the insulation effects that the 300mm layer has on the HDPE liner.

Further to this, it has been proposed to place sacrificial tokens of all lining components (HDPE, GCL and Geotextiles) within the ADF body for retrieval and testing after several years of operation.

Strain Monitoring Devices

It is essential that the strain within the HDPE liner does not exceed 3%, this combined with potential elevated temperatures could have a drastic effect on the service life of the lining system. For this reason, strain gauges will be placed at positions of anticipated elevated strain (foot and crest of perimeter and separation berms). These gauges are to be monitored regularly to confirm the strains induced on the HDPE membrane lining system.

12.13 Service life of other materials

Other materials in the landfill to be considered in terms of Service Life include the following:

- drainage system pipes,
- drainage stone.

The drainage pipes will be perforated PE100 PN12,5 HDPE pipes manufactured according to SANS ISO 4427 and confirmed to withstand the external loading. It is estimated that the service life of HDPE pipes is approximately 190 years in landfill usage. In addition to material strength, the pipe diameter should be sufficient that biological clogging would not occur. Clogging could take place within the stone drainage layer as well. Clogging, both chemical and biological occurs more on hazardous waste sites and co-disposal site, as well as sites with high iron content in the soils.

12.14 Proven Equivalent Performance of Alternative Elements

The alternative elements are the GCL layer in place of the standard’s 300mm natural clay layers, and a protective Geotextile in the place of a silty sand layer. Testing will be done prior to construction to ensure that:

- The GCL should have a permeability of not more than 1 x 10^{-7} cm/s (0,3 m/year).
- The bentonite within the GCL has a satisfactory swell index with site soil and ash specific permeants.
- The geotextile should prevent strain on the geomembrane exceeding a total strain of 3%, tested by means of the protection efficiency test

12.15 Lined Areas

The table below indicates the total lined area for the ADF. The second table indicates the liner areas for the stages of Phase 1. The total lined area for Phase 1 is 136 Hectares.

Table 9: Liner Surface Areas

Name	Area (m^2)	Area (Ha)
Phase 1	1 355 319	136
Phase 2 North	310 246	31
Phase 2 South-East	492 559	49
Phase 3 North	381 087	38
Phase 3 South	372 283	37
Phase 4 North	428 501	43
Phase 4 South	350 843	35
Phase 5	165 948	17
Phase 6	319 545	32
Phase 7	442 604	44
Phase 8	394 261	39
Phase 9	352 446	35
Phase 10	375 287	38

(Note that Phase 11 is on top of previous ash phases with no basal growth)

Table 10: Phase 1 Lined Areas

Name	Area (m^2)
Stage 1	200 670
Stage 2	228 715
Stage 3	213 934
Stage 4	228 456
Stage 5	483 672
TOTAL	**1 355 319**

13. STORMWATER MANAGEMENT

13.1 Water Management Components

The storm water management system is governed by GN 704 requirements and contains the following key infrastructure:

- Pollution Control Dams (PCDs): 1a, 1b and 2,
- Clean Water Dams (CWDS): 1 and 2,
- 2 temporary clean water Attenuation Dams (Dams 1 and 4)
- 3 permanent clean water Attenuation dams, (Dams 2, 3 and 5)
- Network of contaminated stormwater collection channels,
- Clean stormwater diversion berms,
- Clean stormwater drain,
- Stilling basins,
- River Diversion.

13.2 Stormwater Design Methods and Software

A combination of Autodesk Civil3D models, Storm and Sanitary Analysis (SSA), and Excel based spreasheets for Rational Formula and Manning's Equation have been used, with cross-checking between the various manual and automated processes. Hydrocube, a catchment analysis programme has been used as well for pipe and channel size confirmation as well as verifying the performance of chosen pipes.

Most hydrologic models are based on the EPA's Storm Water Management Model (SWMM) model. SWMM is a proven model design and analyzing urban and also rural drainage systems. SSA has a built-in capability to run a SWMM model, as well as TR-55, HEC-1, Rational method, and other hydrologic models. SSA is uniquely suited for land development design due to its integration with Civil 3D.

The Rational Method as described in the document Drainage Manual, Sixth Edition (2013) as published by SANRAL (South African National Roads Agency Limited) was used to determine the magnitude of the 1 in 50 year and 1 in 200 year flows.

The Rational Method is still probably the most commonly used method of estimating the peak runoff value of stormwater runooff generated from urban and rural areas where areas do not exceed 25km^2.

The formula used in this method is

$$\mathbf{Q= f_t \times C \times I \times A/360\ m^3/s}$$

Q= the maximum /peak rate of runoff in m^3/s

F_t= and adjustment factor for the recurrence interval storm considered

C= runoff co-efficient per applicable tables

I= the rainfall Intensity(mm/hr)

A= area of catchment in hectares

The runoff co-efficient is a factor ranging from 0 to 1 which compensates for variations in rainfall over the catchment, infiltration and overland flow velocity during a storm. C can be determined from the Table Method used by the DWAF for consistency of approach.

The initial input was the 0.5 m interval contours that were obtained from the lidar survey. The software Autodesk Civil3D was used to extract the channels network and longsections of the channels. The channels network and longsections were imported into the software “SSA” automatically and calculations were processed. The output generally provides alternative methodolgy’s results, for comparison. ‘Before’ and ‘after’ models can be compared, and earthworks cut and fill volumes calculated.

Data output as well as the full Stormwater Management Plan can be found attached in **Appendix C**.

13.3 Upslope cut-off drain

The regional catchment upslope of Kusile ADF is being diverted around the ADF by the clean upslope stormwater cut-off drain on the Power Station property western and southern boundaries. This cut-off drain, currently under construction, will decrease the catchment of the Tributary Stream significantly, once complete.

13.4 Perimeter Drainage Channels

13.4.1 Design Philosophy

A system of three open drains is proposed for the ADF surface water drainage, namely a clean outer drain direct-discharging to the environment, a middle dirty drain for polluted water, and an inner “dirty-to-clean” drain.

The clean drains running east-west to the south (up-slope) of the ash dump will be deemed temporary, and when the footprint extends, these drains will be demolished, and duplicated further upslope, south of the following phase of the ash dump. With each phase the permanent clean drains to the east and west of the ash dump will be extended and joined to the current temporary southern drain. The Phase 1 drains are shown on drawing 366-511895.

The dirty-to-clean drain is the first drain to be utilised, collecting contaminated run-off from the first area receiving ash, from the ash dump surface and side slopes, where that runoff is not contained behind the perimeter berm and draining into the leachate collection system in the ADF base. Water collected in the dirty-to-clean drain ultimately drains to a PCD.

As the next phase becomes the active disposal area, the first phase is to be progressively covered and rehabilitated. The dirty drain is to be constructed to serve the second active ash disposal area, whilst the first leg of the dirty-to-clean drain is by-passed, allowing the first leg to become isolated from upstream runoff, and become progressively cleaner.

Eventually, when serving the final rehabilitated landform, this innermost dirty-to-clean drain becomes the final clean toe drain. The concept is described by way of a schematic figure shown below.

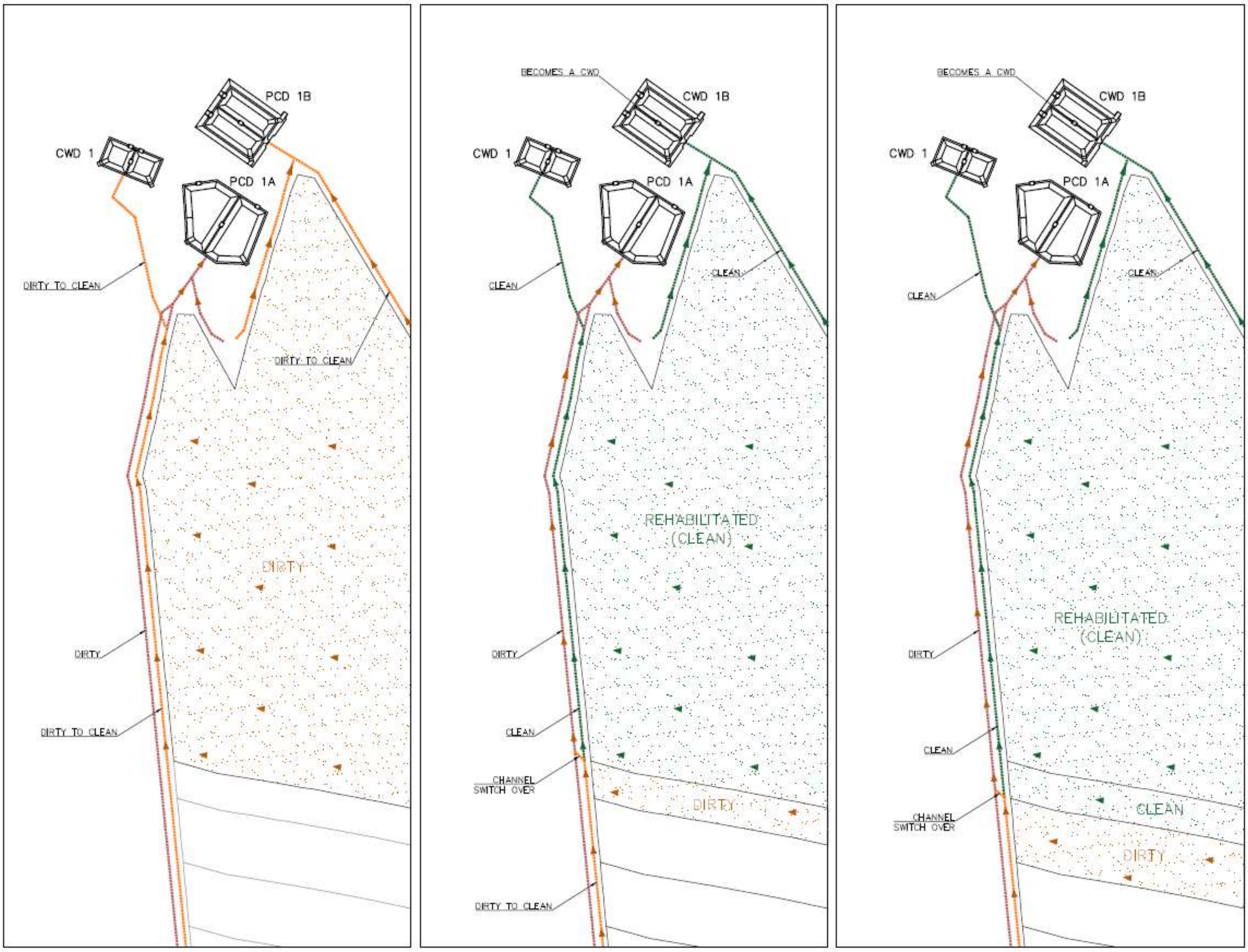


Figure 13: Schematic Open Channels Arrangement

13.4.2 Polluted Water System

The internal drainage system under the ADF basal barrier collects sub-soil water. The internal drainage system above the basal barrier collects leachate from the base of the ash dump. Both these networks are perforated HDPE pipes, that gravity-drain to main collector pipes, which would be solid wall HDPE pipes. The leachate collector pipes will convey the leachate out through the perimeter berm with a sealed pipe penetration. The pipe will discharge the leachate into the “dirty” channels.

13.4.3 Clean Water Stormwater Cut-off Drain

A local clean-water cut-off drain will wrap around Phase 1 of the ash dump footprint. The drain portions running east-west to the south (up-slope) of the ADF will be deemed temporary, and when the footprint extends, these drains will be duplicated further upslope, south of the following Phase of the ADF.

In principle, the clean cut-off drains flow around the ADF into permanent clean water drains both on the east and west of the ADF to discharge into the natural stormwater regime. These clean drains are not to be confused with the existing 12m wide upslope cut-off drain on the site boundary, which is much larger and channels the off-site catchment runoff around the Kusile site.

13.4.4 Clean Water Stormwater By-pass Drains

During the construction of the attenuation dam structures and diversion pipeline, clean water diversion channels will be constructed to divert water away from the central pipeline basin routing, technically de-watering the channel basin for ease of construction of the pipeline. These channels will be earth lined temporary channels that will be filled in during the basal excavations of the ADF.

13.4.5 Catchment analyses

The stormwater water management systems normally associated with a waste site address three types of runoffs which are:

- uncontaminated upslope run-off,
- contaminated run-off from the landfill surface and side slopes, and
- leachate generated within the waste body.

The site was divided into sub-catchment areas to delineate clean and dirty systems. Runoff calculations were performed for the sub-catchment areas in order to determine the size of the required drains. The catchments are outlined on drawing 366-511894.

13.4.6 Uncontaminated upslope run-off analysis methodology

All upslope run-off water must be diverted away from the waste, to prevent water contamination and minimise leachate generation. The uncontaminated upslope runoff will be prevented from entering the landfill facility area by means of trapezoidal diversion drains along the higher southern, western and the eastern side of the landfill. These drains will end in a velocity reducing structures from where the water will daylight north-east and north-west of the facility.

Unique to this site are two central valleys of the Holfonteinspruit and its western tributary that flow generally northwards and join together. As described in sections above, the streams will be diverted through a large diameter reinforced concrete pipe. After the installation of the clean cut-off drain on the southern and western boundaries, the remaining upslope run-off would be the on-site catchment and the catchment beyond the ADF boundary to the east, currently the New Largo coal mine property.

The catchments reporting to each pipeline were delineated, and the runoff volumes were calculated. In addition to runoff volumes through each leg of the pipelines, peak flows were calculated. A design storm of 1:200 year recurrence interval was utilised.

Calculations were carried out using the computer programme Storm and Sanitary Analysis (SSA), which is an Autodesk product and compatible with Civil3D design and drafting software. Results are included in Appendix C. The output was compared to the results obtained in the conceptual designs thus far, by others, and found to be in the same order.

From the peak flows calculated, various sizes of pipe and channel could be tested with the Manning’s Equation. Through an iterative process in Excel, a suitable pipe size could be chosen. For concrete trapezoidal drains, a Mannings n value of 0,015 was chosen, and for the pipe, a n value of 0,012.

The required size of the eastern leg pipeline was found to be a 2,0m diameter pipe. The required size of the western leg pipeline was found to be a 1,5m diameter pipe. The last, combined portion was required to be 2,5m diameter pipe. For ease of access for inspection, and for manufacturing, pipe sizes will only be 2,0m and 2,5m diameters.

13.4.7 Contaminated surface run-off analysis

Surface run-off from the landfill perimeter berm outer slopes, roads and waste handling areas are considered to be potentially contaminated and should not enter natural drainage courses. Contaminated run-off is to be directed towards the concrete lined trapezoidal drains along the outer toe of the ADF perimeter berms.

In sizing these drains, the worst-case scenario was considered and found to be the scenario of the largest lined-and-contaminated area for the duration of diposal. This would be the case when a new phase is lined, and the previous phase is not yet capped.

A combination of Phases 1 and 2 was considered at the worst case, and runoff volumes and peak flows were calculated based on this catchment, and as per methodolgy described above.

13.4.8 Channel Design

The temporary clean cut-off drain, dirty-to-clean drain and the dirty drain would all be concrete lined trapezoidal channels with a minimum base width of 1m and minimum depth of 0,5m. Concrete is to be a minimum 100mm thick, 35 MPa strength with mesh ref 193 reinforcing. The channels are to have mesh ref 245 reinforcement at crossings and drive-throughs. Channels are to be cast in alternate panels, not exceeding 4,5m in length. Prior to placing concrete, in-situ base soil should be ripped and recompacted and a sand blinding layer placed if necessary. Lengths of channels are tabled below.

Gradients of channels are designed such that velocities during low flows ensure self cleansing occurs (de-silting of the channel). De-silting is expected to be undertaken by a small excavating machine from time to time. The plant used for this purpose should be a light-weight machine with a narrow bucket (maximum weight 8,0t). Heavy dozers may not be used for this purpose.

Table 11: Channel lengths

Name	Length (m)
Dirty water concrete trapazoidal channel - West	397
Dirty water concrete trapazoidal channel - North 1	185
Dirty water concrete trapazoidal channel - North 2	468
Dirty water concrete trapazoidal channel – North East	965
Temp dirty water concrete trapazoidal channel - East	1461
Temp dirty water concrete trapazoidal channel – South East	405
Temp dirty water concrete trapazoidal channel – South 1	535
Temp dirty water concrete trapazoidal channel – South 2	205
Temp dirty water concrete trapazoidal channel – South 3	274
Dirty/ clean water concrete channel - West	1264
Temp clean water concrete channel – East	1667
Temp clean water concrete channel – South East	430
Temp clean water concrete channel – South 1	534
Temp clean water concrete channel – South 2	205
Temp clean water concrete channel – South 3	274
Road dirty water concrete channel – North West	135
Road dirty water concrete channel – South East	42
TOTAL LENGTH OF LINED CHANNELS	9446
TOTAL LENGTH OF UNLINED BY-PASS DRAINS	3800

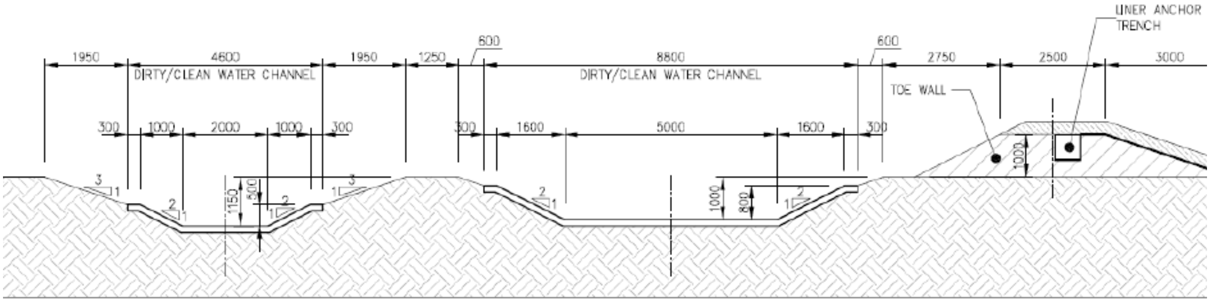


Figure 14 - Typical Channel Configuration

13.4.9 The attenuation water diversion (by-pass) drains

The drains spilling from the Attenuation dams are referred to as “by-pass” drains to differentiate these drains from other clean water cut-off drains. The by-pass drains are to be lined with 100mm thick, nominally compacted topsoil-rich soil. A soil-preservation loosely woven, bio-degradable fabric is to be installed to preserve the topsoil whilst vegetation establishes. The by-pass drains are at an almost flat gadient in some cases and flow will be low-velocity. The excavated material from the drain is to be placed on the downslope side as a low diversion berm. See **Figure 18** below.

14. ATTENUATION DAMS

A total of five (5) attenuation dams will need to be constructed during Phase 1 of the 60yr ADF project in order to divert, capture and reticulate the stream diversion water and facilitate construction activities downstream. Temporary attenuation dams will only serve to contain and divert runoff during construction downstream, and will not require a controlled underdrain outlet. Temporary attenuation dams will drain into a temporary diversion channel discharging to the environment.

The permanent attenuation dams will have spillways as well as controlled-outlet underdrains which will feed into the stream diversion pipelines. Three (3) permanent attenuation dams shall consist of a permanent inlet / outlet system and are expected to remain in place for 15 to 35 years.

14.1 Geological Profiles

The general soil profile on site confirms the geology according to the geological map 2528 Pretoria with sandstone underlain by tillite and shale on the northern part of the site.

The generalized profile in the valley line can be summarised as follows:

0.0 – 1.2m	Topsoil, Alluvium and Colluvium - Ferruginised in places
1.2 – 2.0m	Residual Material
2.0 – 4.4m	Bedrock – Refusal on Tillite, Sandstone and Shale.

Refusal varies due to the presence of a ferricrete layer and the uneven weathering of the local bedrock.

The pipe foundations can be founded on the highly weathered tillite/sandstone at a depth of 3.0 – 3.5m below NGL. Foundations can be designed for a bearing capacity of 150 kPa at this depth.

No suitable engineering fill is present along the pipe diversion route. Therefore, a G6 quality material must be sourced from a reliable source close to, or on site. Shallow diabase material was present at AD TP23 – 25 that can be further explored if possible.

14.2 Proposed Layout

Figure 15 below (extracted from Drawing 366-511846) shows the proposed layout and construction ordering of the attenuation dams and channels to be constructed during Phase 1.

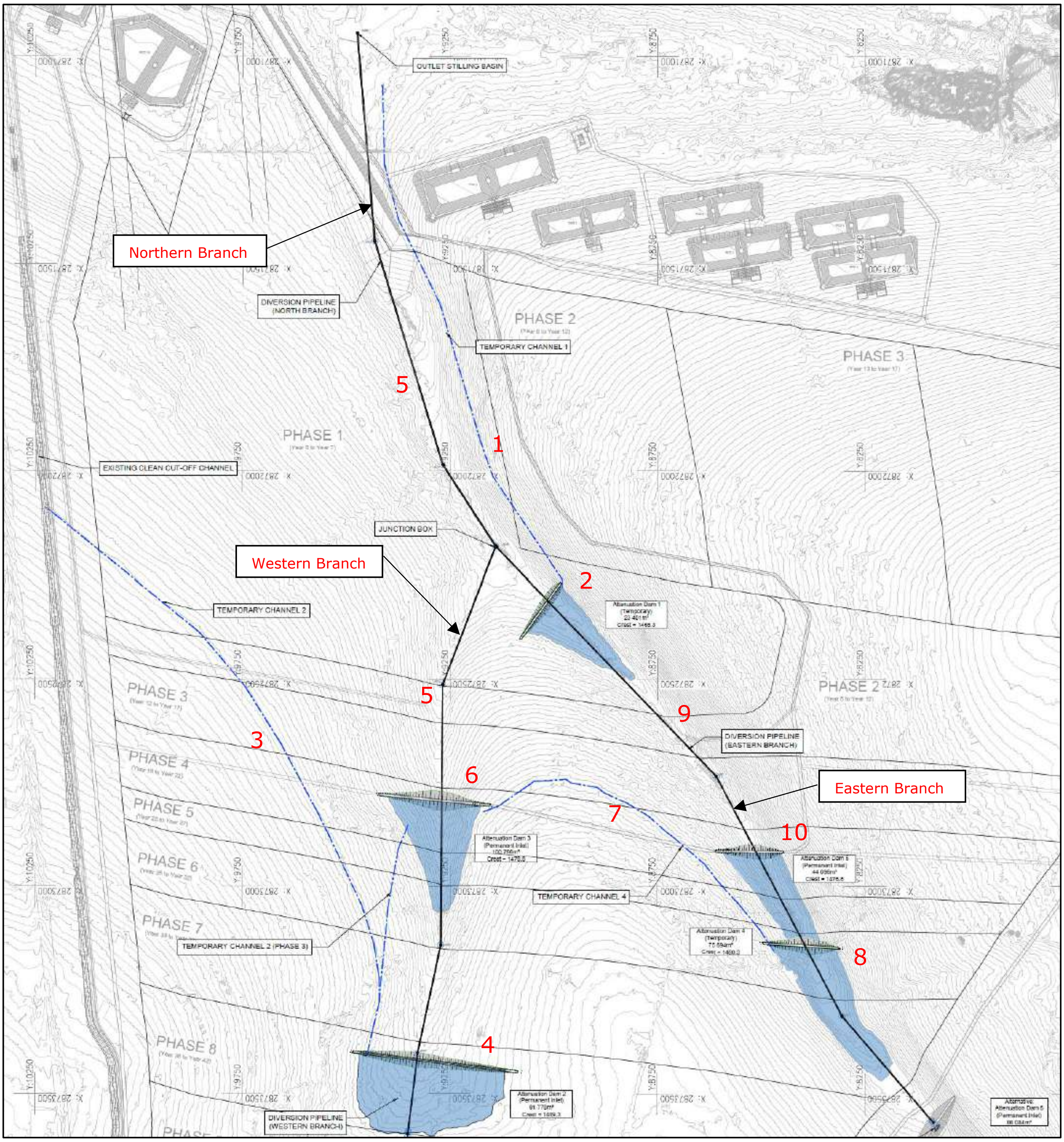


Figure 15 - Attenuation Dam Layout

14.3 Design Rationale

Refering to the Figure 6 above the envisaged order of construction shall be as follows:

1. Construct temporary diversion channel No. 1 to daylight to river area (earth lined channel)
2. Construct temporary attenuation dam No. 1 (earth embankment) to feed channel No. 1.

The northern branch has now been de-watered of any inflow from the eastern branch tributary.

3. Construct temporary diversion channel to tie into existing clean water channel (earth lined channel)
4. Construct permanent inlet attenuation dam No. 2 (clay core dam) to feed channel No. 2.

The northern branch has now been de-watered of any inflow from the western branch tributary.

5. Construct stilling basin, northern branch pipeline, junction box and western branch pipeline.
6. Construct permanent inlet attenuation dam No. 3 and connect western pipeline branch to outlet chamber.
7. Construct temporary diversion channel No. 4 (earth lined channel)
8. Construct temporary attenuation dam No. 4 (earth embankment) to feed channel No. 4.

The upper eastern branch has now been de-watered of any inflow from the upstream eastern branch tributary.

9. Demolish temporary attenuation dam 1 and construct eastern branch pipeline tying into main junction box with northern and western branches.
10. Construct permanent inlet attenuation dam No. 5 and connect eastern pipeline branch to outlet chamber.

Attenuation dam No. 4 can now be demolished.

Temporary diversion channel No. 2 can be re-routed to feed to Permanent Inlet Attenuation Dam No. 3 body.

14.4 Summary of Dam Information

Table 12 below summarises each dam's phycial characteristics:

Table 12 - Dam Information

Dam No.	Type	Composition	Crest Lvl	Spillway Lvl	Max Crest Height	Crest Width	Length	Storage Volume	Classification - DWA	Estimated Service Lifespan
1	Temporary	Homogenous Earthfill	1465.3	N/A	4.3m	5.0m	167.7m	23 401 m^3	Class I	2-3 years
2	Permanent Inlet	Zoned Earthfill Clay Core	1489.3	1488.5	5.9m	5.0m	400.3m	81 778 m^3	Class II	35 years
3	Permanent Inlet	Zoned Earthfill Clay Core	1478.8	1478.0	8.2m	5.0m	270.0m	100 788 m^3	Class II	15 years
4	Temporary	Homogenous Earthfill	1480.3	N/A	7.8m	5.0m	179.3m	75 694 m^3	Class I	2-3 years
5	Permanent Inlet	Zoned Earthfill Clay Core	1476.8	1476.0	5.8m	5.0m	163.8m	44 098 m^3	Class II	15 years

The **dam safety legislation** is covered by chapter 12 of the National Water Act, 1998 (Act 36 of 1998) [NWA] and by dam safety regulations, published in Government Notice R. 139 of 24 February 2012. Only dams with a safety risk (that is dams with a maximum wall height exceeding 5,0 m **and** with a storage capacity exceeding 50 000 m3, or any other dam declared by the Minister as a dam with a safety risk) are subject to these Regulations.

The attenuation dams for the Kusile 60 year ADF project that meet this dam safety requirement above are attenuation dams 2, 3 and 4.

14.5 Attenuation Dam Composition

14.5.1 Earthfill Embankment Dams – Temporary

Dams No. 1 and No. 4 consist of a homogenous earthfill body of clayey silty sand obtained from local excavations and compacted to 95% Mod AASHTO in layers of 150mm (see **Figure 16** below).

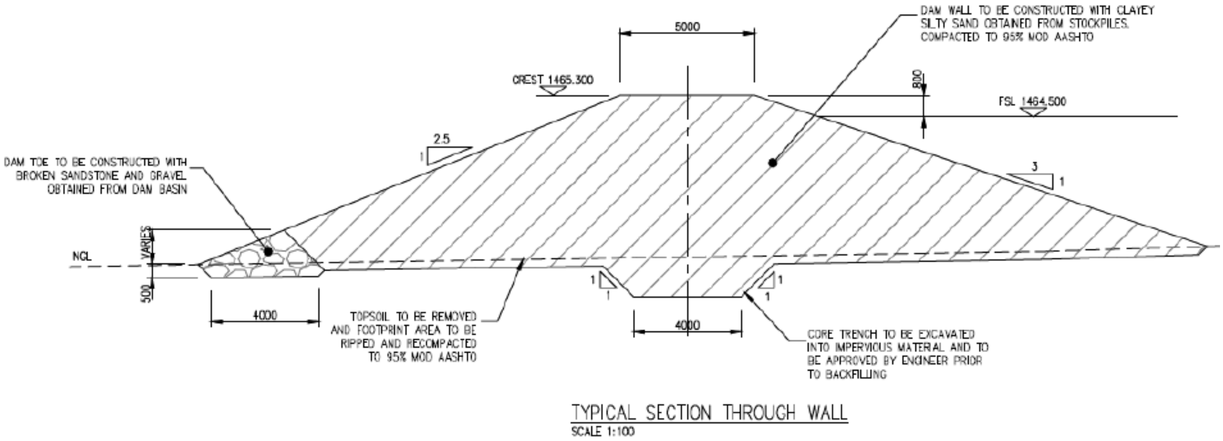


Figure 16 - Homogenous Earth Fill Dam Typical Section

Dams shall consist of a 1:3 slope on the upstream face and a 1:2.5 slope on the downstream face. Dam shall be anchored into the natural ground by means of a 4.0m wide core trench approximately 2.0m deep (excavated into impervious material and to be approved by engineer).

The dam shall consist of a downstream drainage toe constructed from broken sandstone and gravel obtained from local excavations. While attenuation dams No. 1 and No. 4 are of a temporary nature, the contractor is to ensure any erosion or excessive vegetation growth on the dam walls is managed during their lifespan.

14.5.2 Clay Core Embankment Dams – Permanent Inlet

Dams No. 2, No. 3 and No. 5 comprise of a zoned earthen fill dam with clay core (see **Figure 17** below).

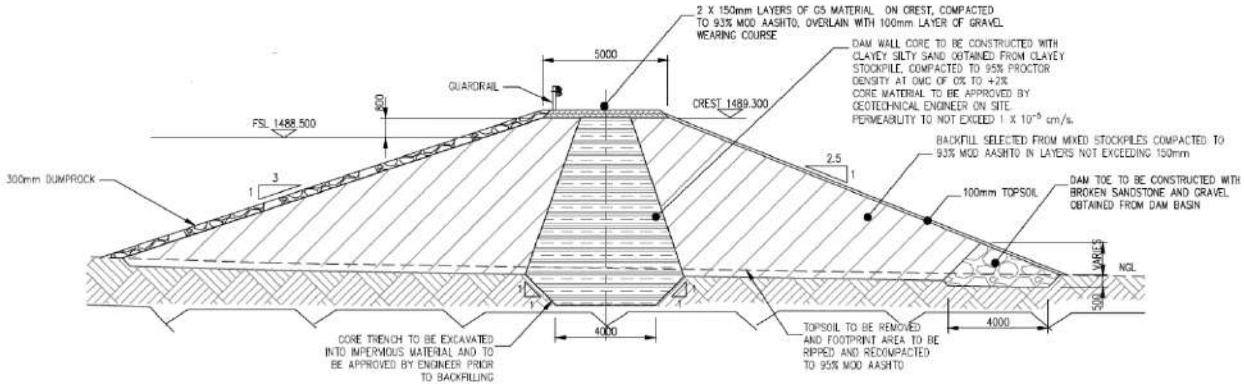


Figure 17 - Zoned Clay Core Dam Typical Section

The clay core shall consist of clayey silty sand obtained from clayey stockpile (from other local excavations) compacted to 95% Proctor Density at OMC of 0% to +2%. Clay core material to be approved by the Geotechnical Engineer on site during construction with permeability not exceeding 1x10^{-5}cm/s.

The outer zones of the dam shall be constructed of material from mixed stockpile from local excavations to 93% MOD AASHTO in layers not exceeding 150mm. This material is also to be selected and approved by the Geotechnical Engineer on site during construction.

Dams shall consist of a 1:3 slope on the upstream face and a 1:2.5 slope on the downstream face. Dam shall be anchored into the natural ground by means of a 4.0m wide core trench approximately 2.0m deep (excavated into impervious material and to be approved by engineer). The upstream face (attenuating face) of the dam shall be covered with 300mm of dump rock for erosion protection (from wave action) while the downstream face should be topsoiled (100mm) and hydroseeded.

All permanent inlet attenuation dams shall have a 5.0m wide crest wearing course (2 x 150mm layers) of G5 material compacted to 93% MOD AASHTO. As dams No. 3 and No. 5 shall be used by vehicles to access the site across the drainage valley, guard rails have been positioned on both sides of the dam wall. Dam No. 2 which will not have vehicles using it during Phase 1 will only have guard rails on the attenuated water side of the dam wall crest.

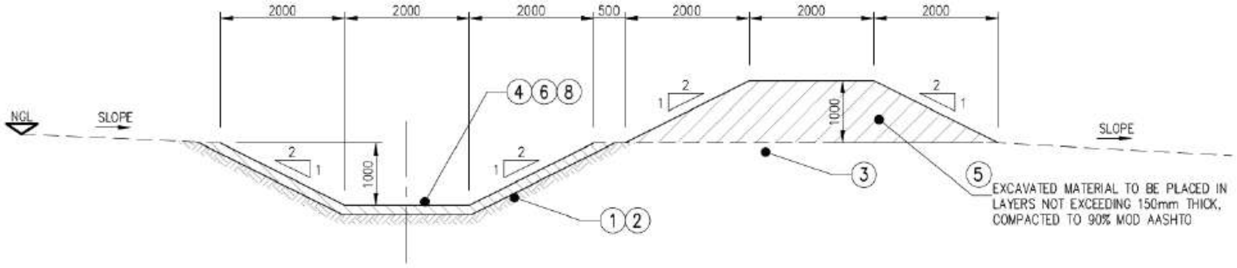
14.5.3 Spillways

Permanent inlet dams (No. 2, No. 3 and No. 5) shall comprise of a mesh reinforced spillway drift while temporary dams (No. 1 and No. 4) shall spill to their respective diversion channels at the full supply level of the dam / invert of the diversion channel.

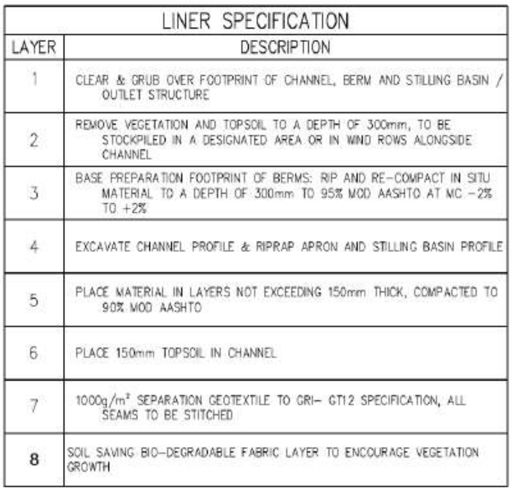
Permanent dam spillway drifts shall be mesh ref. 395 reinforced, 300mm thick and a total of 45.0m long. Energy disspators consist of 26 fig. 7 pre-cast kerbing cast into the spillway itself – refer to drawing 366-513548 for the spillway detail.

14.5.4 Diversion Channels

Dams No. 1 and No. 4 shall have diversion channels constructed leading from their spillway full supply level draining to the natural river area and clean water channel respectively. Diversion channels will be earth lined as shown in **Figure 18** below and drawing 366-511924.



EXCAVATED MATERIAL TO BE PLACED IN LAYERS NOT EXCEEDING 150mm THICK, COMPACTED TO 90% MOD AASHTO



LINER SPECIFICATION	
LAYER	DESCRIPTION
1	CLEAR & GRUB OVER FOOTPRINT OF CHANNEL, BERM AND STILLING BASIN / OUTLET STRUCTURE
2	REMOVE VEGETATION AND TOPSOIL TO A DEPTH OF 300mm, TO BE STOCKPILED IN A DESIGNATED AREA OR IN WIND ROWS ALONGSIDE CHANNEL
3	BASE PREPARATION FOOTPRINT OF BERMS: RIP AND RE-COMPACT IN SITU MATERIAL TO A DEPTH OF 300mm TO 95% MOD AASHTO AT MC -2% TO +2%
4	EXCAVATE CHANNEL PROFILE & RIPRAP APRON AND STILLING BASIN PROFILE
5	PLACE MATERIAL IN LAYERS NOT EXCEEDING 150mm THICK, COMPACTED TO 90% MOD AASHTO
6	PLACE 150mm TOPSOIL IN CHANNEL
7	1000g/m² SEPARATION GEOTEXTILE TO GRI- GT12 SPECIFICATION, ALL SEAMS TO BE STITCHED
8	SOIL SAVING BIO-DEGRADABLE FABRIC LAYER TO ENCOURAGE VEGETATION GROWTH

Figure 18 – Diversion Earth Channel Typcial Section

14.5.5 Inlet Chamber

The inlet chambers for the permanent attenuation dams shall consist of a reinforced concrete chamber to the height of the incoming pipe soffit. Inlet weirs / openings shall be three (3) in total (2 No. of x 2.0m x 1.0m, 1 No. of 3.0m x 1.0m) in size with flap gates fitted (see **Figures 19 & 20** below).

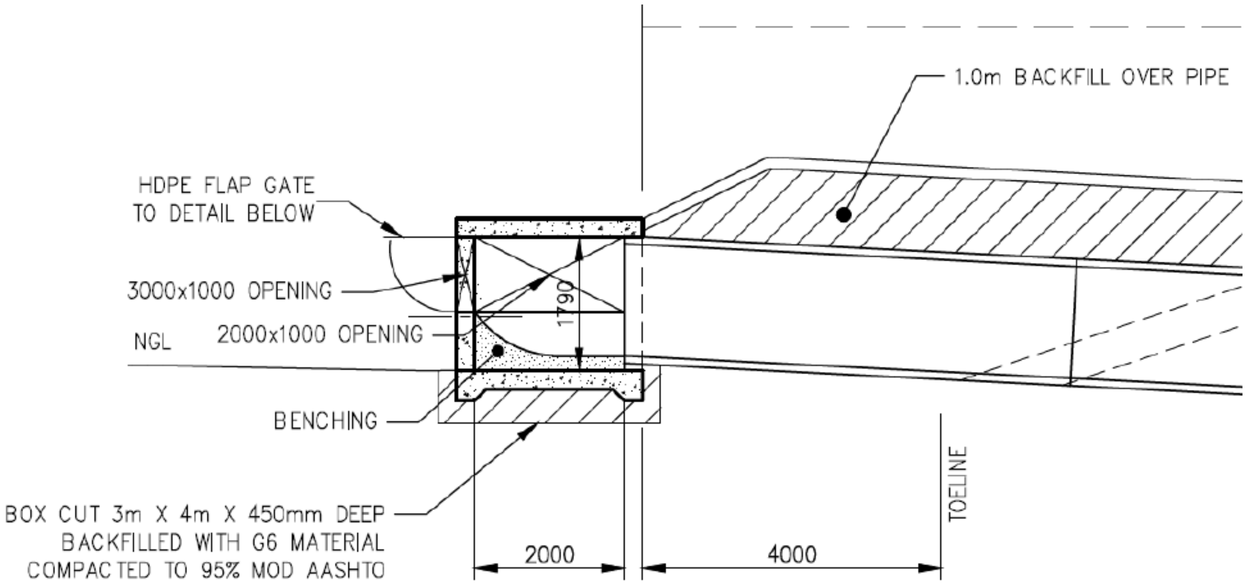


Figure 19 - Inlet Chamber Typical Section

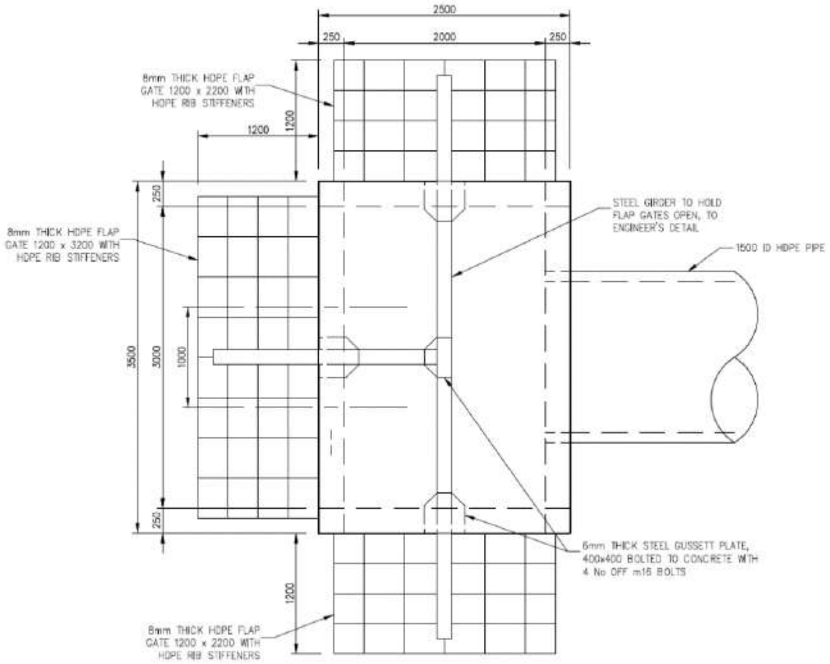


Figure 20 - Inlet Chamber Plan View

Flap gates will allow for the isolation and sealing of the inlet chamber and downstream pipeline for maintenance, inspections and emergencies. Flap gates shall be constructed out of solid HDPE (capable of withstanding 200kPa pressure) similar to **Figure 21** below.

Figure 21 - Isolation Flap Gate

As the invert of the flap gate opening will sit between 0.5m and 1.0m above the NGL, silt will settle here before spilling into the inlet chamber and through the pipe. It is essential that the de-silting around the base of the inlet chamber occurs regularly to prevent excessive build up and possibly silt entering into the pipeline through the inlet weir spillways.

In order to isolate the inlet chamber, flap gates are to be lowered by means of the winch and steel girder support system and closed. Once water builds up against the flap gate, hydrostatic pressure will seal the gates in position this isolating and sealing the inlet chamber.

14.5.6 Outlet Chamber

In order to access the pipeline during maintenance and regular inspections, personel and / or cctv drone / rovers can access the pipeline through the outlet stilling basin or the proposal outlet chamber at the downstream face of the attenuation dams. The outlet chambers shall consist of a reinforced concrete chamber with a standard pre-cast concrete manhole ring and cover on the chamber slab (see **Figure 22** below). During isolation operations of the inlet chamber, access to the pipeline can be made through the outlet chamber to confirm the isolation and sealing of the inlet chamber and its flap gates.

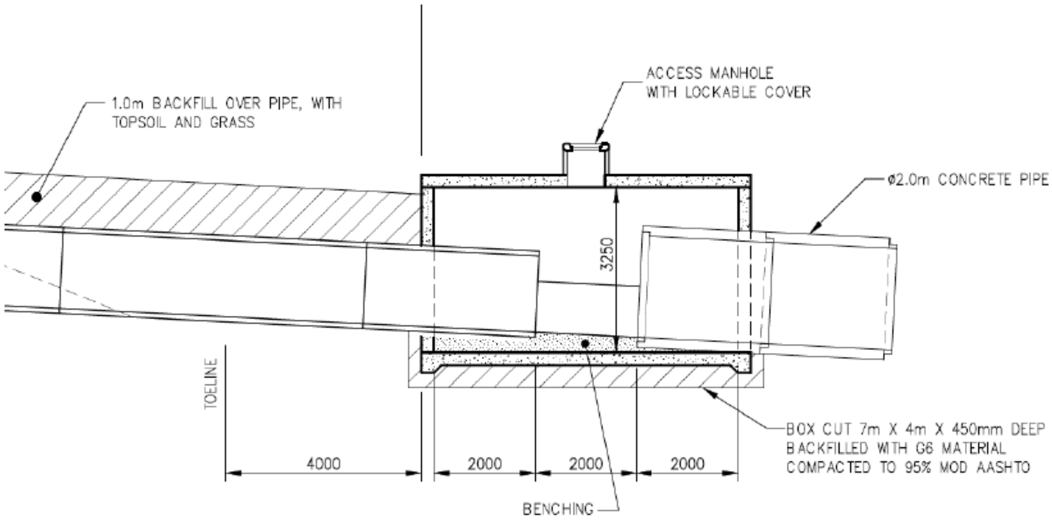


Figure 22 - Outlet Chamber Typical Section

14.5.7 Pipe in Dam Body

The pipe within the permanent inlet attenuation dam body shall consist of a 1.5m diameter HDPE class 16 pipe with 2.5m diameter puddle flanges placed and welded to the pipe at the start of both outer embankments as well as either side of the clay centre core. The puddle flanges shall act as a wall to prevent water from flowing along the outer surface of pipe from the attenuated upstream face to the outlet downstream face.

14.6 Stability

Stability analyses were carried out using GEO5 2024, a slope stability program which computes the stability of slopes and embankments with circular or polygonal slip surfaces. In terms of polygonal or non-circular slip surfaces, the programme can perform the Bishop, Spencer, Janbu, Morgenstern-Price and Fellenius / Petterson methods. Results from all methods were used for this analysis.

The results of the stability analyses carried out on Attenuation Dam No. 3 (largest dam) are given in **Table 13** below.

Table 13 - Stability Results

Software	FoS	Method
GEO5 2024	2.19	Morgenstern-Price
	2.19	Bishop
	2.15	Fellenius / Petterson
	2.19	Janbu
	2.19	Spencer

All factors of safety are greater than the minimum FoS of 1.3 and are therefore acceptable.

Visual representation of the stability output can be seen below in **Figure 23** and attached in **Appendix C**.

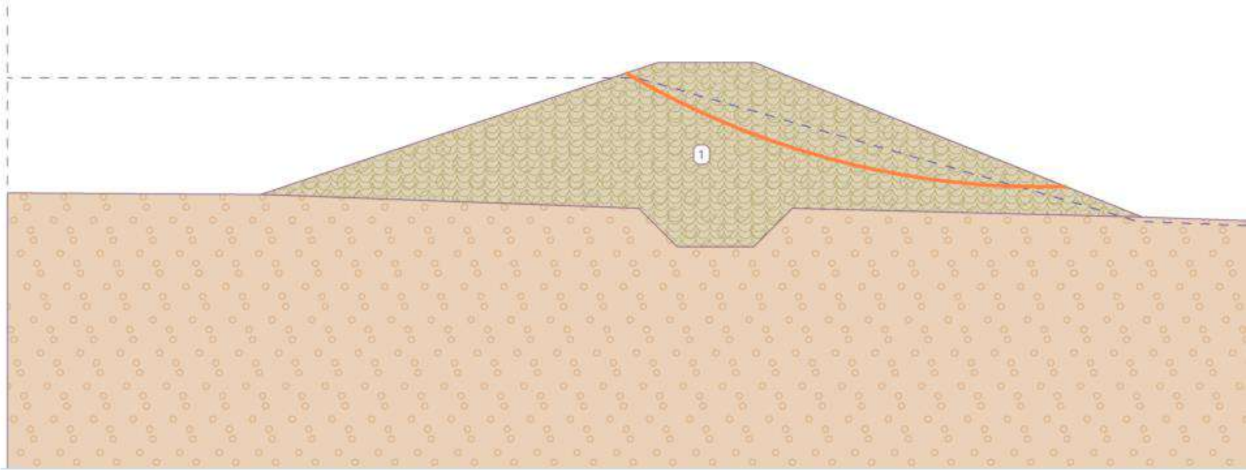


Figure 23 - Stability Output

15. STREAM DIVERSION

The ash dump footprint straddles the Holfonteinspruit and its' south-western tributary. These streams will each be routed through a large diameter pipeline under the ash dump. The two streams would join into one pipeline, about halfway down the ash dump footprint, which then discharges back into the original stream at the convergence with the Klipfonteinspruit.

15.1 Stream Diversion Alignments

The pipelines have been designed to align with the centre of the drainage lines as far as possible, and close to the existing natural ground level. Existing ground levels have been assumed and will need to be confirmed with detailed physical survey prior to construction.

As the horizontal alignment is determined by the existing topography, the pipelines do not run in a straight line. So, in addition to a cast-in-situ concrete junction box at the joining of the two streams, there would be cast-in-situ junction structures at bends. The number of junctions has been optimised, taking both horizontal and vertical alignment into consideration.

The pipelines run in a straight line and on a single gradient between junction boxes.

15.2 Geological Profiles

The general soil profile on site confirms the geology according to the geological map 2528 Pretoria with sandstone underlain by tillite and shale on the northern part of the site. A summary Geotechnical Report is included in **Appendix B**.

The generalized profile in the valley line can be summarised as follows:

0.0 – 1.2m	Topsoil, Alluvium and Colluvium - Ferruginised in places
1.2 – 2.0m	Residual Material
2.0 – 4.4m	Bedrock – Refusal on Tillite, Sandstone and Shale.

Refusal varies due to the presence of a ferricrete layer and the uneven weathering of the local bedrock.

The pipe foundations can be founded on the highly weathered tillite/sandstone at a depth of 3.0 – 3.5m below NGL. Foundations can be designed for a bearing capacity of 150 kPa at this depth.

No suitable engineering fill is present along the pipe diversion route. Therefore, a G6 quality material must be sourced from a reliable source close to, or on site. Shallow diabase material was present at AD TP23 – 25 that can be further explored if possible.

15.3 Groundwater

Groundwater seepage in the valleys was encountered between 0.7 to 3.4m below NGL and was also dependant on the proximity to the water course. The seasonal flow in the water courses varies, but even in winter 2023 at a site inspection, there was water flowing at a depth ranging from 0,5m to 1m.

15.4 Constraints and Geotechnical Considerations

Due to saturated nature of the sub-soil and the proximity of the water course, stability of the test pits played a big factor. Therefore, precaution must be taken during excavation by shoring or benching the box cut.

It is recommended to excavate the entire pipe position with an initial trench to ensure seeping water drains away from the construction site and that the construction should be started at the upstream position of the stream diversion pipeline.

A pioneer layer made up of dump-rock, at least 300mm thick, should be constructed as a drainage layer below foundation levels with the engineered base layers for the pipe bedding material on top of the dump-rock.

The dump-rock can be compacted with at least 8 passes with a drum roller.

A separation geotextile is to be placed on top of the dump-rock with a G5 class material compacted to at least 95% Mod AASHTO that would form the base for the founding material.

It is also critical that all material must be placed in layers no thicker that 150mm and compacted to 95% Mod ASSHTO around the entire pipe diameter to ensure the integrity of the pipe.

The proposed cross section is shown below in drawing 366-511918.

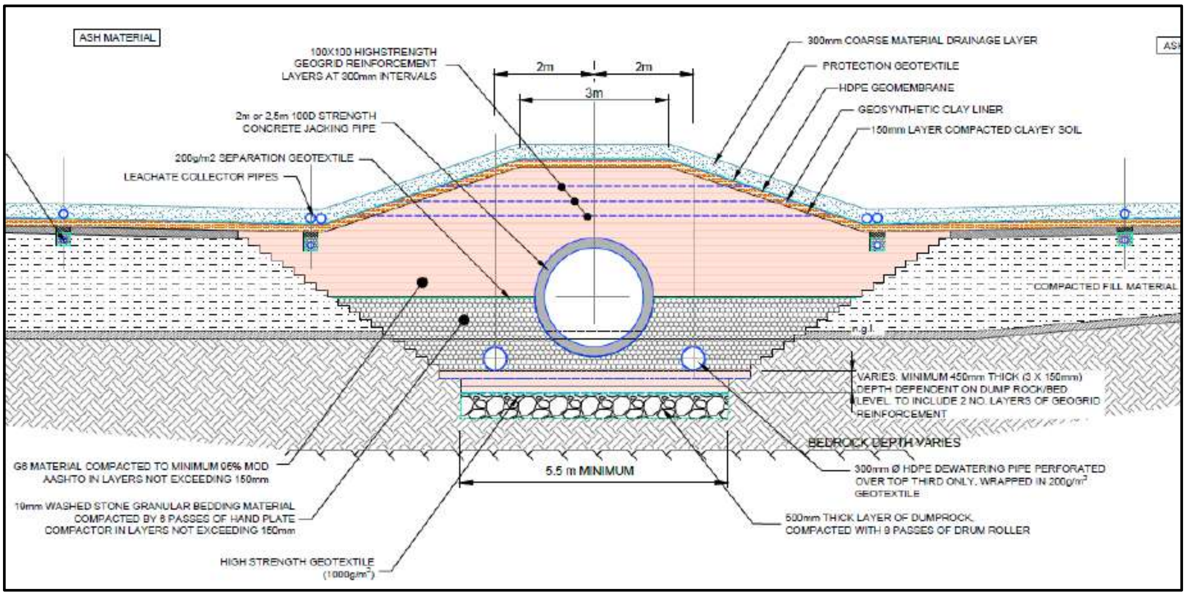


Figure 24: Typical section through stream diversion pipeline

15.5 Catchment analyses

The catchments reporting to each pipeline were delineated, and the runoff volumes were calculated. Catchment no.1 is the Catcment for the south-western tributary and Catchment no. 2 is the catchment for the Holfonteinspruit. The Basis for Design input is tabled below. In addition to runoff volumes through each leg of the pipelines, peak flows were calculated. A design storm of 1:200 year recurrence interval was utilised.

Table 14: Pipelines Catchment Data

Catchment	1	2
Total Catchment Area	220.203 ha	378.699 ha
Mean Annual Precipitation	697mm	697mm
SAWB MAP Station	Wilgerivier (SAR)	Wilgerivier (SAR)
Impervious Area	5%	5%
Average Slope	2.55%	1.57%
Design Flood	1:200 (RMF)	1:200 (RMF)
Overland Manning's factor		
Pervious fraction (n)	0.40	0.40
Impervious fraction (n)	0.13	0.13
Infiltration Settings		
Initial infiltration rate	60.0 mm/hr	60.0 mm/hr
Final infiltration rate	10.0 mm/hr	10.0 mm/hr

Table 15: Pipelines Peak Flows

Catchment	1	2
Return Periods (yrs.)	Q200	
Peak Flows (m^3/s)	11.31	13.81

Calculations were carried out using the computer programme *Storm and Sanitary Analysis* (SSA), which is an Autodesk product and compatible with Civil3D design and drafting software. Results are included in Appendix C. The output was compared to the results obtained in the conceptual designs thus far, by others, and found to be in the same order.

From the peak flows calculated, various sizes of pipe and channel could be tested with the Manning's Equation. Through an iterative process in Excel, a suitable pipe size could be chosen. The required size of the western leg pipeline was found to be a 1,5m diameter pipe. The last, combined portion was required to be 2,5m diameter pipe. The required size of the eastern leg pipeline was found to be a 2,0m diameter, based on a 1:200 design storm. Physical pipe properties discussed in Table 16 and in Section 15.6 below.

Table 16: Pipelines Flow Parameters

Pipe Type	Concrete pipes - SABS 677
Minimum Pipe Class	100D
Pipe Manning Factor	0.013
Minimum Flow Velocity	0.7m/s
Maximum Flow Velocity	4.5m/s
Freeboard	At least 10%

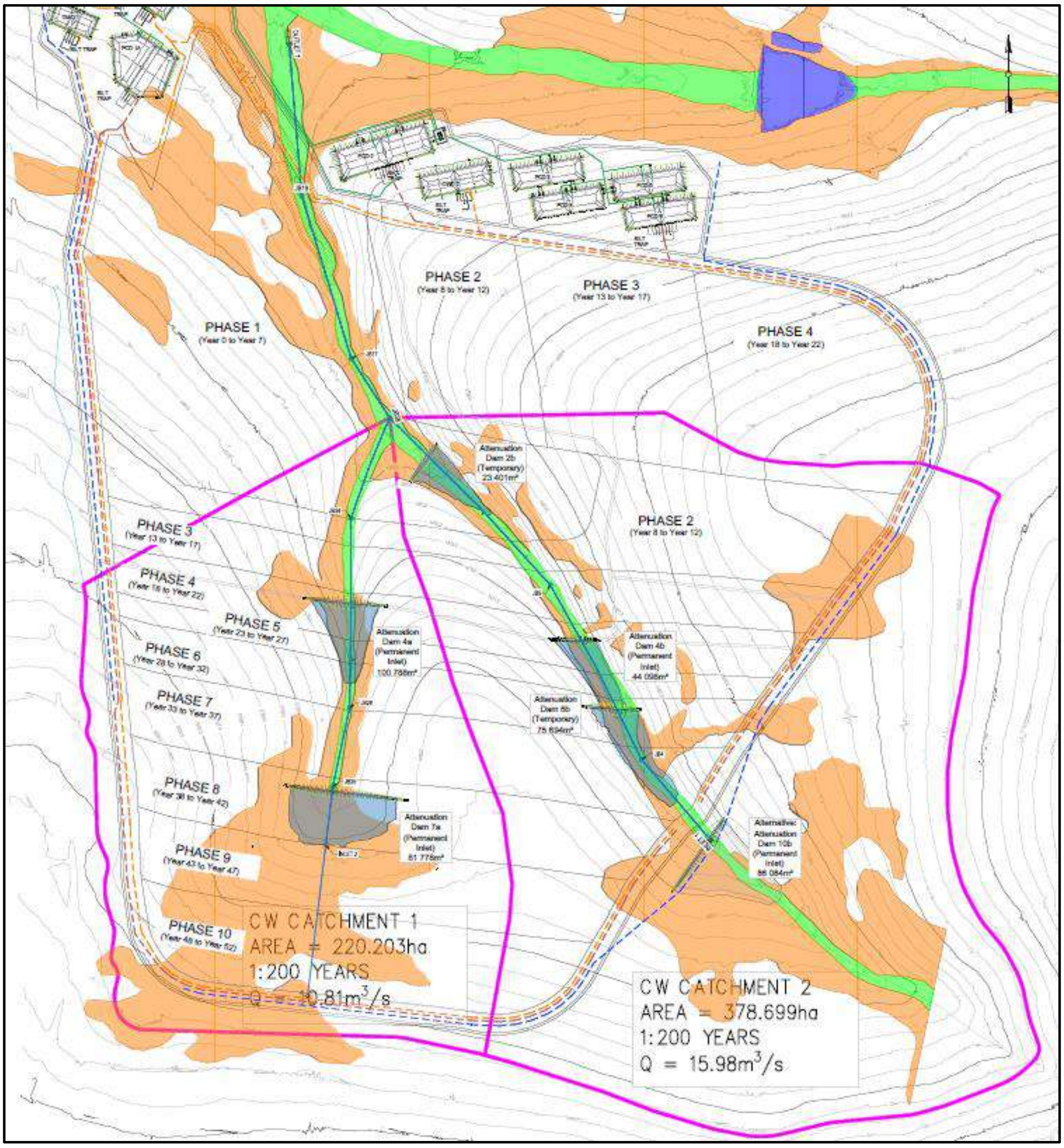


Figure 25: Pipeline Catchments

15.6 Pipe Description

It is proposed that the concrete pipe be lined in the lower half with an HDPE cast-in sheet. There are suitable products specifically manufactured with anchor knobs on one side, which act as lugs when cast into a concrete surface. The lining would protect the pipe but still allow visual inspections of the pipe soffit, which is where stress cracking and spalling would be expected to show first, should damage occur. Joints between the pipes and their embedded lining would then be extrusion welded with an HDPE flap briding the unlined concrete joint area to ensure a continuous HDPE lined lower portion.

Pipe loading calculations are based on 150 kN. A draft specification on the pipe is that of jacking pipes from Rocla or similar approved by the Engineer. Pipe section diameters required are 1.8m, 2m and 2.5m. It has been decided to optimise, for ease of access to 2m and 2,5m diameter.

Jacking type pipe sections have been proposed, of 100D strength. Other manufacturers are to be researched and considered as well.

Table 17: Pipelines Length and Diameter

Pipe Diameter	Concrete pipes - SABS 677
Tributary 2m diameter	1760.42m
Holfonteinspruit 2m diameter	1682.31m
TOTAL LENGTH 2m DIAMETER	**3442.73m**
Holfonteinspruit lower, 2,5m diameter	1380.16m

15.7 Pipe Loading

The most critical loading is generally found to be during installation – both during placement and during compaction of fill next to and on top of the pipe. Once compacted fill is in place it serves as protection to the pipe. Excellent site supervision is required at this time. Long term loading of the ADF will be dispersed throughout the pipeline and aurrounding natural ground at a loading angle of 35-45 degrees. The inclusion of a geo-grid reinforced soil mattress above the pipeline will allow for additional protection and load distribution throughout the pipeline.

15.8 Stormwater Design Methods and Software

A combination of Autodesk Civil3D models, Storm and Sanitary Analysis (SSA), and Excel based spreasheets for Rational Formula and Manning's Equation have been used, with cross-checking between the various manual and automated processes. Hydrocube, a catchment analysis programme has been used as well for pipe and channel size confirmation as well as verifying the performance of chosen pipes.

Most hydrologic models are based on the EPA's Storm Water Management Model (SWMM) model. SWMM is a proven model design and analyzing urban and also rural drainage systems. SSA has a built-in capability to run a SWMM model, as well as SCS TR-55, HEC-1, Rational method, and other hydrologic models. SSA is uniquely suited for land development design due to its integration with Civil 3D.

The Rational Method as described in the document Drainage Manual, Sixth Edition (2013) as published by SANRAL (South African National Roads Agency Limited) was used to determine the magnitude of the 1 in 50 year and 1 in 200 year flows.

The Rational Method is still probably the most commonly used method of estimating the peak runoff value of stormwater runooff generated from urban and rural areas where areas do not exceed 25km^2. Larger area can be considered, with adjustments.

The formula used in this method is

$$\mathbf{Q= f_t \times C \times I \times A/360\ m^3/s}$$

Q= the maximum /peak rate of runoff in m^3/s

F_t= and adjustment factor for the recurrence interval storm considered

C= runoff co-efficient per applicable tables

I= the rainfall Intensity(mm/hr)

A= area of catchment in hectares

The runoff co-efficient is a factor ranging from 0 to 1 which compensates for variations in rainfall over the catchment, infiltration and overland flow velocity during a storm. C can be determined from the Table Method used by the DWAF for consistency of approach.

The initial input was the 0.5 m interval contours that were obtained from the lidar survey. The software Autodesk Civil3D was used to extract cross sections of the watercourse at intervals. The surface and longitudinal information were imported into the software "SSA" automatically and calculations were processed. The output generally provides alternative methodolgy's results, for comparison. Longitudonal sections are produced from the Civil3D model for channels and pipelines. 'Before' and 'after' models can be compared, and earthworks cut and fill volumes calculated.

For sizing of pipelines and channels, manning's formula below was used.

$$Q = VA = \left(\frac{1.49}{n}\right) AR^{\frac{2}{3}} \sqrt{S} \quad [U.S.]$$

$$Q = VA = \left(\frac{1.00}{n}\right) AR^{\frac{2}{3}} \sqrt{S} \quad [SI]$$

Q = Flow (m^3/s)
V = Velocity (m/s)
A = Cross sectionalm Area (m^2)
R = Hydraulic Radius
S = Channel or Pipe Slope (m/m)
n = Manning's Roughness Coefficient

Further Stormwater Management information can calculations can be found in **Appendix C**.

16. POLLUTION CONTROL DAMS

16.1 General Arrangement

The general slope of the ADF site is to the north. It should be noted that Phase 1 of the ash disposal and related infrastructure is located down-slope of all later phases, and therefore Phase 1 design, including the stream diversion pipelines, must allow for the required final-stage capacity of channels, pipelines and CWDs / PCD's. Phase 1 of the ADF operations is expected to last seven years and includes four PCDs and two CWDs.

Refering to the PCD and CWD General Arrangement Drawing 366-511850, the Road PCD is located between the conveyor bridges on the north bank of the Klipfontein Spruit. PCD 1 is located between the top and bottom extendible conveyors and CWD 1 is located west of the bottom extendible conveyor. Figure 14 shows the layout of PCD 1 and CWD 1. Both dams respectively service the contaminated and clean water runoff that is generated on the western side slope of the proposed ADF.

PCDs 3, 4, 5 and 6 are located east of the top extendible conveyor and will be constructed in future phases.

The catchment for PCD 2 is the northern face of the ADF and the dam also receives overflow from PCDs 3 to 6. The catchment for PCD 6 is the upper eastern face of the ADF. The dam, once full, will overflow into the downstream dam, PCD 5, which will in turn overflow into the downstream dam, PCD 4, which will in turn overflow into the downstream dam, PCD 3. PCD 3 will then overflow into PCD 2. A water transfer provision has been added to PCD 2 to enable transferring of water from PCD 2 to PCD 6. The provision ensures that PCD 2 does not spill whilst there is still capacity in dams PCD 3 to PCD 6.

PCD 2 has an abstraction pipeline which is a suction pipeline to Pump Station No. 2. PCDs 3 to 6 have separate abstraction pipelines which feed the inlet to PCD 2 via open channels. These abstraction pipelines and channels are intended to be used only for maintenance purposes of the PCDs or to supply water to PCD 2 if there is no water in PCD 2 for dust suppression.

CWD 2 similarly services the clean water runoff that is generated on the northern and upper eastern side slope of the proposed ADF.

16.2 Clean and contaminated water management

A system of three open drains is proposed for the ADF surface water drainage, namely a clean outer drain discharging to the environment, a middle dirty drain for polluted water, and an inner "dirty-to-clean" drain. Clean and dirty ash areas' runoff is directed via the channels to a complex of PCDs and CWDs. With each phase the perimeter drains to the east and west of the ash dump will be extended.

Dirty channels from the ADF drain to PCDs, as well as the contaminated runoff from the conveyor corridors.

Clean runoff from clean / rehabilitated areas drains via channels to the clean water dams. As stages of the ADF are rehabilitated, toe drains become "clean" drains, hence the term "dirty-to-clean" channel. Similarly, PCD-1b in time will become a clean water dam that receives clean runoff water.

16.3 Constraints and Geotechnical Considerations

- Due to possible saturated nature of the sub-soil and the proximity of the water course, precautions must be taken during excavation by shoring or benching the excavations /box cuts.
- Permanent sub-surface drainage must be installed to make sure that all the water seepage will be diverted from below the dam basins, and to prevent possible uplift of the geomembrane liner, and concrete protection / ballast layer.
- Fin-drains 2,5 metres to 3 metres deep are to be installed upslope of the dams to divert as much up-slope groundwater as possible.
- The installation of the barrier system and sub-soil drains should preferably take place in mid-to-late dry season.
- The management of silt, generally ash, is a considerable problem at the ADF. Silt traps are to be installed at each dam. The maintenance of silt traps, and clearing of channels as well, should be prioritised during operations. To facilitate silt removal from the dams themselves, a ramp is placed in the corner of each compartment for a small dozer or TLB. A concrete protection layer is to be installed over the base and sides of the dam.

- The in-situ material does not qualify as suitable fill material for the dams' berms (walls). Imported fill material of minimum G6 grade will be required.

17. DAMS COMPLEX TECHNICAL DESIGN

17.1 General Arrangements of Dams

The Phase 1 dams are mainly clustered in two areas, as can be seen in Figure below, an extract from Drawing 366-511845 Phase 1 General Layout. PCD-1A, PCD-1B and CWD-1 are situated north-west of the ADF. PCD-2 and CWD-2 are situated north-east of the ADF, receiving the water from channels on the eastern toe of the ADF. Pump Stations 1 (west) and 2 (east) are indicated as well.

As can be seen, each dam is divided into two compartments. This allows the dam to remain functional when one half is out of service, for repair or maintenance, for example.

Each compartment of the PCDs has a dedicated silt trap, i.e. two silt traps per dam. The CWDs only require one silt trap each that would serve both compartments of the dam.

Future dams will be added east of CWD-2 in following phases.

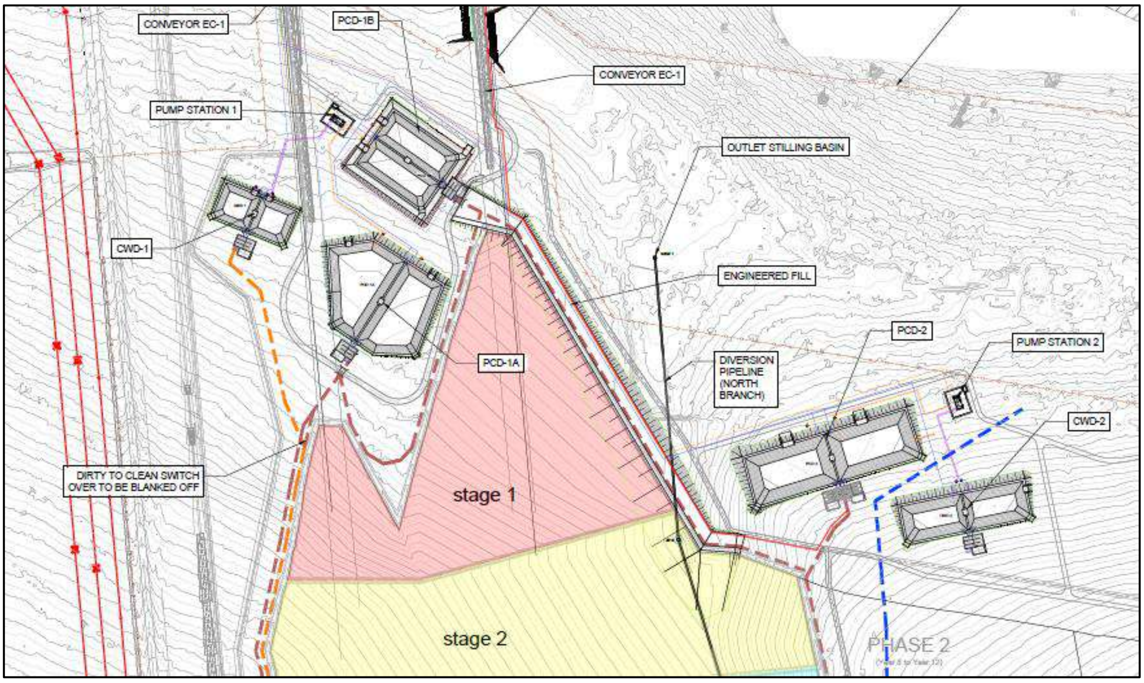


Figure 26: Layout of Dams North of ADF

The Road Dam lies further north, between the conveyors, receiving run-off from the conveyor and road crossing bridges. The Road Dam location is shown in **Figure 27** below.

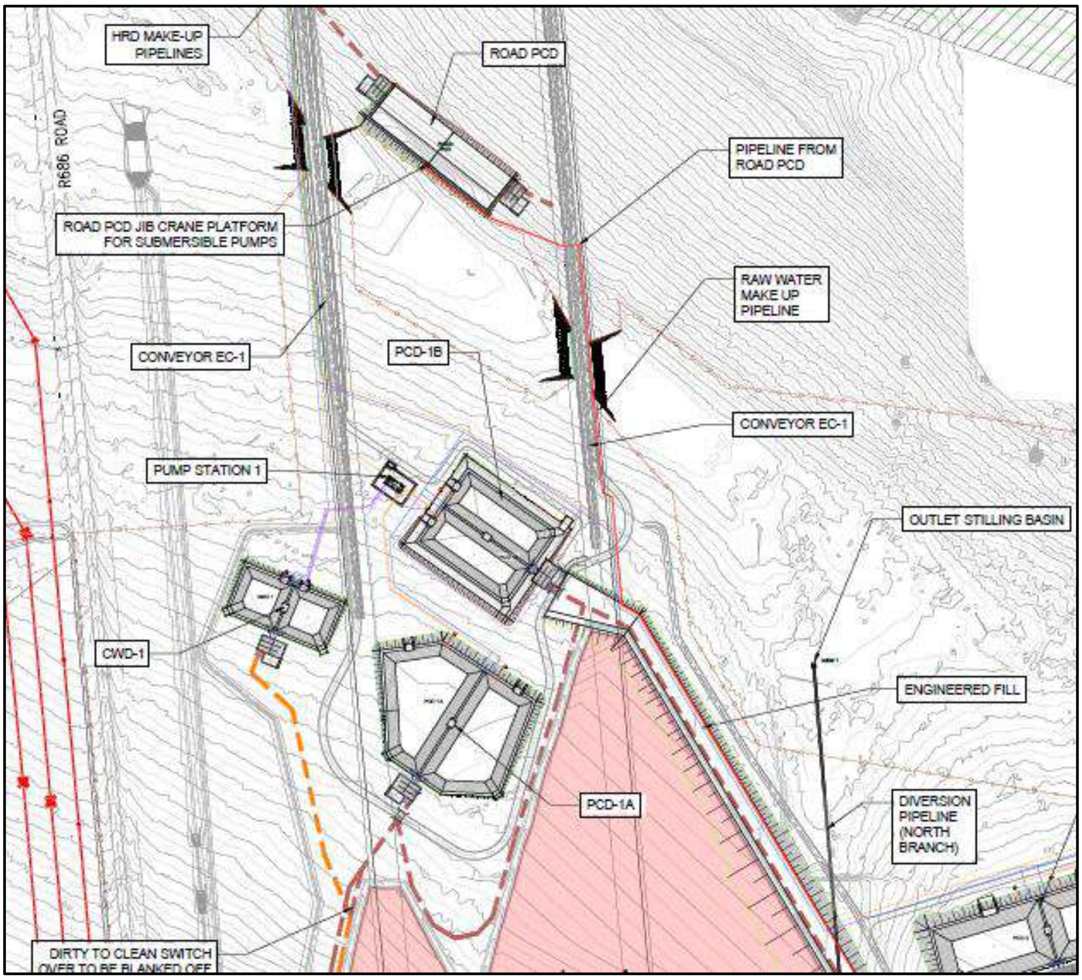


Figure 27: Road Dam Location

17.2 Dam sizing

Calculations were carried out using the computer programme *Storm and Sanitary Analysis* (SSA), which is an Autodesk product and compatible with Civil3D design and drafting software. Catchments reporting to each channel were analysed by the Rational method. Results are included in Appendix C. A 1:50 year storm of 24-hour duration was considered in the sizing of the dam. Operational requirements also influenced the dam sizing.

A Water Balance will be reported on separately, including stormwater input and water output from the dams for dust suppression. Water levels will be managed between dams via the pump-stations and piped connections.

17.3 Dam Inlets

Both PCDs and CWDs inlets will be from a silt trap. A system of sluice gates at the silt trap outlets will dictate to which dam compartment the water will flow.

17.4 Dam outlets

All dams will have an emergency overflow spillway, and a freeboard of 800mm above full supply level. An internal spillway between dam compartments will be lower than the external spillways. At a height of 1m off the dam floor will be a HDPE outlet pipe of 600mm diameter, with a valve on the outside of the dam, usually on the northern (downslope) side. Each compartment will have a spillway and outlet pipe.

17.5 Battery Limit

The battery limit for the civil and lining contractor will be the downstream flange of the outlet valve. The lining contractor will install a HDPE pipe boot sleeve to house the penetration through dam wall, which will be welded to the dam geomembrane liner. Details are shown on the drawing 366-511878.

17.6 Dam Classification

The two main aspects that must be considered for the classification of the dam are as follows:

- The size of the dam wall based on Table 1 in **Figure 28** as well as the storage capacity of the dam; and
- The hazard potential of the water stored in the dam.

Table 18 below presents the summary of dam properties for the Phase 1 Kusile 60 year ADF pollution control and clean water dams.

Table 18 - Summary of Dam Properties

	No	Dam Wall Height (m)	Volume (m^3)	DWA Classification	Dam Safety Approval Required	Water level (MSL)	
						Min	Max
Pollution Control Dams (PCD's)	PCD 1A	4.48	102 284	Category II	-	1 452.14	1457.7
	PCD 1B	1.15	42 875	Category II	-	1 447.016	1451.5
	PCD 2	**7.30**	**113 471**	Category II	Yes	1 452.853	1460.2
	PCD 3 (*)	5.33	48 657	Category II	-	1462.368	1468.4
	PCD 4 (*)	4.70	48 734	Category II	-	1467.268	1473.3
	PCD 5 (*)	6.47	48 706	Category II	-	1467.968	1474.6
	PCD 6 (*)	5.38	48 790	Category II	-	1472.168	1478.2
Clean Water Dams (CWD)	CWD 1	2.65	18 555	Category I	-	1 451.36	1455.1
	CWD 2	4.90	48 917	Category I	-	1 460.87	1466.9
Road Dam (PCD) North of Klipfonteinspruit		5.30	24 000	Category II	-	1 439.50	1 444.50

(*) : Future phases

In accordance with the DWA Dam classication tables, the size classification of all dams is a “Small dam”. Taking the potentially contaminated nature of the dam water, and the economic loss to infrastructure should the dam fail could be significant, hazard potential is considered “significant” for all PCD’s while CWD’s are considered “low”.

The tables below are referred to for the classification of the dams as per the National Water Act, 1998 (Act 36 of 1998).

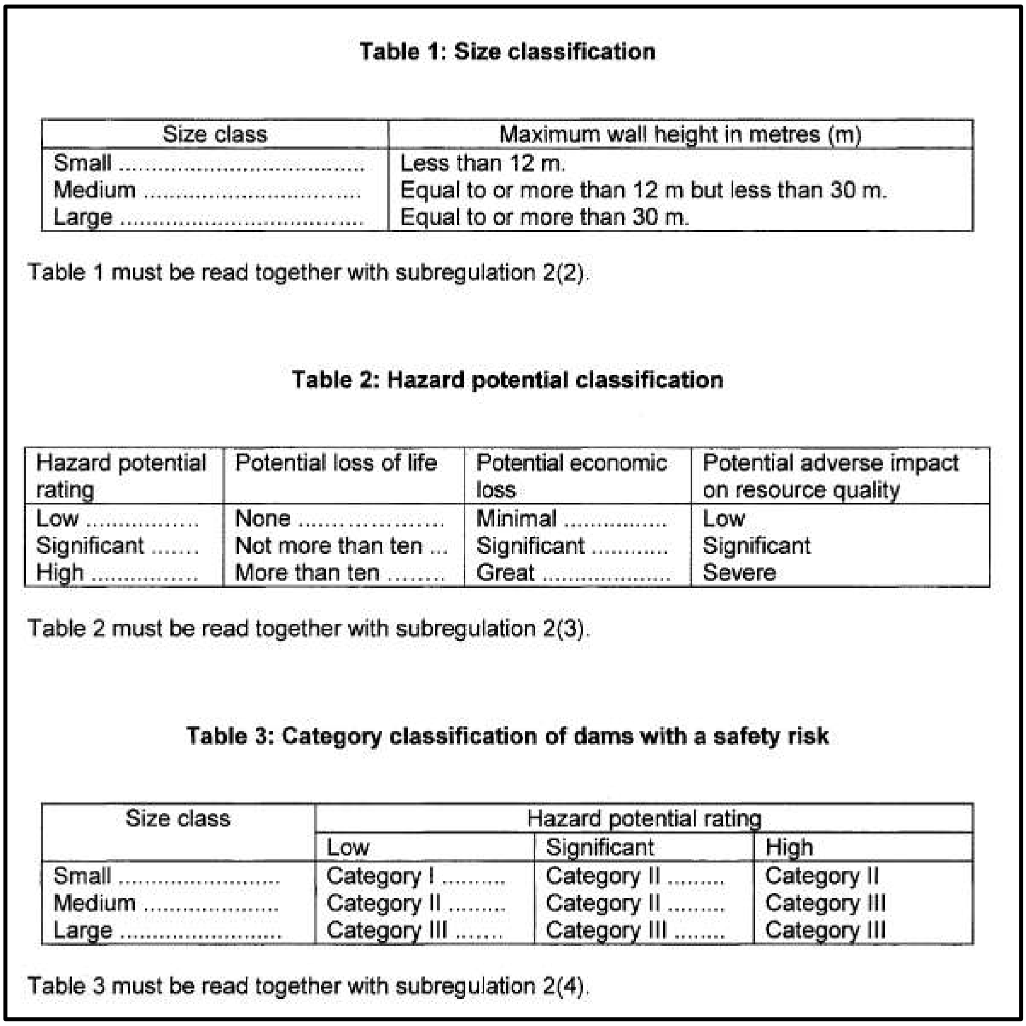


Table 1: Size classification

Size class	Maximum wall height in metres (m)
Small	Less than 12 m.
Medium	Equal to or more than 12 m but less than 30 m.
Large	Equal to or more than 30 m.

Table 1 must be read together with subregulation 2(2).

Table 2: Hazard potential classification

Hazard potential rating	Potential loss of life	Potential economic loss	Potential adverse impact on resource quality
Low	None	Minimal	Low
Significant	Not more than ten ...	Significant	Significant
High	More than ten	Great	Severe

Table 2 must be read together with subregulation 2(3).

Table 3: Category 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 III	Category III	Category III

Table 3 must be read together with subregulation 2(4).

Figure 28: Dam Classification (DWA)

The **dam safety legislation** is covered by chapter 12 of the National Water Act, 1998 (Act 36 of 1998) [NWA] and by dam safety regulations, published in Government Notice R. 139 of 24 February 2012. Only dams with a safety risk (that is dams with a maximum wall height exceeding 5,0 m **and** with a storage capacity exceeding 50 000 m3, or any other dam declared by the Minister as a dam with a safety risk) are subject to these Regulations.

The pollution control dam for the Kusile 60 year ADF project that meets this dam safety requirement above is PCD 2.

17.7 Polluting Contaminants

A waste classification study was undertaken by Jones and Wagener Consultants in January 2014, Report no. JW030/13/D121- Rev3. The **Type 3** waste classification was the result of the LC value of Boron exceeding the value of 0,5mg/l and the TC value of barium and fluoride exceeding their respective TC0 values.Principal contaminants may be associated with:

- Metals/metalloids: arsenic, cadmium, chromium (total and hexavalent), iron, lead, manganese, mercury, nickel, PGMs, silver, vanadium and zinc; and,
- Inorganics: acids/bases

17.8 Waste Class for Barrier Design

In view of the contaminants to be expected on site, as described above, a barrier liner will be required. Although the design is not for a landfill but for a PCD, barrier design requirements are to take into account the National Norms and Standards for Disposal of Waste to Landfill (R636 of August 2013). The PCD will receive "**Type 3**" wastes and therefore requires a "**Class C**" containment barrier system. Both PCDs and CWDs will have the same barrier specification. A typical Class C barrier system for Type 3 waste is shown below:

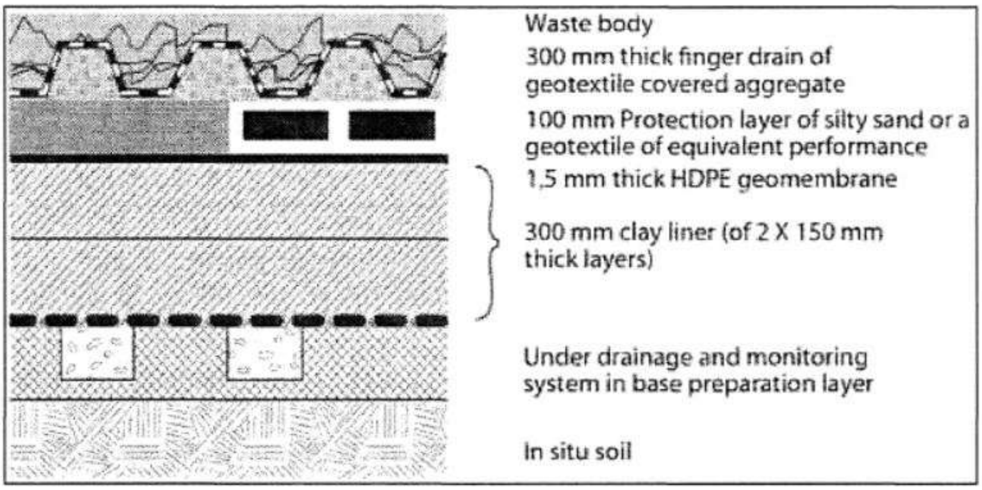
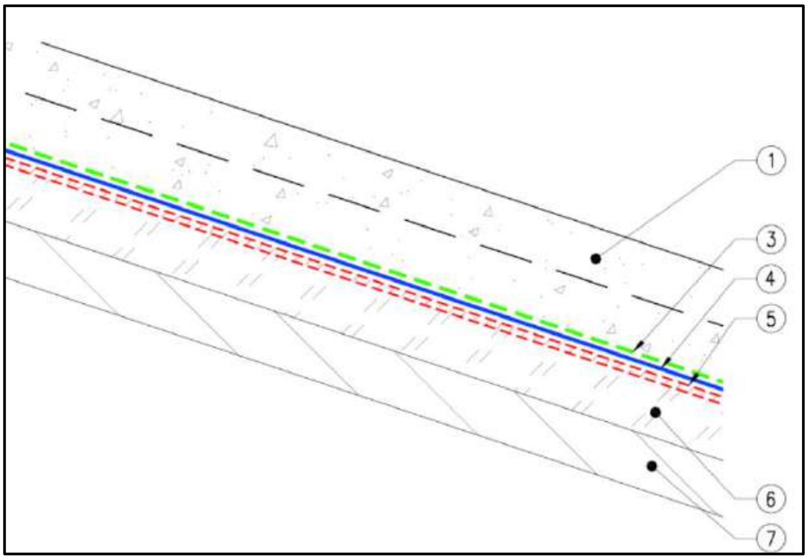


Figure 29: Norms and Standards Class C Barrier

17.9 Proposed Site-Specific Barrier Design

Taking into account site specific factors and the fact that the containment facility is a dam rather than a dry landfill, the proposed liner design is described below. An upper stone leachate collection system (referred to as finger drains in **Figure 17** above) does not apply to a dam and will be replaced with a ballast and protection layer, which will be a mesh reinforced 35 MPa concrete layer on the base and lower side slopes. Concrete strength on upper slopes to be 25 MPa unreinforced.



LINER SPECIFICATION	
LAYER	DESCRIPTION
1	300mm THICK 35MPa CONCRETE SLAB ON BASE AND UP TO 1.0m HEIGHT ON SIDE SLOPES, REINFORCED WITH ONE LAYER MESH REF. 395
2	300mm THICK 25MPa CONCRETE ON SIDE SLOPES ABOVE 1.0m HEIGHT
3	1000g/m² PROTECTION GEOTEXTILE TO GRI- GT12 SPECFICATION
4	2mm MONO-TEXTURED GEOMEMBRANE TO GRI-GM3 SPECIFICATION, WITH TEXTURE FACING DOWNWARDS
5	3,7kg/m² GCL TO GRI-GCL 3 SPECIFICATION
6	150mm LAYER OF CLAYEY SOIL COMPACTED TO 100% MOD PROCTOR AT MC 0% TO +2%, OR 93% MOD AASHTO TO SUIT SOIL TYPE
7	BASE PREPARATION LAYER: RIP AND RE-COMPACT TO 100% MOD PROCTOR AT MC 0% TO +2%, OR 93% MOD AASHTO TO SUIT SOIL TYPE
8	SELECTED FILL MATERIAL COMPACTED IN LAYERS NOT EXCEEDING 150mm THICK TO 95% MOD AASHTO
9	INCOMPETENT MATERIAL TO BE REPLACED TO A DEPTH OF 600mm WHERE DIRECTED BY ENGINEER ON SITE, COMPACTED IN LAYERS NOT EXCEEDING 150mm THICK TO 95% MOD AASHTO

Figure 30: Proposed Dams Barrier, Class C

A 1000g/m^2 protection geotextile is proposed over the geomembrane instead of 100mm sand. The proposed barrier system replaces 300mm compacted clay layers with 1 x 150mm layer of clayey material in addition to a geosynthetic clay layer (GCL).

Note that compacted natural clay layers have been replaced with a (GCL) due to the lack of sufficient suitable clay on site. The in-situ clayey material permeability is not consistently low enough to qualify as a clay barrier layer. In addition to this, the surface areas to be lined at each stage are large areas which become extremely difficult for a contractor to maintain at optimum moisture content prior to placement of the geomembrane.

Under-drains (sub-soil drains) will drain to a manhole, where monitoring can take place, thus functioning as both a sub-soil drainage and leakage detection system. Clean sub-soil water will be pumped out to the environment.

The proposed barrier containment system in **Figure 30** above are as follows:

- Concrete Protection Layer – 300mm thick concrete cast in alternate panels. The concrete on the ramp, base and up to a vertical height of 1m on the slopes will be 35 MPa mesh-reinforced. The remainder of the slopes will be 25 MPa concrete.
- Geotextile Protection Layer – needle punched non-woven geotextile layer of nominal mass 1000 g/m^2, to GRI-GT12(a) standard.
- Primary Liner – 2mm mono-textured HDPE geomembrane liner to GRI – GM13.
- Geosynthetic Clay Liner– 3700g/m2 Geosynthetic Clay Liner conforming to GRI-GCL 3 specifications.
- Compacted Clay Layer – 150mm thick layer of compacted clayey soil layer (no particles >10mm compacted to 100% standard proctor at OMC to ±2.0% / 93% MOD AASHTO – Only required a levelling layer where a GCL liner cannot be placed directly onto in-situ material.
- Basal Preparation – Rip and recompact 150mm thick layer of in-situ material to 95% MOD AASHTO. Incompetent material to be replaced with 600mm G6 material compacted in layers not exceeding 150mm to 95% MOD AASHTO.

The liner system design makes provision for the collection and drainage of leakage detection and collection under the liner in sub-soil drains which drain to monitoring manholes where testing can occur or extraction and over pumping back into the dams or to the CWD’s.

17.10 Operating Period and Polluting Period of PCD Facility

The planned operating period of the facility is 60 years. Assuming a polluting period of 100 years post closure, the polluting period of the facility is 160 years, including the operating phase. The service life of the containment materials is addressed in the CQA plan appended to the final ADF Design Report.

17.11 Dam Embankments

Imported fill of G5 quality or above will have to be imported for the wall embankments, to achieve a compaction of 98% Mod AASHTO. It is proposed that the walls on the lower (downslope) sides of the dams, will have a keyed-in trench backfilled with the same material used for the berms, where berm heights exceed 5m. Fill material is to be placed in layers not exceeding 150mm thick, and compacted. Walls are to be constructed at 1:3 (vertical: horizontal) side slopes both internally and externally.

17.12 Slope stability analysis

A stable slope is anticipated as the FoS calculated for overall failure is greater than 1.3 for the given conditions. However, the following is required to confirm parameters and site conditions more accurately.

- Strength tests of the embankment fill must be confirmed during construction stage based on the imported materials finally used in the embankment.
- The phreatic level within the PCD embankment wall must be monitored closely with the performance of the under drains and the integrity of the liner system.
- Construction of the embankment must be carried out to the design specifications.
- The slope geometry must be confirmed following the construction phase.
- The analyses should be repeated with refined data.
- Actual leakage rates of the dams can have an effect on the embankment stability and should be monitored continuously.

Ongoing monitoring and surveillance must be maintained with the above data taking into consideration of all the relevant operational data.

The perimeter berm height above ground varies, but will rise at least 1m high, to prevent outside surface water and silt flowing over the crest on the upstream side of the dams. The maximum slope of the dam's outer slopes will be 1 vertical: 3 horizontal. The outer berm on the northern side of PCD 1B may be covered by riprap rock to provide scour protection from possible water in the Vlei area (due to its close proximity to the river). The exact extent of cover will be determined by the Engineer on site.

The internal slopes will be 1 vertical : 3 horizontal with the perimeter wall crest will be 4m wide. An anchor trench for the dam lining components will be excavated into the crest and backfilled once the lining layers have been installed.

17.13 Veneer Stability Analysis

Previous veneer stability analyses were carried out on landfill cells, leachate and pollution control dam liner system in accordance with the methodology of Soong and Koerner (2005). This methodology is described in "Designing with Geosynthetics, 6th Edition, by Robert M. Koerner.

These previous veneer stability calculations considered the weakest or most critical failure surface (critical interface) where sliding is likely to occur. For a dam, however, the risk of veneer failure is significantly lower than a waste cell, and veneer stability calculations are therefore not relevant. (Note that there are no permanent down-drag forces on the slopes, and slopes are generally shorter.)

Any ingress of leakage under the liner would be drained by the leakage detection system, preventing saturation within the liner layers. The leakage detection system would be monitored, and any problems found would be rectified.

Stability of the perimeter berms will be ensured by strict compaction testing during construction (in accordance with the CQA) as well as strict CQA monitoring during the lining process.

17.14 Compatibility of Liner Materials with the Waste Stream

Geotextiles, both woven and non-woven have been found to be compatible with ash leachate historically, provided that the geotextiles are not left exposed to ultraviolet light. The geotextiles installed must comply with GRI-GT12 and GRI GT13 as applicable with additional specifications as detailed in the report by GMJ Consulting which can be found in Appendix C.

Testing has been undertaken with site-specific ash (Kusile Power Station Ash) and bentonite powder taken from a commonly used, locally produced GCL. These tests would need to be re-done once a contractor is appointed for the construction, and their lining supplier is made known, to ensure the compatibility of the proposed materials.

The recent test results, shown in Section 12.6, indicate that the site-specific ash's permeant solution (leachate) did not restrict the bentonite powder swell. This indicates that a GCL is suitable for use in the barrier system for the PCD's and CWD's.

Additionally, similar swell testing was undertaken with site specific soil permeant and bentonite powder. The results indicate that the site-specific soils do not restrict the bentonite powder swell and a GCL is fully compatible for the lining of this 60yr ADF Facility.

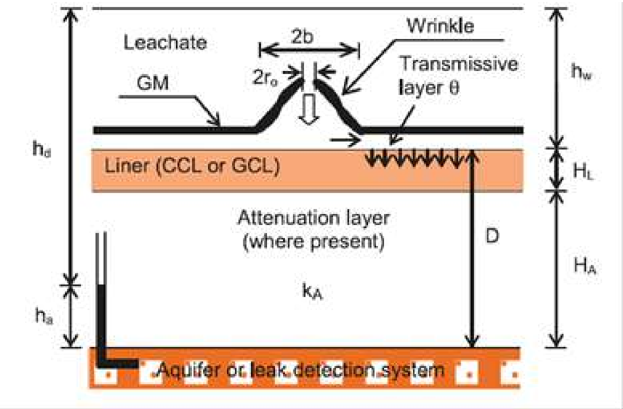
17.15 Composite Liner Leakage Rates

The leakage through liners is usually interpreted in terms of the actual advective flow through the liner. The combination of a geomembrane and GCL substantially reduces leakage relative to a geomembrane alone and in addition geomembrane/GCL composite liners can be constructed to give lower leakage than a geomembrane/CCL composite (Rowe 2005).

Various methodologies may be used to calculate leakage rates i.e. Giroud and Bonaparte (1989), Rowe and Booker (1998), Rowe (2005) and Rowe (2012) however, all methodologies have various variables, assumptions and specific conditions (site and laboratory).

Historic studies have also shown a significant discrepancy between theoretical and measured leakage rates. The major variables to consider are the size of the hole/s, the hydraulic conductivity of the CCL/GCL, the head difference across the liner, the transmissivity of the interface between the geomembrane and GCL/CCL (dependent on good or poor contact) and CQA. Using Rowe (1998), Rowe (2005) and Rowe (2012), the following equation was used to estimate the potential leakage rate within the PCD's and CWD's:

$$[6] \qquad Q = 2L[kb + (kD\theta)^{0.5}]h_d/D$$ **(Rowe, 2012)**



Schematic showing leakage through a wrinkle of length L and width 2b with a hole of radius r_o (adapted from Rowe 1998)

The estimated leakage rate based on the PCD liner system, with stringent CQA and assuming 5 No. wrinkles with holes per hectare, equals to approximately **212 lphd** outside a wrinkle and 683 lphd beneath a wrinkle. (Please refer to the attached estimated potential flow rate calculation spreadsheet in Appendix C). This leakage rate is in-line with the norms as indicated and documented theoretically for a GCL. This will be further monitored post construction via the leachate leak detection system / sub-soil drainage system.

17.16 Service Life of a Liner System

To ensure a long service life of a HDPE geomembrane exposed to contaminated water it is necessary to limit the tensile strains/ stresses in the geomembrane to an acceptable low level (Rowe, R.K. and Yu, Y., 2018). In addition, the service life of a containment barrier system is highly dependent on the exposure temperatures on the liner system and more specifically on the HDPE geomembrane. The service life of the HDPE geomembrane is normally evaluated using a three-stage degradation model which consists of Stage A (depletion of antioxidants through volatilization, diffusion or oxidation), Stage B (induction or start of polymer degradation) and Stage C (polymer degradation and decrease in key physical properties). The boundaries between these stages are not so distinct in practice, and the end of Stages B and C may vary depending on the parameters being considered, e.g. tensile break strength, tensile break strain and stress crack resistance.

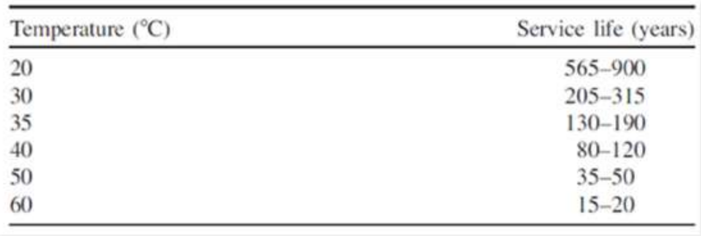
Because stress cracking is the mode of final failure, stress cracking is the most appropriate determinant of end of life when data is available, as stated by Jafari, N.H., Stark, T.D. and Rowe, R.K. (2014). “Service Life of HDPE Geomembranes Subjected to Elevated Temperatures.” Journal of Hazardous, Toxic and Radioactive Waste.

17.17 Anticipated Leachate Temperature Range and Estimated Service Life

The heat generated by a waste containment facility is a function of type of waste, management practice and the nature of the waste degradation process. Considering that the contaminated runoff is not expected to contain much organic material, and the fact that the water is impounded in sumps and silt traps en-route to the PCD’s and CWD’s, heat is not expected to be generated within the Dams. Direct sunlight on the black HDPE geomembrane, being the predominant heating factor, is prevented by the concrete protection layer.

The temperature of the geomembrane is expected to be below ambient soil temperatures and fairly constant, estimated to be around 15 to 22 degrees Celsius. From the table below, the service life is therefore expected to be at least 500 years.

Table 19 - Estimated HDPE Geomembrane Service Life Based on 50% Reduction in Tensile Strength at Break for Different temperatures (Based on Rowe (2005))



Temperature (°C)	Service life (years)
20	565–900
30	205–315
35	130–190
40	80–120
50	35–50
60	15–20

17.18 Tensile strains in HDPE Geomembrane

The two key related tensile strains to be considered in a GMB in the landfill cells are the following:

- local indentations of GMBs induced by the overlying drainage materials, and
- down-drag load for GMBs on side slopes.

As the PCD will not have any ballast drainage material and consists of considerably shorter side slopes, tensile strain within the leachate dam lining system is negligible.

As noted from previous projects, the expansion and contraction of the concrete panel protection layer is of concern as it may induce unnecessary strains on the geomembrane liner. In order to ensure that this expansion and contraction is negligible a mono-textured geomembrane has been specified with the texturing placed downwards (onto the GCL). This will ensure that the critical interface remains between the protection GT and the GM (smooth face) and not the GCL / GM interface which can result in induced strain and even the pull out of the lining, particularly during concrete placement.

The PCD protection layer consists of a layer of mesh reinforced 25 MPa (side slopes) / 35MPa (base) mass concrete on a 1000g/m^2 protection geotextile. The methodology for concrete casting would entail overhead pouring, with even less chance of construction damage / loading when compared to ballast stones. Furthermore, the concrete will also protect the liner from theft, vandalism and damage while further aiding in ballasting / weighing down the liner from possible uplift from sub-soil flows (which are not anticipated due to the intricate sub-soil drainage system beneath each dam).

Tensile strains due to down-drag on side slopes

Based on tensile peak strain test results from previous projects and from the above discussion we are satisfied that the GMB tensile strains will be within the 3% limit.

17.19 Service life of other materials

Other materials in the landfill to be considered in terms of Service Life include the following:

- drainage system pipes,
- drainage stone.

The leakage detection drainage pipes will be solid HDPE pipes. The pipes are to be drilled with 15mm diameter holes in the upper third of the pipe circumference. It is estimated that the service life of HDPE pipes is approximately 200 years in similar landfill usage. In addition to material strength, the pipe diameter should be sufficient- a minimum of 110mm diameter so that biological clogging would not occur. Clogging could take place within the stone drainage layer as well. Clogging, both chemical and biological occurs more on hazardous waste sites with high iron content in the soils. These conditions are not expected at the ADF site or dams complex site.

17.20 Proven Equivalent Performance of Alternative Elements

The alternative elements are the GCL layer in place of the standard's 600mm natural clay layers, and a protective Geotextile in the place of a silty sand layer. Testing will be done prior to construction to ensure that:

- The GCL should have a permeability of not more than 1 x 10-7 cm/s (0,3 m/year).
- The bentonite within the GCL has a satisfactory swell index with site soil and ash specific permeants.
- The geotextile should prevent strain on the geomembrane exceeding a total strain of 3%, tested by means of the protection efficiency test

17.21 Leakage Detection / Sub-soil drains

Beneath the barrier system, a leakage detection system of 110mm diameter perforated HDPE pipes will collect sub-soil drainage below the base of the dams. The pipes are laid in a shallow trench of minimum depth 400mm, surrounded by 19mm stone wrapped in 200g/m^2 separation geotextile, forming leakage detection "finger drains" within the sub-soils. The drains are spaced at about 10m centres. The floor of the PCD slopes towards the ramps, against the natural gradient, whereas the sub-soil drains drain diagonally with the natural gradient, therefore becoming deeper across the floor of the dam. The diagonal intercepting drains feed into a collector pipe of 160mm diameter, which will drain to a monitoring manhole.

Water collected in the manhole will be tested and pumped out to the environment or CWDs if clean, or back into the PCDs if contaminated.

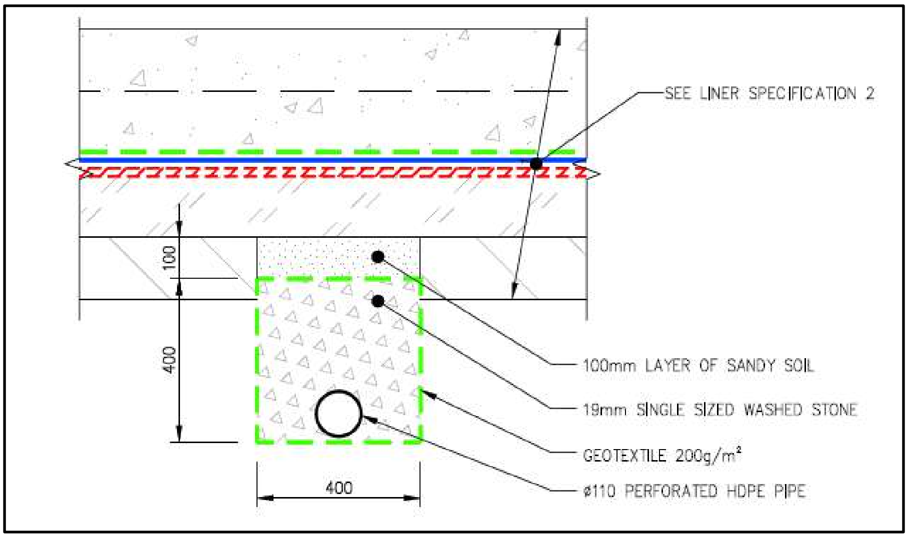


Figure 31: PCD Sub-soil / leakage detection drain

17.22 Pipe Specifications

The drainage pipes will be perforated, HDPE pipes manufactured according to SANS ISO 4427 and confirmed to withstand the external loading. The load bearing capacity of the HDPE pipes should have at least a factor of safety of 1,3. A ring stiffness greater than 450 KPa is specified. The feeder pipes are to be a 110mm diameter PE100 PN10 HDPE pipes, butt-welded. The collector pipes are to be 160mm diameter PE100 PN10.

The outlet pipe at each dam that penetrates the lining system, about 1m from the base level, is to be a 600mm diameter HDPE, PE100 PN10. The pipe penetration detail is shown below:

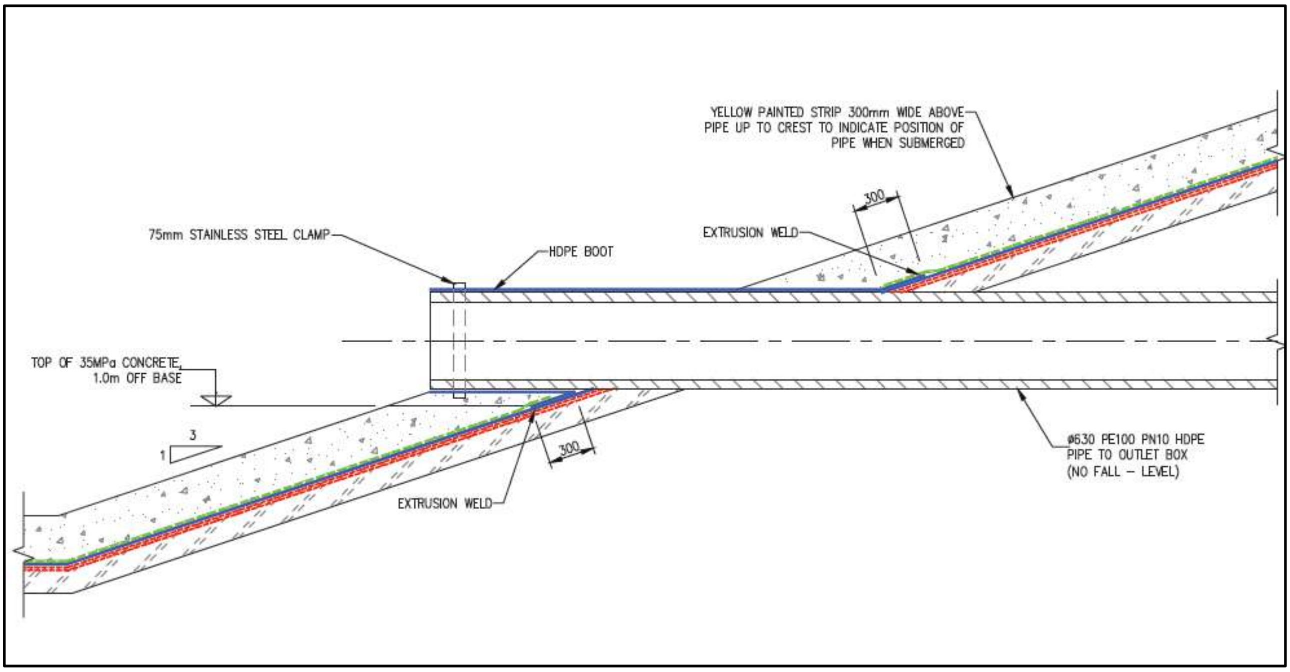


Figure 32:Pipe "top hat" penetration through dam wall, extract Drg 366-511878

17.23 Gas Management

The nature of the underlying soils is relatively permeable, and it is unlikely that natural gases will accumulate under the PCD barrier layer. Gases would vent through the sub-soil system and manholes with lid vent-holes.

17.24 Summary

A summary table of dam properties is shown below:

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Table 20: Summary of Dams Properties

	PCD 1A Compartment		PCD 1 B **Compartment**		PCD 2 **Compartment**		Road PCD **Compartment**		CWD 1 **Compartment**		CWD 2 **Compartment**	
	1	**2**	**1**	**2**	**1**	**2**	**1**	**2**	**1**	**2**	**1**	**2**
Crest Level (m)	1458.50	1458.50	1450.40	1450.40	1461.00	1461.00	1445.00	1445.00	1455.90	1455.90	1467.70	1467.70
Internal Spillway Level (between compartment) (m)	1457.50	1457.50	1449.40	1449.40	1460.00	1460.00	Variable		1454.90	1454.90	1466.70	1466.70
External Spillway Level (m)	1457.70	1457.70	1449.60	1449.60	1460.20	1460.20	1444.20	1444.20	1455.10	1455.10	1466.90	1466.90
Internal Base Level (m)	1452.14	1452.14	1445.12	1445.12	1452.85	1452.85	1438.58	1438.58	1451.36	1451.36	1460.87	1460.87
Internal Depth (m)	6.36	6.36	5.28	5.28	8.15	8.15	6.42	6.42	4.54	4.54	6.83	6.83
Capacity (m^3)	102285		42875		113471		24000		18555		48917	
Freeboard (mm)	800	800	800	800	800	800	800	800	800	800	800	800
Side slope	1:3	1:3	1:3	1:3	1:3	1:3	Vertical		1:3	1:3	1:3	1:3
Outlet Pipe Diameter (mm)	600	600	600	600	600	600	TBC		600	600	600	600
Invert Pipe Level (m)	1454.50	1454.50	1447.29	1447.29	1455.00	1455.00	TBC		1452.90	1452.90	1462.70	1462.70

18. WATER BALANCE

An integrated water balance model has been developed in the concept phase and was used to size the dams required for storm water management. There are four main pollution control dams, PCD 1A, PCD 1B, Road PCD and PCD 2, which by regulation are not allowed to spill to the environment more than once in a 50-year period. The rest of the dams are designed to overflow to the other dams within the system. The results of the water balance simulation indicate no spillages over the simulated period if transfer systems between all dams are incorporated under the conditions as defined in this report. The system is therefore in compliance with the requirements of the governing regulation, GN 704, for the control of storm water at the facility.

The system is however dependent on operational conditions in the form of water abstraction parameters, dam operating levels, water transfer capacity, etc. These parameters have been taken into consideration in the design process with Eskom's LPS CoE and they will be clearly defined in the Operations and Maintenance manual.

An updated water balance has been prepared based on final design elements and is included in the Operating Plan in **Appendix E**.

19. OPERATING PLAN

An Operating Plan has been drawn up for the ADF site. Due to the complexity of the water management at the dams complex, the Operating Plan for the dams complex and dust suppression might require a stand-alone document. Relevant aspects should be included in the over-arching Eskom Kusile Power Station Operations Manual/s.

The principle of the Operating Manual is that it is a live document, and continually updated to suit on-site experiences and changing conditions. The goal of the Operating Manual is to provide a reference resource for all necessary procedures and activities on site. Work methods are described to ensure consistency and safety. Monitoring, recording and reporting requirements are included. The Operating Manual therefore provides a training resource as well. The Operating Plan appended to this Design Report is the baseline for the Operating Manual that would ultimately be developed for each facility and that would include equipment-specific operating method statements, task specific PPE as well as training and qualifications requirements for operating personnel.

20. MONITORING PLAN

The standing Monitoring Plan for the Kusile Power Station should be updated to include for the monitoring requirements for the ADF. The Monitoring Plan is to be included in the Operating Manual. The activities to monitor include:

20.1 Disposal

Ash disposal monitoring includes side slopes profiles and staying within the current dedicated and lined footprint.

Monitoring instrumentation is to be set up prior to ash disposal. This would include liner temperature measurement and ash moisture content monitoring, in addition to settlement beacons.

20.2 Dust Suppression

Monitoring of dust suppression and dust outfalls should be on-going as per the existing methodologies. Records of water source, dust suppression spraying rates, duration and frequency should be recorded and monitored.

20.3 Condition of open drains

The perimeter open drains, both clean and dirty should be monitored in terms of their condition and whether repair or clearing is required.

20.4 Ongoing rehabilitation

Monitoring of topsoil availability should also include management of designated topsoil stockpiles in terms of preventing contamination, managing stormwater run-off and monitoring usage by means of physical survey and load records.

As a stage is completed, topsoil should be placed as per the capping design, as soon as possible, and watered to encourage capping vegetation. The condition of the capping should be monitored.

20.5 Monitoring of security, health and safety

Aspects such as fence security patrols, operational staff's health and safety requirements such as wearing correct PPE, and plant maintenance are all covered under standard Eskom protocols.

20.6 Clearing of vegetation

Vegetation should not be allowed to establish on lined, open areas of the ADF. Grass around the ADF, PCDs and CWDs should be trimmed to prevent veld fires damaging the liners.

20.7 Surface Water monitoring

Surface water monitoring is to be in accordance with the Licence conditions, at set points on site.

20.8 Leachate monitoring

Leachate monitoring is to take place at all exit points such as pipe outlets to dirty water channels and monitoring manholes, where applicable. Frequency of testing and reporting to be in accordance with the Licence conditions.

20.9 Sub-soil monitoring

Monitoring boreholes are to be monitored in accordance with the Licence conditions. There should be at least one upstream and two downstream monitoring boreholes at the ADF and at the dam complexes, east and west.

20.10 Pipeline monitoring

The stream diversion pipelines should be monitored for the following:

- Access control,
- Silt,
- Subsidence,
- Stress cracking or spalling of concrete surface,
- General scouring or other damage,
- Inlet condition,
- Outfall condition.

It is recommended that an internal inspection is carried out twice annually inside the pipe. An inspection protocol is to be set up and results monitored. Monitoring can be done visually by means of remote-control camera equipment, and manually inspected at set intervals and when deemed necessary. It is important that a protocol/ methodology is prepared timeously and adhered to. As with all monitoring relating to possible settlement and / or distortion, a designated and consistent datum should be recorded and used, from the start.

20.11 Attenuation Dams

The condition of the dam walls, spillways and By-pass drains is to be monitored and recorded. Dam 3 is to be inspected by an APP, as required.

Downstream toe seepage and erosion should be monitored and repaired where necessary.

20.12 Pollution Control Dams, Clean Water Dams

Dam levels and inputs should be recorded daily, as well as rainfall figures, and draw-offs. Any discharge events over spillways should be recorded. Water quality should be monitored as per requirements and as per stipulated frequency. PCD-1a, PCD-1b and PCD-2 are to be inspected by an APP, as required

Visual inspections should take place daily for signs of seepage on outer walls and toe, for damage to lined surfaces and for erosion of walls. Silt build-up in dams should be monitored.

The silt traps should also be inspected daily, so that de-silting can be planned pro-actively.

The sub-soil monitoring manholes are to be monitored daily and regularly sampled to monitor water quality. If clean, this water is to be pumped to the environment. If contaminated, the source of contamination is to be sought since this could be indicative of a liner leakage.

20.13 Geomembrane monitoring

Monitoring of strain gauges and temperature moitoring equipment for the geomembrane are to be recorded at frequenccies as per manufacturer’s specification and per Licence requirements, if any. The equpiment may be monitored automatically and even remotely, but manual records must be filed as well.

During construction the Contractor’s surveyor is to establish suitable permanent Bench Marks at positions directed by the Design Engineer, in order to stablish a datum point to monitor stability / movement of the ash dump body. Movement could be indicative of incorrect ash disposal methodology or barrier system failure.

21. CLOSURE PLAN

21.1 Stream Diversion Pipeline

The final stilling basins at Holfonteinspruit pipeline and western tributary and upslope cut-off drains would continue to function post closure. A single perimeter ADF toe drain would remain for clean run-off routing to a controlled discharge point. During the life of the site, these structures, as well as the pipelines themselves, would be monitored, maintained and re-furbished as necessary.

During a long-term risk assessment analysis, it was concluded that, the western Tributary stream would no longer have any significant upslope surface catchment. Access into the pipeline would still be maintained, however, in order to convey a small volume of local stormwater and to allow ongoing monitoring. Should this Tributary pipeline collapse or block post closure, there would be little stormwater flow impact.

The catchment to the Holfonteinspruit is more extensive, and extends into the neighbouring properties to the east. There are mining operations to the east of the ADF. However, the activities are expected to have ceased and to have been rehabilitated by the time the ADF is closed. The final intake structure on the eastern Holfonteinspruit pipeline must therefore continue to operate post closure. Should this section of pipeline collapse or become obstructed, an attenuation dam of wall height approximately 7m will divert upslope runoff around the west and south of the ADF perimeter.

A buttress wall on the south-eastern flank of the ADF will be incorporated into the ash dump itself to a height of approximately 25m height over the valley of the Holfonteinspruit, decreasing to the north and south to the height of the perimeter berm. The buttress wall would provide stablity to the ADF in the event of a pipe collapse by providing an “earth dam wall” should the attenuation structures at that time be inundated or fail. The designs of these will be included in future submission phases of the Kusile 60 year ADF.

21.2 Final Attenuation Dams

The Attenuation Dams on the ADF footprint would all have been removed by the time of final closure.

21.3 ADF Capping and Closure

The ADF side slopes are to be shaped to the required profile during operations. The preliminary closure plan calls for the ongoing capping and rehabilitation of the ADF as each portion of the ADF is completed to its final height. Capping will include a 300mm minmum depth of soil cover, 100mm minimum topsoil and greening with indigenous grasses.

At final closure the goal is to leave a stable, and capped ash landfill with surface drains, chutes and bench drains installed to facilitate controlled surface runoff into the toe drain. The final surface is to be free-draining- shaped on top as per designed levels and no localised ponding areas are allowed.

21.4 PCDs and CWDs

The PCDs will be monitored post closure until such time that all water from the channels is of good enough quality to be diverted to a CWD. The PCDs would be removed and the barrier materials disposed of on a suitable, licenced landfill. A CWD on the east and a CWD on the west should remain, to allow for monitoring prior to water being discharged to the environment. The dams should remain securely fenced, with warning signage for “no entry” and “no swimming”.

22. CONCLUSIONS AND RECOMMENDATIONS

Some of the notable conclusions and recommendations are:

a) Very little suitable engineering fill is present on site. Therefore, a G6 specification material must be sourced from a reliable source / borrow pit nearby.
b) The in-situ materials are highly variable, and it is imperative that a qualified geotechnical engineer is on site during earthworks activities.

c) An on-site soils and concrete laboratory could be considered for the duration of construction for common everyday types of testing such as concrete cubes, soils grading and classification.

d) The attenuation dams will control upslope surface runoff. However, seepage will still occur and construction for Phase 1 pipelines therefore should start at the higher ends. At the start of construction temporary water diversion measures (diversion berms and /or damming berms / pumping of water) may be required under “water management” by the Contractor.

e) Prior to construction an accurate physical survey of valleys of 100m width was recommended, which has now been undertaken, and should be made available to the Contractor for verification and confirmation of benchmarks.

f) The procurement of the pipe sections should be carried out timeously, with consideration of the possible long lead time.

g) Similarly, the availability of various geosynthetic products should be confirmed as shortages do occur from time to time. Local producers should be notified in good time, where local products are available.

h) It is recommended that the tenderers, and later the Contractor, should be provided with the Constructability Report prepared by EPCM which highlights logistical and technical aspects of the project and project programming.

23. REFERENCES

Reports

Design Review - Technical Report, Report No. 22020-T01-CE-RPT-001 Rev B, by EPCM, 12 April 2022.

Design Review - Technical Report, Report No. 22020-T01-CE-RPT-001 Rev B, by EPCM, 12 April 2022.

Application for Water Use Licence for the Kusile 60 Year Ash Disposal Facility at Kusile Power Station, Mpumalanga, Report no. 12712-46-Rep-001 Kusile IWULA-Rev0, by Zitholele Consulting (Pty) Ltd, 6 November 2015.

Request for Approval of Design Drawings for Eskom SOC Kusile Power Station 60 Year Ash Disposal Facility in Terms of Section 50 of the National Environmental Management Act: Waste Act, 2008 – Mpumalanga Province, Ref No. 12/9/11/L193/6 by DFFE 9 June 2021.

Waste Classification Report no. JW030/13/D121-Rev3 by Jones and Wagener Consultants in January 2014,

Application for Water Use Licence for the Kusile 60 Year Ash Disposal Facility at Kusile Power Station, Mpumalanga, Report no. 12712-46-Rep-001 Kusile IWULA-Rev0, by Zitholele Consulting (Pty) Ltd, 6 November 2015.

Jones and Wagener Detailed Conceptual Design Report No. JW140/13/D121 Rev 1, January 2014.

R.M. Koerner, 1994, “Designing with Geosynthetics” 3rd edition.

SANS 1200 – Standard Specifications for Civil Engineering Construction, various.

SANS 10409 (2020)- Design, Selection and installation of Geomembranes;

Drawings

Bound Book Of Drawings A3 size

366-511841 Rev0 Locality Plan
366-511842 Rev0 Topographical Survey
366-511844 60 Year General Arrangement

366-511845	Phase 1 General Arrangement
366-511846	Attenuation Dam Layout
366-511847	Phase 1 Fencing Layout
366-511848	60 Year General Arrangement Layout
366-511849	Phase 1 PCD General Arrangement
366-511850	60 Year CWD General Arrangement
366-511851	Phase 1 CWD General Arrangement
366-511852	PCD 1a Layout and Setting Out Data
366-511853	PCD 1a Sections
366-511854	PCD 1a Sub-soil Drains
366-511855	PCD 1a Details
366-511856	PCD 1b Layout and Setting Out Data
366-511857	PCD 1b Sections
366-511858	PCD 1b Sub-soil Drains
366-511859	PCD 1b Details
366-511860	PCD 2 Layout and Setting Out Data
366-511861	PCD 2 Sections
366-511862	PCD 2 Sub-soil Drains
366-511863	PCD 2 Details
366-511864	CWD 1 Layout and Setting Out Data
366-511865	CWD 1 Sections
366-511866	CWD 1 Sub-soil Drains
366-511867	CWD 1 Details
366-511868	CWD 2 Layout and Setting Out Data
366-511869	CWD 2 Sections
366-511870	CWD 2 Sub-soil Drains
366-511871	CWD 2 Details
366-511872	Road Dam Layout and Setting Out Data
366-511874	Piping and Electrical Reticulation Layout
366-511875	Road Dam Details
366-511876	Phase 1 Attenuation Dam Layout
366-511878	Typical Outlet Details

366-511880	Silt Trap Type 4 Plan Layout
366-511881	Silt Trap Type 4 Sections
366-511883	River Pipeline Catchments
366-511884	River Pipeline Layout
366-511885	Holfonteinspruit Longsection Sheet 1
366-511886	Holfonteinspruit Longsection Sheet 2
366-511887	Holfonteinspruit Longsection Sheet 3
366-511888	Holfonteinspruit Longsection Sheet 4
366-511889	Tributary Longsection Sheet 1
366-511890	Tributary Longsection Sheet 2
366-511891	Distribution Line Diversion Layout
366-511892	ADF Liner and Drainage Details
366-511893	ADF Berm Details
366-511894	ADF Stormwater Management Catchment Details
366-511895	ADF Stormwater Management Plan Details
366-511896	ADF Channels Longitudinal Sections CWD East Temporary
366-511897	ADF Channels Longitudinal Sections CWD East Temporary
366-511898	ADF Channels Longitudinal Sections Dirty W East Temporary
366-511899	ADF Channels Longitudinal Sections Dirty W East Temporary
366-511900	ADF Channels Longitudinal Sections Dirty W North-East
366-511901	ADF Channels Longitudinal Sections Dirty W West
366-511902	ADF Channels Longitudinal Sections Dirty W West
366-511903	ADF Channels Longitudinal Sections Clean W West
366-511904	ADF Channels Longitudinal Sections Clean W West
366-511905	ADF Channels Longitudinal Sections Dirty W North 1 and 2
366-511906	ADF Channels Longitudinal Sections Dirty W Southwest and Southeast
366-511907	ADF Channels Longitudinal Sections Dirty W Southwest and Southeast
366-511908	ADF Channels Longitudinal Sections Dirty W Southwest and Southeast
366-511909	ADF Channels Longitudinal Sections Temporary Drains SE, Dirty W NW
366-511910	ADF Channels Longitudinal Sections Temporary Drains SE, NW

366-511912	PCDs and CWDs Silt Trap Type 4 reinforcement
366-511916	Spillway Type 1 for PCDs and CWDs
366-511917	Spillway Type 2 for PCDs and CWDs
366-511918	Stream Diversion Pipeline Typical Details
366-511919	Attenuation Dam No.1, Layout and Setting out details
366-511920	Attenuation Dam No.2, Layout and Setting out details
366-511921	Attenuation Dam No.3, Layout and Setting out details
366-511922	Attenuation Dam No.4, Layout and Setting out details
366-511923	Attenuation Dam No.5, Layout and Setting out details
366-511924	Attenuation Dams By-pass drains details
366-511925	Attenuation Dam 2 inlet / outlet
366-511926	Attenuation Dam 3 inlet / outlet
366-511927	Attenuation Dam 5 inlet / outlet
366-511928	Attenuation Dams By-pass drains Setting out details
366-511929	Attenuation Dams 1 and 2 Wall setting out
366-511930	Attenuation Dams 3, 4 and 5 Wall setting out
366-511931	Road-channel culvert crossing no.1
366-511932	Road-channel culvert crossing no.2
366-511933	Road-channel culvert crossing no.3
366-511934	Road-channel culvert crossing no.4
366-511935	Base Model Starter Embankment Layout
366-511936	Basin Excavation Layout
366-511937	Basin Excavation Setting Out Points
366-511938	Phase 1 Fill Model Layout
366-511939	Phase 1 Fill Model Sections
366-511940	Pre-Deposit Layout
366-513495	Pre-Deposit Setting Out Points
366-513496	ADF Leachate Drainage Layout
366-513497	ADF Sub-soil Drainage Layout
366-513498	PCD-1A Silt Trap Setting out and details
366-513499	PCD-1B Silt Trap Setting out and details
366-513500	PCD-2 West Silt Trap Setting out and details

366-513501	PCD-2 East Silt Trap Setting out and details
366-513502	CWD-1 Silt Trap Setting out and details
366-513503	Channels Setting Out Co-Ordinates
366-513504	Channel Sections sheet 1
366-513505	Channel Sections sheet 2
366-513506	Channels Drop Box West
366-513507	Channels Drop Box East
366-513508	ADF Final Fill model
366-513509	Road-Channel Culvert crossing No. 5
366-513510	Road-Channel Culvert crossing No. 6
366-513511	Channel switching detail
366-513512	Dams complex groundwater cut-off drains
366-513513	ADF south groundwater cut-off drain
366-513514	ADF pipeline Junction Boxes Setting Out Details
366-513515	ADF pipeline Outlet Stilling Basin
366-513516	Leachate and Sub-Soil Collectors Fittings Schedule
366-513517	Road PCD Silt Trap 1 Setting Out
366-513518	Road PCD Dirty water Channels
366-513522	Stream Diversion Pipeline Junction Box No.4 Plan and Elevations
366-513523	Stream Diversion Pipeline Junction Box No.4 Reinforcing Plan and Details
366-513524	Stream Diversion Pipeline Junction Box No.9 Plan and Elevations
366-513525	Stream Diversion Pipeline Junction Box No.9 Reinforcing Plan and Details
366-513526	Stream Diversion Pipeline Junction Box No.17 Plan and Elevations
366-513527	Stream Diversion Pipeline Junction Box No.17 Reinforcing and Details
366-513528	Stream Diversion Pipeline Junction Box No.19 Plan and Elevations
366-513529	Stream Diversion Pipeline Junction Box No.19 Reinforcing and Details
366-513530	Stream Diversion Pipeline Junction Box No.25 Plan and Elevations
366-513531	Stream Diversion Pipeline Junction Box No.25 Reinforcing and Details
366-513532	Stream Diversion Pipeline Junction Box No.28 Plan and Elevations
366-513533	Stream Diversion Pipeline Junction Box No.28 Reinforcing and Details
366-513534	Stream Diversion Pipeline Junction Box No.34 Plan and Elevations
366-513535	Stream Diversion Pipeline Junction Box No.34 Reinforcing and Details

366-513536	Stream Diversion Pipeline Junction Box No.38 Plan and Elevations
366-513537	Stream Diversion Pipeline Junction Box No.38 Reinforcing and Details
366-513538	Stream Diversion Pipeline Stilling Basin Reinforcement Plan and Details
366-513539	Stormwater Channels North Crossing Rebar Layout
366-513542	Engineered Fill Plan and Sections, Setting Out
366-513544	Test Pit Positions
366-513545	ADF Base Sections Sheet 1
366-513546	ADF Base Sections Sheet 2
366-513547	Attenuation Dams 2, 3 and 5 Guardrails
366-513548	Attenuation Dams 2, 3 and 5 Spillway details
366-513550	Road PCD Tank Layout and Section
366-513551	Road PCD Reinforcement Layout
366-513552	Road PCD Bending Schedule
366-513553	Road PCD Silt Trap 2 Setting Out
366-513556	Growth Plan Phase 1 Stage 1 Platform
366-513557	Growth Plan Phase 1 Stage 1 Section
366-513558	Growth Plan Phase 2 Stage 1 Platform
366-513559	Growth Plan Phase 2 Stage 1 Section
366-513560	Growth Plan Phase 3 Stage 1 Platform
366-513561	Growth Plan Phase 3 Stage 1 Section
366-513562	Growth Plan Phase 4 Stage 1 Platform
366-513563	Growth Plan Phase 4 Stage 1 Section
366-513564	Growth Plan Phase 5 Stage 1 Platform
366-513565	Growth Plan Phase 5 Stage 1 Section
366-513566	Growth Plan Phase 6 Stage 1 Platform
366-513567	Growth Plan Phase 6 Stage 1 Section
366-513568	Growth Plan Phase 7 Stage 1 Platform
366-513569	Growth Plan Phase 7 Stage 1 Section
366-513570	Growth Plan Phase 8 Stage 1 Platform
366-513571	Growth Plan Phase 8 Stage 1 Section
366-513572	Growth Plan Phase 9 Stage 1 Platform
366-513573	Growth Plan Phase 9 Stage 1 Section

366-513574	Growth Plan Phase 10 Stage 1 Platform
366-513575	Growth Plan Phase 10 Stage 1 Section
366-513576	Growth Plan Phase 11 Stage 1 Platform
366-513577	Growth Plan Phase 11 Stage 1 Section
366-513578	General Arrangement Bottom Shiftable Conveyor
366-513579	General Arrangement Top Shiftable Conveyor
366-511580	EC1 Longitudinal Section
366-511581	EC2 Longitudinal Section
366-513582	Attenuation Dam 3 Inlet Chamber Reinforcement
366-513583	Attenuation Dam 3 Outlet Chamber Reinforcement
366-513584	Attenuation Dam 2 Inlet Chamber Reinforcement
366-513585	Attenuation Dam 2 Outlet Chamber Reinforcement
366-513586	Attenuation Dam 5 Inlet Chamber Reinforcement
366-513587	Attenuation Dam 5 Outlet Chamber Reinforcement
366-513588	Attenuation Dams 2, 3 and 5 Spillway reinforcement

Websites

None.

Appendix A

Authorisations

forestry, fisheries
& the environment

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Ref No.: 12/9/11/L193/6
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Attention: Ms Herbst

REQUEST FOR APPROVAL OF DESIGN DRAWINGS FOR ESKOM SOC KUSILE POWER STATION 60YEAR ASH DISPOSAL FACILITY IN TERMS OF SECTION 50 OF THE NATIONAL ENVIRONMENTAL MANAGEMENT: WASTE ACT, 2008 – MPUMALANGA PROVINCE.

The request for approval of design drawings for Kusile Power Station 60 year Ash Disposal facility has reference:

The design drawings were received during December 2020.

1. The following documentation was taken into consideration for decision making:

- 1.1. Kusile 60 year ADF Detail Design Report having reference 15167-4S-Rep-004 Rev 6 dated September 2018 and compiled by Zitholele Consulting.
- 1.2. A4 Box of rolled up drawings.
- 1.3. Zitholele email of Thursday, 25 March 2021, identifying the Blight 2010 reference as a Mine Waste Disposal Design with filters for gild tailings (not ash).
- 1.4. Eskom WeTransfer link sent via email on 24 March 2021, from Mr. Beneke, providing appendices referenced in the 2018 Report Rev 6 but not included therein and some addenda dated November 2020.
- 1.5. DWS email acknowledgement to Eskom and Zitholele Consulting Engineers dated 26 March 2021.
- 1.6. Comments from DWS: Engineering Services dated 20 April 2021.

2. Consideration of Design Compliance with Norms and Standards

2.1. The Kusile Power Station (PS) 60 year Ash Disposal Facility (ADF) design report dated September 2018 by Zitholele Consulting and checklist dated 5 February 2021 advises the foot print area is 748 ha in quaternary catchment B20F. Note that the Applicants Consultants have previously met with the DWS Engineering Services on the Kusile PS for the 60 year ADF designs in 2013 and 2016.

2.2. The works include the 60 year ADF to be constructed in modules. Pollution Control Dams (PCDs) 1 to 6 and Clear Water Dams (CWDs) 1 and 2. The report section 4.1 lists the standards applied in the design of infrastructure but omits SANS 1526 (2015) and SANS 10409 (2020). Note that the Engineer applies NWA mining regulations to the Kusile ADF facilities and in the geotechnical assessment considers permeability of 10^{-5} cm/s to be adequate for the Compacted Clay Liner (CCL) (see page 27) which contradicts Minimum Requirements section 8 and influences containment performance. Foundation settlement of 400-900mm is anticipated under full ADF height.

2.3. The ADF barrier includes a herringbone under drainage system of 600mm by 600mm trenches with 100mm sand bed, High Density Polyethylene (HDPE) drainage pipe and 19mm aggregate stone which is geotextile wrapped, with finger drains having 160mm pipes that lead to a collector drain of 20mm diameter pipe. This layout mimics the Leachate Collection System (LOS) collector pipes, which drain through the normal starter wall and the liner system being fitted with a boot to facilitate drainage to the dirty collection system. The liner is a 300mm thick CCL of selected material with a 1.5mm Single Textured (ST) HDPE Geomembrane (GM), followed by a cushion Geotextile (GT) and 300mm ash drainage layer augmented by the LCS finger drains. Section 4.6.5.6.3 defines the soil selection criteria for the CCL. The CCL material selection is not ideal and deviates from the Minimum Requirements due to the more silty nature, but claimed adequate impermeability. **Thus, Construction Quality Control (CQC) and Construction Quality Assurance (CQA) of the CCL and in-situ permeability testing will be critical to acceptability.**

2.4. The Engineer assumes that Kusile ash will be similar to that of Kendal PS, the latter being renowned for its low pozzalanic action and hence low LCS temperature. It is deemed prudent to measure the leachate temperature at the existing Kusile ADF which has operated for some years.

2.5. The stability analysis is based on inter alia assumed critical sections and material assumptions related to Kendal PS and geosynthetic published data for residual shear strength (2005). Thus, the conclusion (page 67) that the static Factor of Safety (FoS) is above 1.27 will have to be confirmed based on actual material properties in the early stage of construction and with ash currently produced at Kusile PS, as required by SANS 1526 (2015) and GRI GM13 (2019).

2.6. The storm water management infrastructure includes two clean water dams, six PCDs of which two are required in the 1st Phase, 18 temporary clean water attenuation basins, diversion of the Klipfonteinspruit and associated attenuation basins, separation of clean and dirty water by channels and berms. The water balance modeling includes provision for inter PCD transfer to minimize the risk of spillage and optimize the reuse of PCD water, however, makeup water will still be required. The conclusion of the model is that the PCDs 1 and 2 are to be operated below 50% capacity to avoid spillage risk exceedance based on a daily time step model (see page 85). Note the PCD volume requirement is over half a million cubic meters spread between the six PCDs, all of which have a storage capacity of Full Supply Level (FSL) above 50 000m². The two CWDs have capacities of 38 000m² and 74 000m² respectively. Note all the PCDs except number 2 have a depth of 5m or greater and need to comply with Dam Safety Regulation 2012. **Note: The report states the Non Overspill Crest (NOC) is at least 0.8m above FSL of the PCDs, which is not the same as the best practice guideline of and operational freeboard of 0.8m below the FSL to accommodate wave action and storm influences etc.**

2.7. Note also that the PCD embankment material specification has not been confirmed as suitable for the material on site and the design report stipulates a borrow pit or commercial source for embankment material may need to be sought – which would significantly increase costs without informing performance. It is inexplicable why Zitholelele would have undertaken an extensive geotechnical site investigation and produced an embankment dam design with specifications without having confirmed availability of material on site (see page 101).

2.8. The PCD is a conventional Class C composite liner (300mm CCL plus 1.5mm HDPE GM ST on wall area) with 100g/m² GT cushion layer, a floor area ballast of 50mm thick screed, plus 150mm thick lightly reinforced 30/19 concrete slab, whereas the wall area has a 75mm thick Geocell filled with 8% cement stabilized soil or ash. Under drainage performance is not defined in the report which states drainage is to keep "hydrostatic pressures to a minimum", despite the shallow groundwater presence – this could lead to buoyancy and damage as recently seen at Majuba PS after the December 2020/January 2021 construction break. The estimated leakage rates reported on page 105 are found to be excessive and reflect inadequate CQC and construction competence. **Note the claim on CQC being thoroughly addressed in the QCA document, which was not appended to the design report – a November 2020 QCA plan by Golder Associates was forwarded electronically by ESKOM in March 2021.** Each PCD also has a concrete outlet facility and while the GM make-off to spillway and concrete works is specified as a stainless-steel baton system, the peak tensile stain at this point is not addressed. Lessons could be learnt from the Medupi PS raw water reservoir experience.

2.9. The 5 single barrel silt traps proposed are of reinforced concrete.

2.10. The river diversions for the Klipfonteinspruit and Holfonteinspruit are considered under the WUL application. The channels are sized using the Rational Method for catchment runoff and Manning's equation for channel requirements.

2.11. The new/additional facilities for the operation of the ADF are based on the Camden facilities and are not considered under the WUL application other than expected to comply with other legislation, in particular the NWA and NEMA and associated regulations. It is noted that the additional ablution drains to a conservancy tank (see page 130), which is to be serviced. Monitoring records and performance can be addressed by condition of authorization.

2.12. The 2018 report REV6 contains numerous quotes reflecting the lack of diligence and care by the Engineer and signatories to the report and checklist.

2.13. It is noted that the filter design criteria referenced as Blight 2010 refer to Terzaghi criteria for granular material as amended by Bertram and the USBR in and about the 1950's with a simplistic approach to geotextile filter design based on Apparent Opening Size (AOS) of the geotextile. That reference to filter criteria applicable to Mine Waste Disposal/Tailings Dam design does not consider the cohesive nature of ash deposits due to both calcium presence and pozzolanic action granular filter embrittlement and GT filter clogging and should be amended to confirm construction yields competent filter and drainage systems such as advocated by the Sherard criteria of 1989 for critical filters, the ICOLD Guideline in filters and drains and on tailings facilities and more recent advances in geotextile filter design and compatibility testing such as the gradient rotation and fine fraction tests amongst others.

2.14. The stability and safety of the facilities is of extreme concern. The 2018 design report Rev6 specifies a peak and a residual shear values of 34 and 30.7 degrees respectively, whereas the 2020 addendum defines the respective values of 24 and 21 degrees, whereas literature references 18 and 16 degrees respectively for a textured HDPE GM/CCL, which significantly influences stability. This alludes to a predetermined geomembrane/CCL test results which has not been provided. The concern is based on past discussions with consultant JV where GSE textured material form the Houston factory was tested in the USA as the closest supplier to the lab and that the material is known to be significantly "rougher" than the similar form of textured material produced by SOLMAX Malaysia plant and the Roward plant in Saudi Arabia. Similarly, microspike specifications are improper for interface shear alone because density, shape and pattern influences performances, as shown at the LaWTIG 2017 keynote address and confirmed in the GRI GM13 amendment of 2019. Thus cost and performance may be severely affected by the specification. The basis of the 24 and 21 degree specification for the particular HDPE GM and Soil interface shear specification should be provided post haste, because it seems that it is more of a sandy material rather than clayey material being used in the CCL.

3. DECISION

Based on the technical assessment of the documents submitted by Zitholele Consulting, the Department conditionally approves the Eskom Holdings SOC Kusile PS design report and drawings for the 60 year dry ADF Phase 1, pollution control dams 1 and 2 and clean water dams 1 and 2 as recorded in report number 15167-45-REP-004 Rev 6 by Zitholele Consulting dated September 2018 (signed by N Rajasakran, PrEng 20070218) and design amendment appendices dated November 2020, submitted in March 2021 based on the following:

3.1 General Conditions

a) Limitations: This conditional approval does not exempt the designer from complying with any other legislation. This review refers only to the activity as specified and described in the signed design report and drawings listed under documentation submitted for consideration.

b) Commencement: One month's written notice must be given to the Department before commencement of construction activities. Such notice shall make clear reference to the site location details and the reference number of the project as indicated above and one month's written notice must also be given to the DWS (Directorate RP&W) before commencement of the operational phase activities. Such notice shall make clear reference to the site location details and the reference number of the project as indicated above.

c) Deviation from accepted design: The licence holder must notify the Department in writing within 24 (twenty-four) hours if any condition of this design and its acceptance cannot be, or is not, adhered to during construction and operation. The notification must be supplemented with reasons for non-compliance, and proposed rectification measures.

d) Engineering Records: Design and construction records, including topographical surveys and methodical material test results (on all materials used), shall be maintained and archived and accessible for life of the facilities (including decommissioning).

e) Accountability: The authorities shall not be held responsible for any damages or losses suffered by the applicant or its successor in title in any instance where construction or operation subsequent to construction is temporarily or permanently stopped for reasons of non-compliance by the applicant with the conditions of approval as set out in this document or any other subsequent document emanating from these conditions of acceptance.

f) Overtopping of PCDs: The discharge of leachate or polluted water from the pollution control dam and clean water dams is to be reported as an incident within 24 hours of an event, and treated as such, with appropriate remediation.

g) Demarcation, cordons, barriers and warning systems: The facility manager must place cordons, barriers and warning systems around facilities to define the nature and extent of each disposal or waste management area, and avoid intersection of different waste types as per the NEMWA Regulations 2013 (e.g. there must be a clear distinction between pollution control dams and clean water dams).

h) Reporting to authorities: All significant differences between predicted and actual performances of waste management facilities shall be reported to the authorities annually in writing.

i) Independent Auditing: The facility owner is to ensure that there is no alignment between Engineer, contractor, subcontractor, material suppliers and CQA agent in the developments design, construction and implementation of construction quality assurance.

3.2 Specific Conditions

a) Starter embankment co-ordinates: At the end of each phase of barrier construction, the nominal 1.5m high starter wall embankment shall be surveyed and the updated list of coordinates of each point of intersection shall be included in the construction completion report and forwarded to the authorities along with the certificate of completion and its supporting evidence.

b) Service life determination: Due to the service life determination in the design report being limited to temperature assumptions alone and not recognizing the effect of tensile strain on the GM performance, the licence holder shall:
    i. Include at least 3 thermocouples midway between herringbone drains to confirm temperature assumptions; and
    ii. Include a total tensile strain assessment at critical points in the GM such as during construction placement of the pioneering layer in the trial pad (albeit part of the final footprint) and at the GM/concrete outlet works make-off in the PCD embankment penetration.

c) Standard specifications: The standard specifications for the materials and methods employed in the waste containment shall be revised to be the latest version thereof including SANS 1526 (2015) for HDPE GMs revised (in 2019) and SANS 10409 specified as 2005 shall be revised to the 2020 version, amongst others.

d) Compacted clay liner performance: The CCL permeability shall be confirmed by double ring infiltrometer tests or similar approved on each year at least three positions for each five-year phase on the in-situ material (as per the Minimum Requirements 1998). The results of all permeability tests shall be included in the construction completion report (noting the nature of the CCL material described and recorded as 10^{-5}cm/s or less, compared to the norms and standards).

e) Instrumentation: The ADF and PCD developments shall include instrumentation to confirm the design and construction assumptions made which influence containment performance such as confirming the above liner piezometric level and GM tensile strain limitation (in particular adjacent to the concrete outlet works of the PCDs).

f) CQA: Due to the failure of the licence holder and Zitholelele Consulting Engineers to provide the design report appendices and CQA plan as referenced in the 2018 report, the licence holder shall comply with the DWS standard CQA plan (available on the DFFE website) as a minimum with all deviations therefrom being agreed to in writing prior to commencement of construction. Noting the need to comply with Treasury Regulation in respect to product specifications.

g) Electric Leak Location Survey (ELLS): Due to contradictions between the 2018 design report, the 2020 addenda and in particular technical memorandum design addendum, the CQA shall include ELLS in accordance with D8265 or similar approved amendment with deviation or relaxation for cost saving based on confirmed competence of the lining material and installer, including certificated/accredited welding technicians.

h) Reference Procedures, Standards and specifications (Work specifications) for Kusile ADF: Due to the undated 181 page standards and specifications document referenced by the CQA requiring the textured HDPE geomembrane/CCL interface shear peak value to be 34 degrees and residual shear value to be 30.7 degrees, and the design addendum specifying 24 and 21 degrees requirements respectively, being in contradiction with the design report which references published literature (giving a peak textured GM/CCL interface shear of 18 degrees and residual of 16 degrees), the design and tender documentation is considered contradictory and unsound. Thus, the requirement for the lining contractor to timeously demonstrate possibly impractical values, which tends to show a predetermined brand of product, the licence holder shall confirm the consulting engineer's specification is reasonable and not anti-competitive (in accordance with the PFMA and Treasury Regulations) for in-situ material prior to award of tender.

i) Filter Compatibility: Filter compatibility between granular and geosynthetic filters ad base material of ash or soil shall be confirmed as being in accordance with current internationally accepted norms and standards for the material on-site. This confirmation shall be in writing to the authorities prior to use for disposal of pollution control.

j) Stability confirmation using site materials: The ADF and PCDs stability minimum FoS shall be confirmed post construction for the actual materials used and ash to be disposed (which is already in production). This confirmation shall be in writing to the authorities prior to use for disposal or pollution control.

k) Stability monitoring: The licence holder shall monitor the area in front of the advancing ash stack toe for signs of deformation in the barrier system/foundation and report such to the authorities as an incident within 24 hours of occurrence.

l) Construction completion: The Engineers certificate of completion shall be augmented by a construction completion report confirming compliance with conditions of authorization and the CQA plan as amended. This report shall include statistical analysis of test results on all materials used as per the specifications and record the nature of test, test method, number of tested, minimum, maximum and mean values and standard deviation as well as the number of failures encountered and performance of repairs.

m) Monitoring of water quantity and quality: The licence holder shall maintain monthly records of clean and dirty water volumes during operational phase, including inter PCD flows. Such monthly monitoring results are to be made available to the authorities annually and be used to inform the licence review every 5 years as well as the water conservation and demand management plan.

n) Water conservation and demand management: Due to large shallow PCDs and predicted water losses with resultant make-up water requirements, the licence holder shall develop and implement a water conservation and demand management plan which pursues water use efficiency and pollution control.

Should you have any queries, please do not hesitate to contact this office.

Yours sincerely

MS MISHELLE GOVENDER
CHIEF DIRECTOR: HAZARDOUS WASTE MANAGEMENT AND LICENSING
DATE: 09/06/2021

Appendix B

Geotechnical Summary

Document Title:	**Consolidated Geotechnical Report**
Eskom document no.:	xxxx
Contractor document no.:	366-511915 – Appendix B
Document type:	Technical Report
Contractor Name:	EPCM Bonisana
Revision no.:	**0.1**
Prepared by	J Bloem
Package/System name:	60 Year Ash Disposal Facility – Task Order 3
Unit/s no.:	xxxx
Contractor Name:	EPCM
Contractor no.:	xxxxx
Plant Identification codes:	

Rev	Date	Document Status	EPCM Reviewed	EPCM Approved	Client Review/ Approval
B	10-11-2023	External Review		Signature	Signature
0	14-12-2023	Construction		Signature	Signature
0.1	**22-02-2024**	**Construction**		Signature	Signature

TABLE OF CONTENTS

APPENDICES

Abbreviations

EPCM	EPCM Consultants SA
SoW	Scope of Work
ADF	Ash Disposal Facility
PCD	Pollution Control Dam
CWD	Contaminated Water Dam

1. TERMS OF REFERENCE

The terms en reference for this report is to collate all previous data to accommodate the development for the phase one ADF, PCD and stream diversion sections. All previous documents done on previous investigations will be used by using exstraction to summarise the findings. All the documents used are listed below in paragraph 2.1.

2. BACKGROUND INFORMATION

2.1 Documents Background

The available reports used are as follows:

Jones & Wagener/ Zitholele JW140/13/D121 rev2; Kusile Power station 60 year ADF: Engineering Detailed Concept Design Report, 2013&14

Jones & Wagener/ Zitholele JW195/15/D121-07; Geotechnical Investigation for the proposed 60 yr ADF at Kusile. Geotechnical Investigation Report, 2015

EPCM 366-490112 REVB; Kusile 60 Year Ash Disposal Facility, Geotechnical Analysis Report, Document no.: 22020-TO1-CE-RPT-04-001 Rev. B 19/06/2023.

In last report data from WSP Golder, EPCM Bonisana, and Soiltecnix were also used and summarised with the existing soil profiles and laboratory data.

2.2 Scope of report

This report surves to abriefly consolidated all previous reports available for the purpose of geotechnicail evalution of the ADF, PCD and CWD areas, and Stream Diversion Pipeline.

2.3 Objectives of this Report

The objective of this report is to document a summary of all data to be used for the specific design of the structures on site.

3. GEOLOGY

According to Map 2528 of Pretoria 1:250 000 scale geological sheet, the regional lithostratigraphy of the investigated site comprises of the Ecca and Dwyka Groups of the Karoo Supergroup. The constituents of these sedimentary formations are predominantly the tillites and shales of Permian age (see Table 1 below). The Karoo sequence is underlain by the Magaliesberg and Silverton Formations, Vaalian aged Pretoria Group which are dominantly shales and quartzites with subordinate hornfels. These lithological units, Karoo Supergroup and Pretoria Group, are intruded by the dark grey diabase of the Vaalian to post Mongolian ageed. The region is not underlain by dolomite.

4. TOPOGRAPHY

The site is characterised by an undulating topography. The main features causing the undulation are the two drainage streams that are present. The Holfonteinspruit that is situated in the middle of the site, drains in a northerly direction. Along the northern boundary, between the existing 10 year facility and Area A (as shown in Figure 3), the Klipfonteinspruit is encountered. The elevations on site range from 1500 metres above mean sea level (mamsl) in the upper crestal areas to 1440mamsl along the Holfonteinspruit

5. CLIMATE

The investigated area has a subtropical climate; the summer seasons are normally characterised by hot, humid and wet weather conditions whilst the winter seasons experience low temperatures and dry conditions. The average annual temperature is 15.5 ºC and approximately 690 mm precipitation falls annually. Monthly precipitation levels reach their peak in January at approximately 124 mm, whilst June the area’s driest month with only about 7 mm of precipitation.

The site is located in an area that is characterised by a humid climate where the Weinerts Climatic N value2 is generally <5. In such areas chemical decomposition of the soil /

rockmass predominates and consequently, depending on the rock mineralogy, residual soils tend to be more deeply weathered and clayey compared to areas affected by mechanical breakdown (N>5) of the rock fabric

6. SITE SOIL TYPES AND DESCRIPTION

This section contains extractions from the existing geotechnical reports that adequitly describes the soils profile on site with section 7 that describes each area seperately.

6.1 Topsoil

Topsoil consistency ranged between very loose to very dense, porous, silty fine to fine sand with roots. The topsoil depth over most of the area is typically 0.3m. There were, however, test pits where the topsoil depth ranged between 0.4m and 0.6m, as well as areas where almost no topsoil was encountered. The topsoil was classified as hillwash in the profiles. The average thickness of the topsoil across the site can be assumed to be 0.3m.

6.2 Gully Wash

These soil materials are generally located adjacent to the drainage channels and account for less than 5% of the ADF footprint area with the thickness that varies between 0.4 to 1.5 m and can generally be described as moist, dark grey mottled yellow brown, soft-firm, slightly shattered and pinholed silty clay with sand components.

6.3 Colluvium/Hill Wash

Topsoil is underlain by hillwash of varying colour across the site. Over most of the site the hillwash ranges from medium dense to very dense, porous and shattered, slightly silty to silty fine sands. Slightly clayey to clayey sand and fine sand hillwash were encountered to the south and east of the site (see possible clay sources below). The hillwash was encountered at depths ranging between 0.3m and maximum 1.6m. The thickness of the hillwash layers ranges between 0.4m to 1.3m with an average thickness of 0.7m.

Towards the bottom of the layer ferruginisation in the form of ferricrete nodules or poorly developed concretions was observed in some of the test pits (see Ferricrete below). Gravels of 20mm and cobbles & boulders of 600mm were encountered in several of the test pits, of which some were an indication of a distinctive pebble marker.

The Golder report states that the colluvial and residual soils (tillite, shale, diabase) encountered on site are suitable for use in the compacted clay layer, other parts of the ADF and the dam barrier systems. This assessment is generally accepted. Further insitu and laboratory testing will however have to be conducted to confirm beyond reasonable doubt that these materials are suitable.

6.4 Ferricrete

The Ferricrete is fairly consistent around the site occupying on about one-third of the surface area, and occurs over the ADF footprint. Fluctuation in groundwater-level has been inferred from the presence of the mainly ferricrete horizons in the aforementioned area. Across the site, this horizon generally occurs below the transported material and is underlain by either the residual material or rock material. Its geotechnical character generally described by sub-angular, medium to coarse clayey gravels with a silty sand matrix, well-cemented, matrix supported with a density ranging from medium-dense to hardpan (rock strength)

In most of the reports ferricrete material is mentioned as a possible material source. However, the spatial distribution and limited thickness of identified seams within the site however limits the potentially usable quantities of these materials. Other options will thus have to be investigated such as the dolerite material on the northern side of the site and the existing quarry to the northwest of the ADF facility.

6.5 Pebble Marker

The occurrences of these units are limited to the areas within the proposed ADF footprint and it generally underlies the topsoil layers in most soil successions. About 34% of the investigated areas is commonly characterised by a 0.15 m horizon of characterised by a 0.15 m horizon of very loose to loose, medium gravel to cobble-size gravels with slightly moist, reddish brown, silty sand matrix.

6.6 Residual Tillite

Hillwash is underlain by tillites in most of the test pits across the site. Residual tillites consist of mostly medium dense, shattered or fissured, slightly silty fine sands to fine sandy silts. Residual and reworked residual tillites range in thickness between 0.6m to 1.7m. These layers were encountered at depths between 0.6m to 1.1m. Residual and reworked tillites are in turn underlain by weathered and completely weathered tillites in most test pits, consisting of dense or stiff, shattered or fissured, clayey sands or fine sandy silts. These weathered tillites range in thickness from 0.9m to approximately 1.4m.

These horizons extends to more than 2.5 m and are generally the thickest unit of the soil succession on the footprint. It occurs over approximately one-third of the site, the soil successions indicated a layer charicterised by slightly moist, firm to stiff, sandy silt with clays and coarse to cobble-sized gravels of mixed origin. Both open-textured (pinhole and honeycomb structures) and slight shattering of the residual material was noted.

6.7 Residual Siltstone/Shale

The residual shales are cohesive and characterised by shale veins and relict bedding from the parent rock shales. The horizon is found over approximately one-third of the study area and are usually underlain by laminated and fissile shales at the bottom of the test pits. In general, the corresponding geotechnical description for the material is Slightly moist to moist, light grey mottled yellow brown, soft to firm, shattered, clayey silt with some sand.

6.8 Residual Sandstone

Residual sandstone layers are typically found overlying sandstone bedrock e.g. test pits AD 17 and AD 47. These soil profiles are confined to the south-western portions of the site.

This horizon covers approximately 5% of the ADF footprint and its descriptions are normally a slightly moist, dusky red, intact, matrix supported, silty sand with coarse gravel to cobble-size sandstone rock clasts. The density of these horizons is generally loose to medium-dense.

6.9 Residual Diabase/Dolerite

These units are usually marked by cobble-size corestones of diabase rock which are supported in a silty clay matrix. The matrix is characterised by firm to stiff silty clay, with some shattering and slickensided structures. The residual diabase has a higher clay content than the residual tillite material and are usually located in the river diversion and northern PCD areas. These horizons have been described as moist, dark yellowish brown mottled grey and dark red, firm, shattered and often slickensided, silty clay with corestones that have characteristic onion-skin weathering

7. STREAM-BED

7.1 Sufficiency of data

The report does have sufficient data to evaluate the sub-soil condition on the stream diversion section. The recent report has sufficient test pits done on the pipeline diversion i.e. AD TP 67 – 77 with the Golder profiles, AD 01, 04, 06, 11, 15, 29, 36, 62,78, 81, 83, and 92, done on an offset that fills some of the gaps in between the latest test pits. The layout of test-pits is attached in Appendix GA.

7.2 Profiles

The general soil profile on site confirms the geology according to the geological map 2528 Pretoria with sandstone underlain by tillite and shale on the northern part of the site.

The generalized profile in the valley line can be summarised as follows:

0.0 – 1.2m	Topsoil, Alluvium and Colluvium - Ferruginised in places
1.2 – 2.0m	Residual Material

2.0 – 4.4m Bedrock – Refusal on Tillite, Sandstone and Shale.

Refusal varies due to the presence of a ferricrete layer and the uneven weathering of the local bedrock.

The pipe foundations can be founded on the highly weathered tillite/sandstone at a depth of 3.0 – 3.5m below NGL. Foundations can be designed for a bearing capacity of 150 kPa at this depth.

No suitable engineering fill is present along the pipe diversion route. Therefore, a G6 quality material must be sourced from a reliable source close to, or on site. Shallow diabase material was present at AD TP53 – 59 that can be further explored if possible.

7.3 Groundwater

Groundwater seepage was encountered between 0.7 to 3.4m below NGL and was also dependant on the proximity to the water course.

7.4 Constraints and Geotechnical Considerations

Due to saturated nature of the sub-soil and the proximity of the water course, stability of the test pits played a big factor. Therefore, precaution must be taken during excavation by shoring or benching the box cut.

It is recommended to excavate the entire pipe position with an initial trench to ensure seeping water drains away from the construction site and that the construction should be started at the upstream position of the stream diversion pipeline.

Sub-surface drainage must be constructed to make sure that all the water seepage will be diverted and controlled below the pipe foundation levels.

A pioneer layer made up of dump-rock, at least 300mm thick, should be constructed as a drainage layer below foundation levels with the engineered base layers for the pipe bedding material on top of the dump-rock.

The dump-rock can be compacted with at least 8 passes with a drum roller.

A geotextile could be placed on top of the dump-rock with a G5 class material compacted to at least 95% Mod AASHTO that would form the base for the founding material.

It is also critical that all material must be placed in layers no thicker than 150mm and compacted to 95% Mod ASSHTO around the entire pipe diameter to ensure the integrity of the pipe.

8. PCD

8.1 Sufficiency of data

The reports do have sufficient data to evaluate the sub-soil condition on the PCD site. The recent report has sufficient test pits done on the PCD sites i.e. AD TP 01 – 14 and 19 - 22. The layout of test-pits is attached in Appendix GA.

8.2 Profiles

The general soil profile on site confirms the geology according to the geological map 2528 Pretoria that the site is underlain by shale and tillite.

The generalized profile can be summarised as follows:-

0.0 – 0.4m	Topsoil and Colluvium
0.4 – 1.3m	Ferricrete to Hardpan ferricrete
1.3 – 3.0m	Residual Material
3.0 – 5.1m	Bedrock – Refusal on Tillite and Shale

Refusal varies due to the presence of a ferricrete layer and the uneven weathering of the local bedrock.

The base of the PCDs should be excavated onto the highly weathered tillite/shale at a depth of 3.5 – 4.0m below NGL. Foundations can be designed for a bearing capacity of 150 kPa at this depth.

Due to the variable nature and limited depth of the suitable engineering fill on site a G6 quality material must be sourced from a reliable source close to, or on site. Shallow diabase material was present at AD TP53 – 55 that can be further explored if possible.

8.3 Groundwater

Groundwater seepage was encountered between 2.6 to 4.6m below NGL. However, the Golder report also mentions slight seepage in two of the test pets at a depth of 1.0m below NGL that indicates a fluctuation of the seepage level where ferricrete is present and was also dependant on the proximity to the water course.

8.4 Constraints and Geotechnical Considerations

Sub-surface drainage must be constructed before construction to make sure that all the water seepage will be diverted and controlled before and during construction.

It is also critical that all material must be placed in layers no thicker that 150mm and compacted to 95% Mod ASSHTO to ensure the integrity of the side walls.

9. CWD

9.1 Sufficiency of data

The reports do have sufficient data to evaluate the sub-soil condition on the CWD site. The recent report has sufficient test pits done on the PCD sites i.e. AD TP 01 – 14 and 19 - 22. The layout of test-pits is attached in Appendix GA.

9.2 Profiles

The general soil profile on site confirms the geology according to the geological map 2528 Pretoria that the site is underlain by shale and tillite.

The generalized profile can be summarised as follows:-

0.0 – 0.4m	Topsoil and Colluvium
0.4 – 1.3m	Ferricrete to Hardpan ferricrete
1.3 – 3.0m	Residual Material
3.0 – 5.1m	Bedrock – Refusal on Tillite and Shale

Refusal varies due to the presence of a ferricrete layer and the uneven weathering of the local bedrock.

The base of the CWDs should be excavated onto the highly weathered tillite/shale at a depth of 3.5 – 4.0m below NGL. Foundations can be designed for a bearing capacity of 150 kPa at this depth.

Due to the variable nature and limited depth of the suitable engineering fill on site a G6 quality material must be sourced from a reliable source close to, or on site. Shallow diabase material was present at AD TP53 – 55 that can be further explored if possible.

9.3 Groundwater

Groundwater seepage was encountered between 2.6 to 4.6m below NGL. However, the Golder report also mentions slight seepage in two of the test pets at a depth of 1.0m below NGL that indicates a fluctuation of the seepage level where ferricrete is present and was also dependant on the proximity to the water course.

9.4 Constraints and Geotechnical Considerations

Sub-surface drainage must be constructed before construction to make sure that all the water seepage will be diverted and controlled before and during construction.

It is also critical that all material must be placed in layers no thicker that 150mm and compacted to 95% Mod ASSHTO to ensure the integrity of the side walls.

10. ADF PHASE 1

10.1 Sufficiency of data

The report does have sufficient data to evaluate the sub-soil condition on the stream diversion section. The recent report has sufficient test pits done on the entire ADF site that includes most of the test pits done on site. The relevant test pits used on the ADF Phase 1 section was AD TP 23 – 30, 65 and 67 – 69, AD 01, 02, 04, 11, 13 – 15, 22, 24, 34, 54, 65 – 68, 82, 83, 89, 90, 97, 99 and 100. The layout of test-pits is attached in Appendix GA.

10.2 Profiles

The general soil profile on site confirms the geology according to the geological map 2528 Pretoria with sandstone underlain by tillite and shale on the northern part of the site.

The generalized profile in the valley line can be summarised as follows:

0.0 – 0.5m	Topsoil,
0.5 – 3.0m	Colluvium & Alluvium
2.0 – 5.2m	Residual Tillite/Shale

.

Refusal varies due to the presence of a ferricrete layer and the uneven weathering of the local bedrock.

Due to saturated nature of the sub-soil and the proximity of the water course, stability of the test pits played a big factor. Therefore, precaution must be taken during excavation by shoring or benching the box cut.

No suitable engineering fill is present on the site. Therefore, a G6 quality material must be sourced from a reliable source close to, or on site. However, some of the material on site does have liner quality but due to the inconsistent and variable thickness of the layers selection of the material will be difficult to control.

10.3 Groundwater

Groundwater seepage was encountered between 0.7 to 3.3m below NGL with most of the test pit indicating side walls.

10.4 Constraints and Geotechnical Considerations

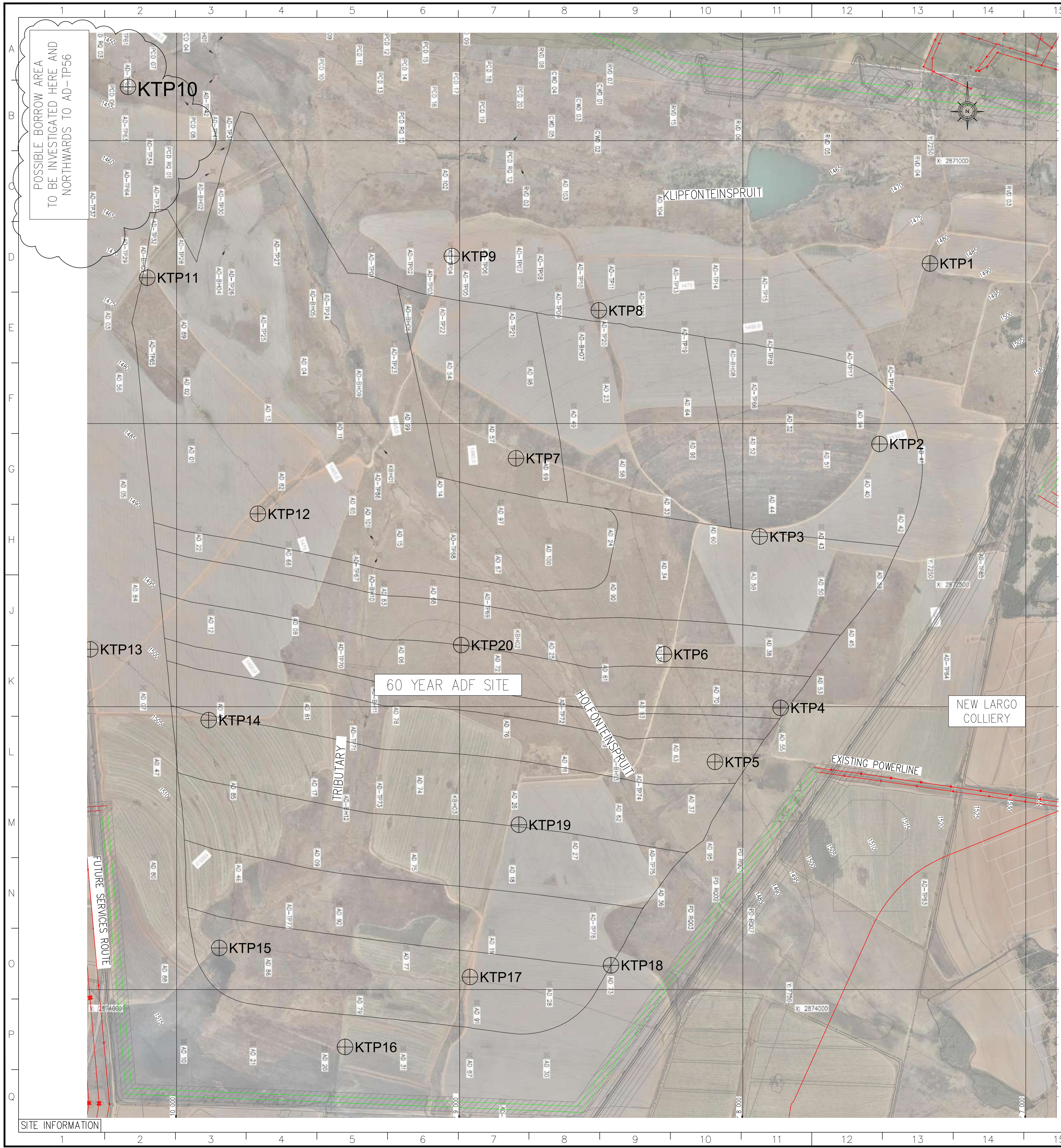
Due to saturated nature of the sub-soil and the proximity of the water course, stability of the test pits played a big factor. Therefore, precaution must be taken during excavation by shoring or benching the box cut.

Sub-surface drainage must be constructed to make sure that all the water seepage will be diverted and controlled below the pipe foundation levels.

Due to the inconsistent nature of the clayey material on site it is recommended that a GCL liner should be considered for the ADF.

APPENDIX GA:

TEST PIT LAYOUT



.	0	27.11.2023	ISSUED FOR CONSTRUCTION	MM	BLJ	KPM			
.	B	30.10.2023	ISSUED FOR CLIENT REVIEW	.	.	.			
.	A	2023.10.10	ISSUED FOR INTERNAL REVIEW	MM	BLJ	KPM			
D.O.	REV	DATE	INDEX REFERENCE :	REV BY	CHKD BY	AUTH BY	KKS APP	REFERENCE DWG. No.	REFERENCE DRAWINGS

60YR ADF SCOPE LEGEND	
NAME	LABEL

KKS PBS: 00UEQ01

ASH DISPOSAL FACILITY

NOTES:
COORDINATE SYSTEM : LO29 WGS-84

Sign:.......
K.P.MATULOVICH
Pr. Eng. 20190729
Date: 30-11-2023

DRG REGISTERED		–	AUTHORISED FOR ESKOM BY	DATE
DESIGN APPROVED	27 11 23	KPM	–	–
KKS APP		–	AUTHORISED FOR ESKOM BY	DATE
			–	–
AUTH BY		–	ESKOM HOLDINGS LTD. REG. No. 2002/015527/06	
CHKD BY	24 22 23	KPM		
DRAWN BY	22 11 23	MM		
SCALE	1:5000			

KUSILE POWER STATION
ASH DISPOSAL FACILITY

TEST PIT POSITIONS

Eskom

366–513544

SHEET	REV
1	0

SIZE GROOTTE A0L

C:\Users\Molly.ENVITECHJHB\Envitech Solutions PTY-LTD\Communication site – Documents\Johannesburg\Projects\ES projects\816-2023 EPCM – Eskom 60yr ADF\3. Project Stage\Design Drawings\366-513544 REV0.dwg Dec 01, 2023 – 6:52pm

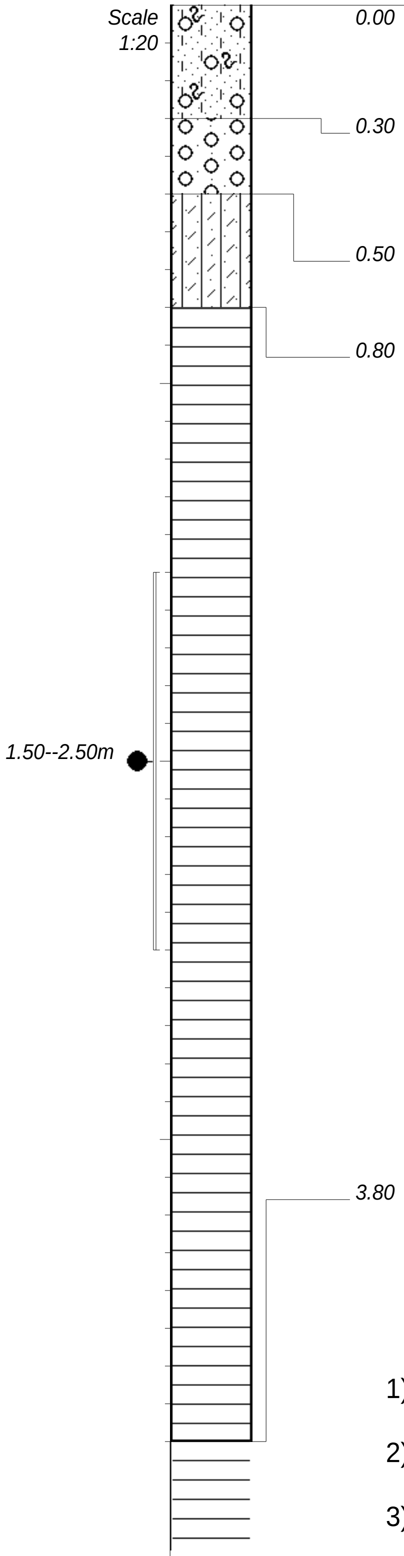
APPENDIX GB: Logs

dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP01
Sheet 1 of 1

JOB NUMBER: RS22025



Moist, brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel to occasional cobbles of mixed origin, **TOPSOIL**. Scattered roots.

Moist, brown, medium dense, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded to rounded gravel to cobbles of mixed origin, **PEBBLE MARKER ZONE**.

Moist, orange streaked red to grey blotched brown, firm to stiff, **relict to laminated to open structure**, clayey sandy SILT, **RESIDUAL SHALE**.

Moist to very wet, yellow to grey speckled black streaked brown to red, fine grained, **completely weathered**, massive to laminated, stiff to very stiff, extremely weak rock (ISRM 1981 rock class, <R0), **SHALE**.

Material properties:

Clay = 12%, Silt = 58%, Sand = 27%, Gravel = 2%, PI = 4.0%, LL = 26.3%, LS = 2.1%, GM = 0.33, SG = 2.661, VDM Heave Potential = Low, HRB class = A-4(2), USCS = ML.

Moist to very wet, yellow to grey speckled black streaked brown to red, fine grained, **completely to highly weathered**, massive to laminated, stiff to very weak rock (ISRM 1981 rock class, <R1), **SHALE**.

NOTES

1) Confined refusal with excavator.

2) Soft to intermediate excavation conditions at termination depth.

3) No water seepage encountered during time of investigation.

4) Sidewall stable during the time of inspection.

5) Disturbed sample retrieved at 1.50--2.50m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION : Vertical
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1463m
X-COORD : 28.90771
Y-COORD : -25.95024

HOLE No: TP01

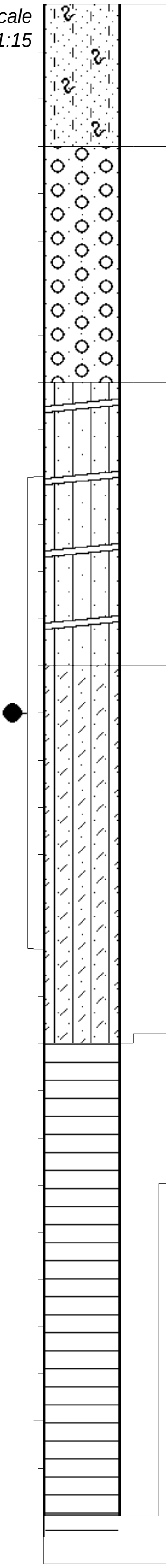
TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP02
Sheet 1 of 1

JOB NUMBER: RS22025



0.00	Moist, brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel to occasionally cobbles of mixed origin, **TOPSOIL**. Scattered roots.
0.30	Moist, brown, loose to occasionally dense, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded to rounded gravel to cobbles of mixed origin, **PEBBLE MARKER ZONE**.
0.80	Moist, yellow mottled red to black, firm to stiff to dense, **open structured to fissured to relict structured**, clayey gravelly sandy SILT with prominent slightly weathered to unweathered shale bands with moderately cemented Fe concretions, **REWORKED RESIDUAL SHALE**.
1.40	Moist, orange streaked red to grey blotched brown, stiff to dense, **relict to laminated to open structure**, clayey sandy SILT, **RESIDUAL SHALE**. **Note**: Occasional slightly to unweathered shale bands. **Material properties**: Clay = 7%, Silt = 18%, Sand = 31%, Gravel = 44%, PI = 13.6%, LL = 31.5%, LS = 6.7%, GM = 1.76, SG = 2.657, VDM Heave Potential = Low, HRB class = A-2-6(0), USCS = SC.
2.20	Moist, light brown streaked black to red, fine grained, **completely to highly weathered**, massive to laminated, stiff to extremely weak to very weak rock (ISRM 1981 rock class, R0 to R1), **SHALE**. **Note**: Unweathered shale bands present.
3.20	As above but highly weathered.

NOTES

1) Confined refusal with excavator.

2) Intermediate excavation conditions at confined refusal depth.

3) No water seepage encountered during time of investigation.

4) Sidewall stable during the time of inspection.

5) Disturbed sample retrieved at 1.00--2.00m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1460m
X-COORD : 28.90911
Y-COORD : -25.95010

HOLE No: TP02

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

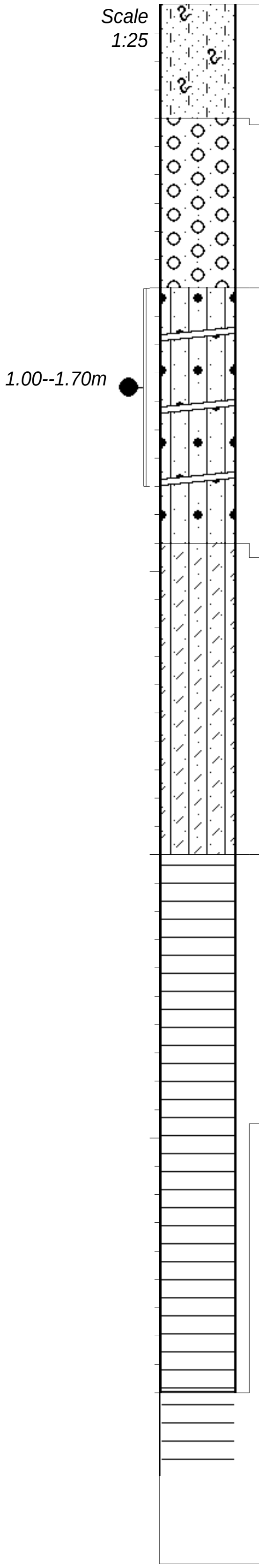
dot●PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP03
Sheet 1 of 1

JOB NUMBER: RS22025

Scale
1:25



0.00

Moist, dark brown, <u>loose to medium dense</u>, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel to cobbles of mixed origin, **TOPSOIL**. Scattered roots.

0.40

Moist, brown, <u>loose to occasionally dense</u>, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded to rounded gravel to cobbles to occasional small boulders of mixed origin, **PEBBLE MARKER ZONE**.

1.00

Moist, yellow mottled red to black, <u>firm</u>, **open structured to fissured to relict structured**, clayey gravelly sandy SILT with prominent slightly weathered to unweathered shale bands with moderately cemented Fe concretions, **REWORKED RESIDUAL SHALE**.

<u>Material properties</u>:

Clay = 5%, Silt = 15%, Sand = 37%, Gravel = 43%, PI = 11.0%, LL = 29.1%, LS = 5.4%, GM = 1.81, SG = 2.661, VDM Heave Potential = Low, HRB class = A-2-6(0), USCS = SC.

1.90

Moist, orange streaked red to grey blotched brown, <u>firm to stiff</u>, **relict to laminated to open structure**, clayey sandy SILT, **RESIDUAL SHALE**.

<u>Note</u>: Occasional slightly to unweathered shale bands.

3.00

Moist to very wet, pink to yellow brown mottled grey, fine grained, **completely weathered**, massive to laminated, <u>firm to stiff</u>, extremely weak rock (ISRM 1981 rock class, <R0), **SHALE**.

<u>Note</u>: Unweathered shale bands present.

4.90

As above.

NOTES

1) Slow excavation near confined maximum reach.

2) Hole terminated close to maximum reach of excavator.

3) Soft excavation conditions at termination depth.

4) No water seepage encountered during time of investigation.

5) Sidewall stable during the time of inspection.

6) Disturbed sample retrieved at 1.00--1.70m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DATUM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1468m
X-COORD : 28.90977
Y-COORD : -25.95076

HOLE No: TP03

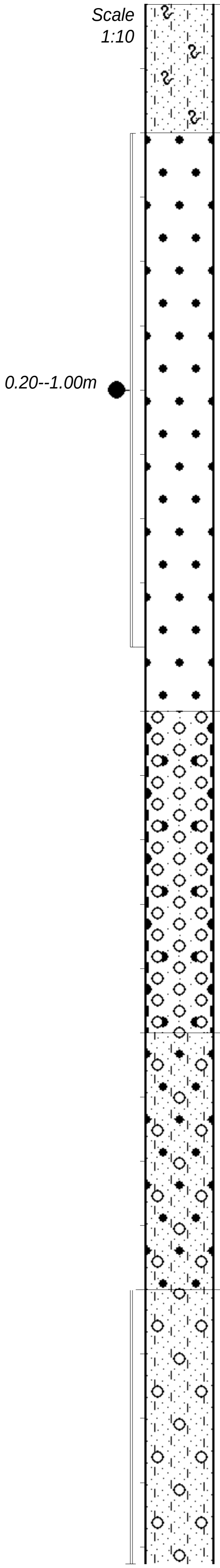
TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP04
Sheet 1 of 2

JOB NUMBER: RS22025



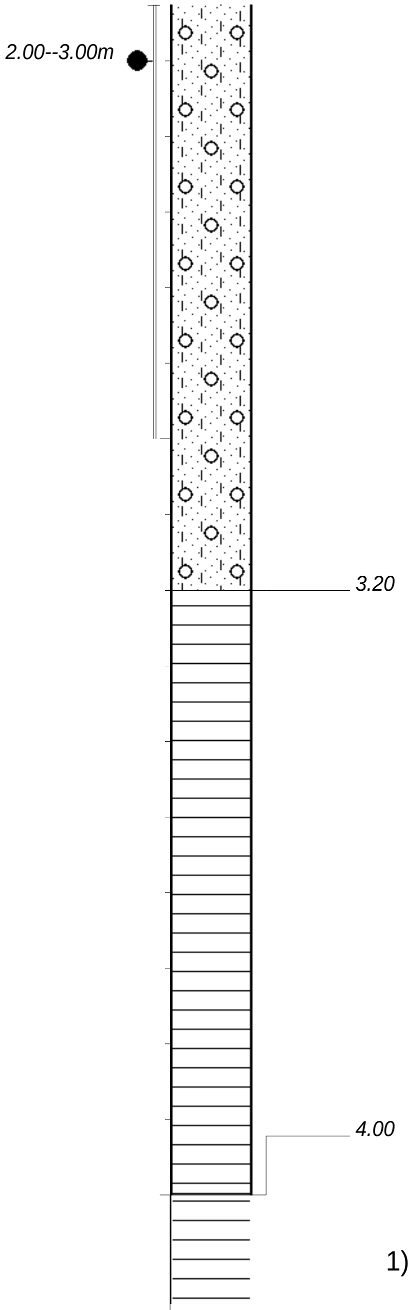
0.00	Moist, dark brown, <u>loose to medium dense</u>, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.
0.20	Moist, brown speckled black, <u>loose to medium dense</u>, **open structured to pinholed**, silty gravelly SAND to silty sandy GRAVEL with abundant amounts of sub-rounded gravel of mixed origin with minor amounts of Fe concretions, **PEBBLE MARKER ZONE**. **<u>Material properties</u>**: Clay = 3%, Silt = 7%, Sand = 17%, Gravel = 72%, PI = 9.2%, LL = 26.3%, LS = 5.3%, GM = 2.40, SG = 2.684, VDM Heave Potential = Low, HRB class = A-2-4(0), USCS = SP-SC.
1.10	Moist, red brown mottled black to orange, <u>dense to very dense</u>, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded gravel to cobble of mixed origin with moderately to strongly cemented ferricrete, **HONEYCOMB FERRICRETE**.
1.60	Moist, yellow mottled red to black, <u>stiff to dense</u>, **open structured to slightly shattered**, clayey gravelly silty SAND with minor amounts of sub-angular to rounded gravel to cobbles of mixed origin with weak to moderately cemented Fe concretions, **REWORKED RESUDAL DIAMICTITE**.
2.00	Moist, light brown mottled black to red, <u>stiff to firm</u>, **open structure to slightly fissured**, clayey silty gravelly SAND to clayey gravelly silty SAND with sub-angular to rounded gravel to cobbles to small boulders of mixed origin with occasional Fe concretions, **RESIDUAL DIAMICTITE**. **<u>Material properties</u>**: Clay = 7%, Silt = 22%, Sand = 40%, Gravel = 30%, PI = 10.2%, LL = 28.4%, LS = 5.3%, GM = 1.42, SG = 2.672, VDM Heave Potential = Low, HRB class = A-2-4(0), USCS = SC, MDD = 1 931kgm3, OMC = 12.8 %.

dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP04
Sheet 2 of 2

JOB NUMBER: RS22025



Moist, light brown streaked red to black, **highly weathered**, fine grained, massive to bedded, very stiff to very weak rock (ISRM 1981 rock class, <R1), **DIAMICTITE**.

As above.

NOTES

1) Confined refusal of excavator.

2) Two excavator buckets excavated.

3) Intermediate excavation conditions at refusal depth.

4) No water seepage encountered during time of investigation.

5) Sidewall stable during the time of inspection.

6) Disturbed sample retrieved at 0.20--1.00m.

7) Disturbed sample retrieved at 2.00--3.00m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1464m
X-COORD : 28.91050
Y-COORD : -25.95009

HOLE No: TP04

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

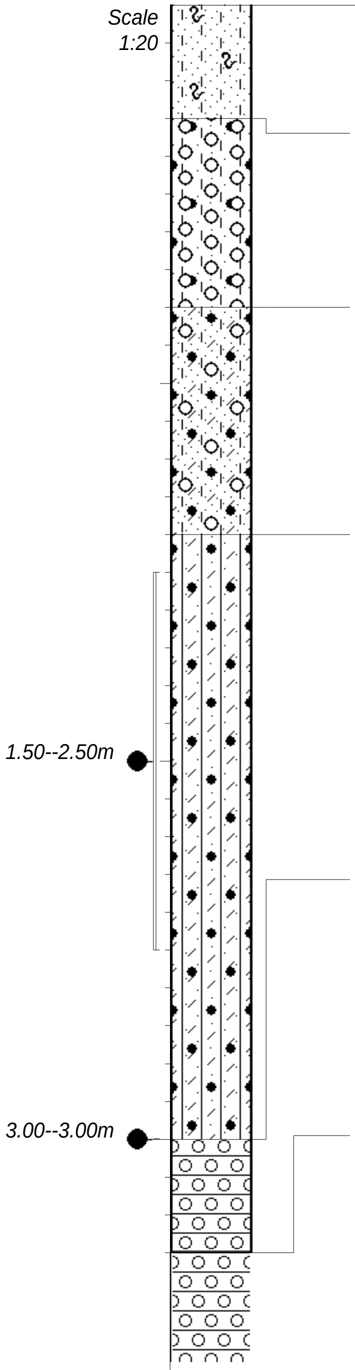
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dot●PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP05
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Moist, brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with sub-rounded to rounded gravel to occasionally cobbles of mixed origin, **TOPSOIL**. Scattered roots.

0.30

Moist, brown speckled black to yellow, loose to medium dense, **open structured**, clayey silty sandy GRAVEL with sub-angular to rounded gravel to cobbles of mixed origin and minor Fe concretions, **PEBBLE MARKER ZONE**.

0.80

Moist, red brown mottled black to yellow, firm, **slightly shattered to open structured to fissured**, clayey gravelly silty sand with completely weathered shale fragments with Fe concretions, **REWORKED RESIDUAL SHALE**.

1.40

Moist, red mottled yellow to black, stiff, **slightly shattered to relict**, clayey sandy SILT with bands of gravelly clayey sandy SILT and ferruginised in some places, **RESIDUAL SHALE**.

Material properties:

Clay = 8%, Silt = 25%, Sand = 38%, Gravel = 30%, PI = 13.4%, LL = 34.5%, LS = 7.1%, GM = 1.37, SG = 2.680, VDM Heave Potential = Low, HRB class = A-2-6(1), USCS = SC, MDD = 1 803kgm3, OMC = 15.4%.

3.00

Moist, light khaki brown speckled to mottled white to black, medium grained, **completely weathered**, massive, stiff to very stiff, extremely weak (ISRM 1981 rock class, <R0), **DIAMICTITE**.

Material properties:

Clay = 17%, Silt = 39%, Sand = 38%, Gravel = 6%, PI = 10.9%, LL = 30.9%, LS = 4.4%, GM = 0.68, SG = 2.590, VDM Heave Potential = Low, HRB class = A-6(4), USCS = CL.

3.30

As above.

NOTES

1) No refusal.

2) Maximum reach of TLB.

3) Soft excavation conditions at termination depth.

4) No water seepage encountered during time of investigation.

5) Sidewall stable during the time of inspection.

6) Disturbed sample retrieved at 1.50--2.50m.

7) Disturbed sample retrieved at 3.00--3.00m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1469m
X-COORD : 28.91105
Y-COORD : -25.95083

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

HOLE No: TP05

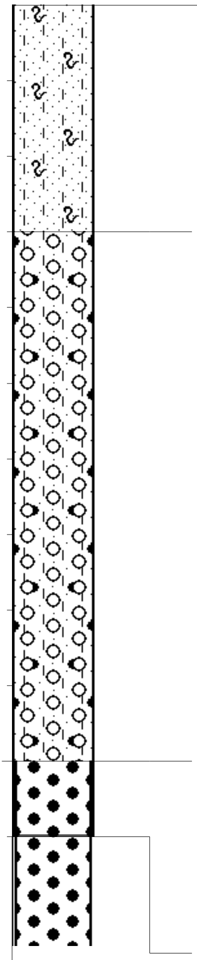
dot PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP06
Sheet 1 of 1

JOB NUMBER: RS22025

Scale
1:10



0.00	Moist, dark brown, <u>loose to medium dense</u>, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.
0.30	Moist, brown speckled black, <u>loose to medium dense</u>, **pinholed**, silty gravelly SAND to silty sandy GRAVEL with abundant amounts of sub-rounded gravel of mixed origin and Fe concretions, **PEBBLE MARKER ZONE**.
1.00	Moist, red brown mottled black to orange, <u>dense to very dense</u>, **honeycomb**, clayey silty sandy GRAVEL abundant amounts of sub-rounded gravel to cobble of mixed origin with minor amounts of strongly cemented ferricrete, **HONEYCOMB TO HARDPAN FERRICRETE**.
1.10	As above.

NOTES

1) Refusal with excavator.

2) Two excavator buckets excavated.

3) Intermediate to hard excavation conditions at refusal depth.

4) No water seepage encountered during time of investigation.

5) Sidewall stable during the time of inspection.

6) No sample retrieved.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1467m
X-COORD : 28.91168
Y-COORD : -25.95010

HOLE No: TP06

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

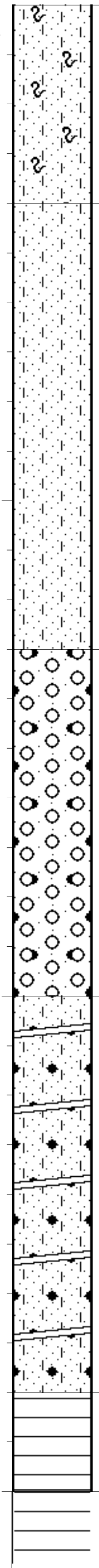
dot PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP07
Sheet 1 of 1

JOB NUMBER: RS22025

Scale
1:15



0.00	Moist, dark brown, <u>loose to medium dense</u>, **pinholed to voided**, gravelly silty SAND with minor amounts of sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.
0.40	Moist, reddish brown, <u>loose</u>, **pinholed to voided**, gravelly clayey silty SAND with abundant amounts of sub-rounded gravel of mixed origin, **COLLUVIUM**.
1.30	Moist, yellow brown mottled red, <u>dense</u>, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded gravel to cobble of mixed origin with minor amounts of weakly to strongly cemented nodular Fe concretions, **PEBBLE MARKER ZONE**.
2.00	Moist, yellow brown purple mottled red, <u>firm to stiff</u>, **open structured to fissured to relict structured**, clayey silty SAND with prominent slightly weathered to unweathered shale bands with moderately cemented Fe concretions, **REWORKED RESIDUAL SHALE**.
2.80	Moist to very wet, pink to yellow brown mottled grey, fine grained, **completely weathered**, massive to laminated, <u>firm to stiff</u>, extremely weak rock (ISRM 1981 rock class, <R0), **SHALE**.
3.00	As above.

NOTES

1) Easy to intermediate excavation conditions down to termination depth.

2) Soft excavation conditions at termination depth.

3) Hole terminated close to maximum reach of TLB.

4) No water seepage encountered during time of investigation.

5) No sidewall instability during time of investigation.

6) No sample retrieved.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1465m
X-COORD : 28.91296
Y-COORD : -25.95009

HOLE No: TP07

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

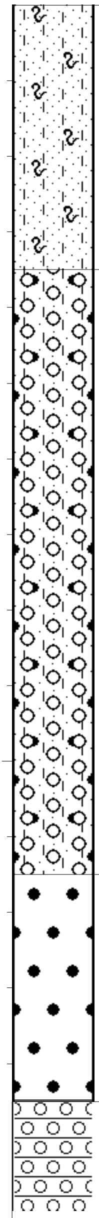
dot PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP08
Sheet 1 of 1

JOB NUMBER: RS22025

Scale
1:10



Depth	Description
0.00	Moist, dark brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.
0.35	Moist, brown speckled black, loose to medium dense, **pinholed**, silty gravelly SAND to silty sandy GRAVEL with abundant amounts of sub-rounded gravel of mixed origin with minor amounts of Fe concretions, **PEBBLE MARKER ZONE**.
1.15	Moist, light brown mottled to blotched red to yellow to black, dense to very dense, **honeycomb**, moderately to strongly cemented ferricrete, **HONEYCOMBED TO HARDPAN FERRICRETE**.
1.45	Moist, light brown mottled to blotched red to yellow to black, very dense, **open structured**, strongly cemented ferricrete with diamictite fragments, **HARDPAN TO MODERATELY WEATHERED DIAMICTITE**.

NOTES

1) Refusal.

2) Slow excavation conditions down to refusal depth.

3) Intermediate to hard excavation conditions at refusal depth.

4) No water seepage encountered during time of investigation.

5) Sidewall stable during the time of inspection.

6) No sample retrieved.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1469m
X-COORD : 28.91365
Y-COORD : -25.95026

HOLE No: TP08

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

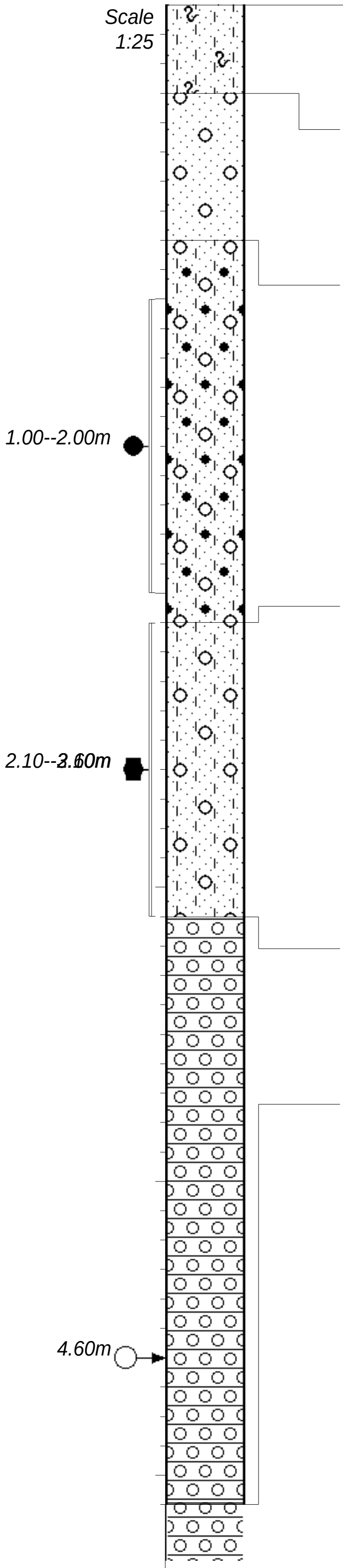
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dot●PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP09
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Moist, dark brown, <u>loose to medium dense</u>, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.

0.30

Moist, brown speckled black, <u>loose to medium dense</u>, pinholed**, silty gravelly SAND to silty sandy GRAVEL with abundant amounts of sub-rounded gravel to cobbles of mixed origin,** PEBBLE MARKER ZONE**.**

0.80

Moist, red mottled yellow to black, <u>firm</u>, **open structured to slightly shattered**, clayey gravelly silty SAND with minor amounts of sub-angular to rounded gravel to cobbles of mixed origin with weak to moderately cemented Fe concretions, **REWORKED RESUDAL DIAMICTITE**.

<u>Material properties</u>:

Clay = 16%, Silt = 23%, Sand = 36%, Gravel = 25%, PI = 17.5%, LL = 37.5%, LS = 8.6%, GM = 1.18, SG = 2.667, VDM Heave Potential = Medium, HRB class = A-6(3), USCS = SC, MDD = 1 858kgm3, OMC = 14.6%.

2.10

Moist, light brown to pink brown speckled black grey mottled red, <u>stiff</u>, **relict to slightly shattered**, clayey silty gravelly SAND to clayey gravelly silty SAND with sub-angular to rounded gravel to cobbles to small boulders of mixed origin with occasional Fe concretions, **RESIDUAL DIAMICTITE**.

<u>Material properties</u>:

Clay = 22%, Silt = 37%, Sand = 26%, Gravel = 14%, PI = 14.9%, LL = 37.6%, LS = 7.1%, GM = 0.73, SG = 2.631, VDM Heave Potential = Medium, HRB class = A-6(7), USCS = Cl, MDD = 1 758kgm3, OMC = 15.9%.

3.10

Moist, light brown stained black to red, fine grained, **completely weathered**, massive to bedded, <u>stiff</u>, extremely weak rock (ISRM 1981 rock class, <R0), **DIAMICTITE**.

<u>Note</u>: Seepage water flow rate at approximately 2l/min.

5.10

As above.

NOTES

1) No refusal.

2) Soft excavation conditions at termination depth.

3) Water seepage at 4.60m during time of investigation.

4) Sidewall stable during the time of inspection.

5) Undisturbed sample retrieved at 2.60m.

6) Disturbed sample retrieved at 1.00--2.00m.

7) Disturbed sample retrieved at 2.10--3.10m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1478m
X-COORD : 28.91433
Y-COORD : -25.95151

HOLE No: TP09

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

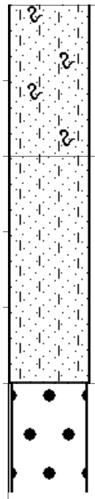
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP10
Sheet 1 of 1

JOB NUMBER: RS22025

Scale 1:10



0.00

Moist, dark brown, <u>loose to medium dense</u>, **pinholed to voided**, gravelly clayey silty SAND with minor amounts of angular to sub-rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.

0.20

Moist, reddish brown, <u>loose</u>, **pinholed to voided**, gravelly clayey silty SAND with abundant amounts of sub-rounded gravel of mixed origin, **COLLUVIUM**.

0.50

Very moist to wet, red brown stained yellow mottled black, overall consistency is <u>dense to very dense</u>, strongly cemented ferricrete, **HONEYCOMBED TO HARDPAN FERRICRETE**.

NOTES

1) TLB refusal

2) Intermediate to hard excavation conditions at refusal depth.

3) No water seepage encountered during time of investigation.

4) No sidewall instability during time of investigation.

5) No sample retrieved.

CONTRACTOR : EPCM
INCLINATION :
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DIAM :
DRILLED BY : RockSoilConsult (Pty) Ltd
DATE : 2022-05-25
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff
DATE : 2022-05-25
TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

ELEVATION : 1471m
X-COORD : 28.91511
Y-COORD : -25.95054

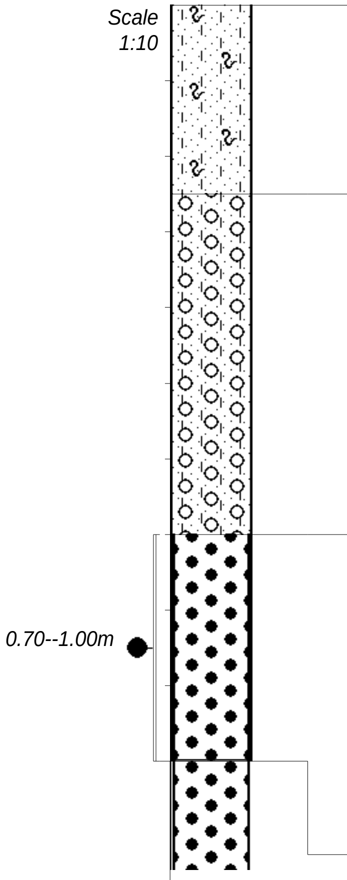
HOLE No: TP10

dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP11
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Moist, dark brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.

0.25

Moist, brown speckled black, loose to medium dense, **pinholed**, silty gravelly SAND to silty sandy GRAVEL with abundant amounts of sub-rounded gravel to cobbles to occasionally small boulders of mixed origin, **PEBBLE MARKER ZONE**.

0.70

Moist, light brown mottled to blotched red to yellow to black, dense to very dense, **honeycomb**, abundant amounts of moderately to strongly cemented honeycombed to hardpan ferricrete, **HONEYCOMBED TO HARDPAN FERRICRETE**.

Material properties:

Clay = 2%, Silt = 9%, Sand = 22%, Gravel = 67%, PI = 6.8%, LL = 25.7%, LS = 3.0%, GM = 2.32, SG = 2.624, VDM Heave Potential = Low, HRB class = A-2-4(0), USCS = GM & GC, MDD = 2 056kgm3, OMC = 12.5%.

1.00

As above but very dense, **HARDPAN FERRICRETE**.

NOTES

1) Excavator refusal.

2) Two bucket width excavation.

3) Intermediate to hard excavation conditions at refusal depth.

4) No water seepage encountered during time of investigation.

5) Sidewall stable during the time of inspection.

6) Disturbed sample retrieved at 0.70--1.00m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1468m
X-COORD : 28.91618
Y-COORD : -25.95063

HOLE No: TP11

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

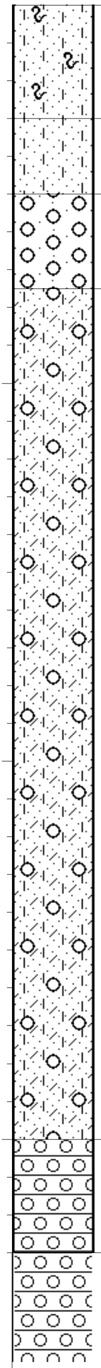
dot PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP12
Sheet 1 of 1

JOB NUMBER: RS22025

Scale
1:20



0.00	Moist, dark brown, <u>loose to medium dense</u>, **pinholed to voided**, gravelly clayey silty SAND with minor amounts of sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.
0.30	Moist, reddish brown, <u>loose</u>, **pinholed to voided**, gravelly clayey silty SAND with abundant amounts of sub-rounded to angular gravel of mixed origin, **COLLUVIUM**.
0.50	Moist, brown orange stained yellow, <u>medium dense</u>, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded gravel to cobble of mixed origin, **PEBBLE MARKER ZONE**.
0.75	Moist, orange brown speckled white and black, <u>loose to medium dense</u>, **fissured to open structured**, gravelly clayey silty SAND with abundant amounts of sub-rounded cobble to boulders of mixed origin, **REWORKED RESIDUAL DIAMICTITE TO COMPLETELY WEATHERED DAMICTITE**.
3.00	Moist, light brown stained yellow grey, <u>dense</u>, **fissured to cross bedded to open structured**, clayey silty SAND with clast present, **COMPLETELY WEATHERED DAMICTITE**.
3.30	As above.

NOTES

1) Easy excavation conditions down to termination depth.
2) Soft excavation conditions at termination depth.
3) Hole terminated close to maximum reach of TLB.
4) No water seepage encountered during time of investigation.
5) No sidewall instability during time of investigation.
6) No sample retrieved.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1480m
X-COORD : 28.91725
Y-COORD : -25.95151

HOLE No: TP12

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

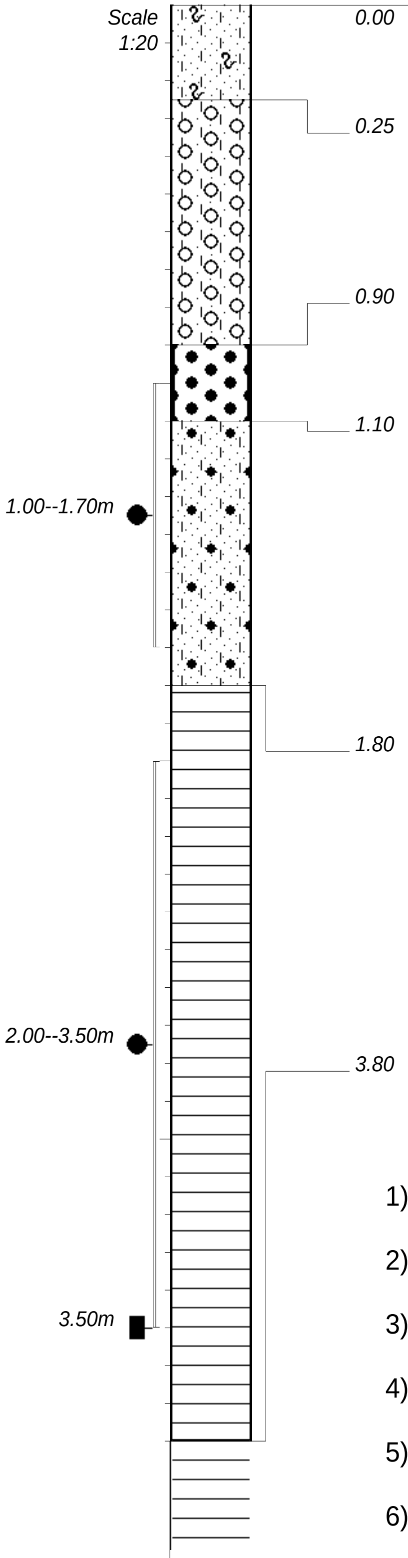
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP13
Sheet 1 of 1

JOB NUMBER: RS22025



Moist, dark brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.

Moist, brown speckled black, loose to medium dense, **pinholed**, silty gravelly SAND to silty sandy GRAVEL with abundant amounts of sub-rounded gravel to cobbles to occasionally small boulders of mixed origin, **PEBBLE MARKER ZONE**.

Moist, light brown mottled to blotched red to yellow to black, dense to very dense, **honeycomb**, strongly cemented ferricrete, **HARDPAN FERRICRETE**.

Moist, brown to light brown mottled purple to black, firm to stiff, **shattered to open structured to fissured**, clayey silty SAND with minor amounts of completely weathered shale fragments with minor amounts of Fe concretions, **REWORKED RESIDUAL SHALE**.

Material properties:

Clay = 8%, Silt = 21%, Sand = 37%, Gravel = 34%, PI = 14.8%, LL = 32.7%, LS = 7.5%, GM = 1.51, SG = 2.707, VDM Heave Potential = Low, HRB class = A-2-6(1), USCS = SC, MDD = 1 881kgm3, OMC = 15.0%.

Slightly moist, purple to brown streaked red black to orange light brown, fine grained, **completely weather to highly weathered**, massive to bedded to laminated, stiff to extremely weak rock (ISRM 1981 rock class, <R0), **SHALE**.

Material properties:

Clay = 8%, Silt = 19%, Sand = 21%, Gravel = 53%, PI = 13.8%, LL = 38.7%, LS = 6.6%, GM = 1.85, SG = 2.736, VDM Heave Potential = Low, HRB class = A-2-6(1), USCS = GC, MDD = 1 833kgm3, OMC = 16.8%.

As above but **highly weathered** with prominent residual zones.

NOTES

1) Highly undulating soil horizons.
2) Confined refusal.
3) Soft to intermediate excavation conditions at confined refusal depth.
4) No water seepage encountered during time of investigation.
5) Sidewall stable during the time of inspection.
6) Disturbed sample retrieved at 1.00--1.70m.
7) Disturbed sample retrieved at 2.00--3.50m.
8) Undisturbed sample retrieved at 3.50m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1475m
X-COORD : 28.91846
Y-COORD : -25.95072

HOLE No: TP13

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

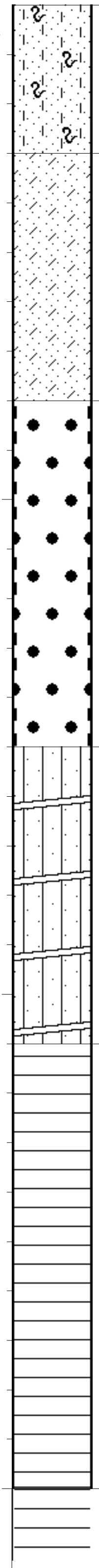
dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP14
Sheet 1 of 1

JOB NUMBER: RS22025

Scale
1:15



0.00

Moist, dark brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with minor amounts of sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.

0.30

Moist, light yellow brown, loose to medium dense, **pinholed to voided**, gravelly silty clayey SAND with minor amounts of sub-rounded gravel of mixed origin, **COLLUVIUM**.

0.80

Moist, yellow brown mottled red and black, dense to very dense, **open structured**, clayey silty sandy GRAVEL, **HONEYCOMBED FERRICRETE**.

1.50

Moist, yellow brown purple, firm, **open structured to fissured to relict structured**, clayey sandy silt with completely weathered patches of shale, **RESIDUAL SHALE**.

2.10

Moist, purple, fine grained, **completely weathered to highly weathered**, massive to laminated, firm to stiff, extremely weak to very weak rock (ISRM 1981 rock class, R0 to R1), **SHALE**.

3.00

As above.

NOTES

1) Easy to intermediate excavation conditions down to termination depth.
2) Soft to intermediate excavation conditions at termination depth.
3) Hole terminated close to maximum reach of TLB.
4) No water seepage encountered during time of investigation.
5) No sidewall instability during time of investigation.
6) No sample retrieved.

CONTRACTOR : EPCM
INCLINATION :
ELEVATION : 1474
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DIAM :
X-COORD : 28.91990
DRILLED BY : RockSoilConsult (Pty) Ltd
DATE : 2022-05-25
Y-COORD : -25.95057
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff
DATE : 2022-05-25

HOLE No: TP14

TYPE SET BY : JI Roux
DATE : 08/11/2022 14:42
SETUP FILE : STANDARD.SET
TEXT : ..s\RS22025SP(V1.4)Lab.txt

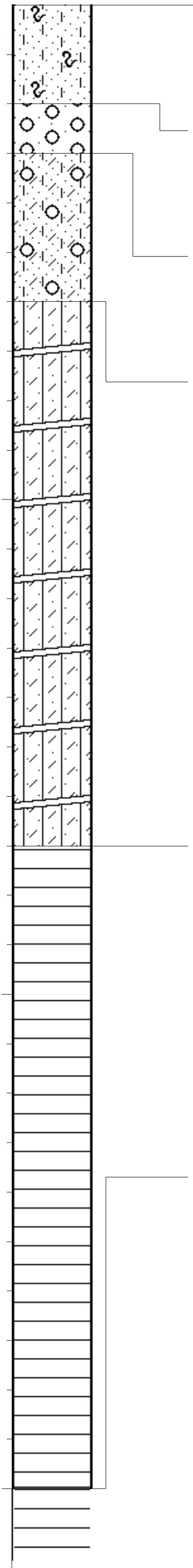
dot●PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP19
Sheet 1 of 1

JOB NUMBER: RS22025

Scale 1:15



0.00	Moist, dark brown, <u>loose to medium dense</u>, **pinholed to voided**, gravelly silty SAND with minor amounts of sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.
0.20	Moist, dark brown, <u>loose to occasionally dense</u>, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded to rounded gravel to cobble of mixed origin, **PEBBLE MARKER ZONE**.
0.30	Moist, reddish brown, <u>soft to firm</u>, **fissured to open structured**, gravelly clayey silty SAND with abundant amounts of completely weathered shale, **REWORKED RESIDUAL SHALE**.
0.60	Moist, reddish brown stained yellow, <u>firm</u>, **open structure to fissured**, clayey sandy SILT with unweathered bands of spheroidal weathered shale, **RESIDUAL SHALE**
1.70	Moist, yellow light brown stained black, fine grained, **completely weathered**, laminated, <u>firm to stiff</u>, extremely weak rock (ISRM 1981 rock class, <R0), **SHALE**.
3.00	As above.

NOTES

1) Easy excavation conditions down to termination depth.

2) Soft excavation conditions at termination depth.

3) Hole terminated close to maximum reach of TLB.

4) No water seepage encountered during time of investigation.

5) No sidewall instability during time of investigation.

6) No sample retrieved.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1487m
X-COORD : 28.91880
Y-COORD : -25.95262

HOLE No: TP19

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

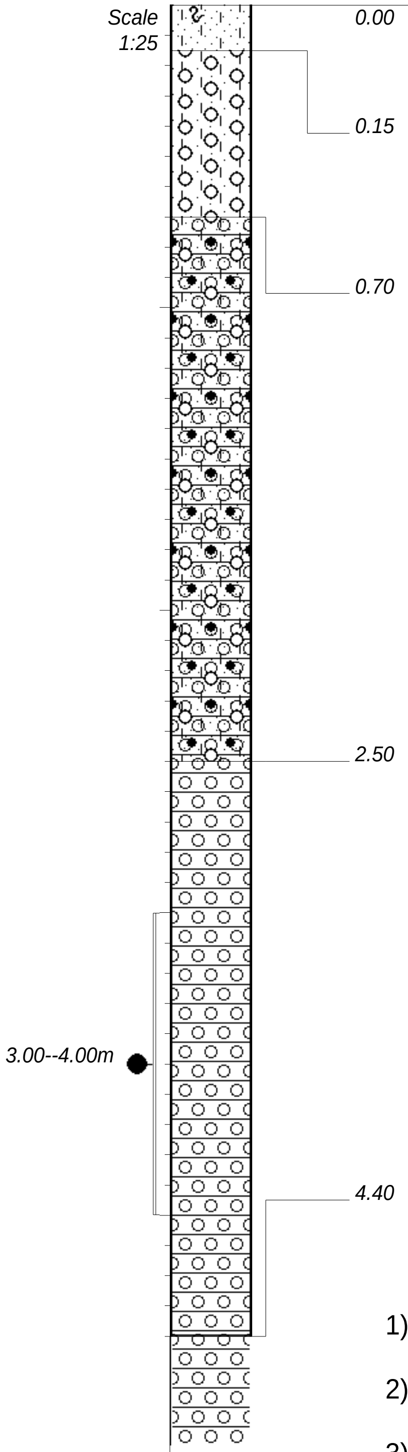
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP20
Sheet 1 of 1

JOB NUMBER: RS22025



Moist, dark brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.

Moist, brown speckled black, medium dense, **open structured to pinholed**, silty gravelly SAND to silty sandy GRAVEL with abundant amounts of sub-angular gravel to cobbles of mixed origin, **PEBBLE MARKER ZONE**.

Moist, red mottled yellow to black, stiff to firm, **open structured to slightly shattered to fissured**, clayey gravelly silty SAND with minor amounts of sub-angular to rounded gravel to occasionally cobbles of mixed origin with weak to moderately cemented Fe concretions, **REWORKED RESUDAL DIAMICTITE**.

Moist, light brown, fine grained, **completely weathered**, massive to relict, stiff, extremely weak rock (ISRM 1981 rock class, <R0), **DIAMICTITE**.

Material properties:

Clay = 5%, Silt = 24%, Sand = 70%, Gravel = 1%, PI = 4.1%, LL = 23.5%, LS = 1.5%, GM = 0.76, SG = 2.617, VDM Heave Potential = Low, HRB class = A-2-4(0), USCS = SM & SC, MDD = 1 789kgm3, OMC = 14.3%.

As above.

NOTES

1) No refusal.

2) Slow excavation near maximum reach.

3) Soft to intermediate excavation conditions at termination depth.

4) No water seepage during time of investigation.

5) Sidewall stable during the time of inspection.

6) Disturbed sample retrieved at 3.00--4.00m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1483m
X-COORD : 28.91595
Y-COORD : -25.95244

HOLE No: TP20

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

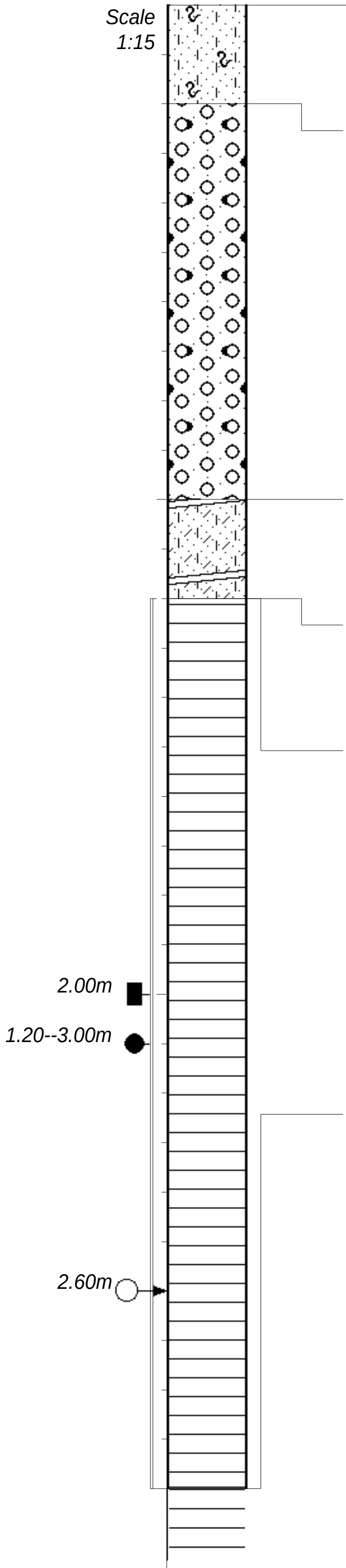
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP21
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Moist, dark brown, loose to medium dense, **pinholed to voided**, gravelly clayey silty SAND with minor amounts of angular to sub-rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.

0.20

Moist, reddish brown, medium dense to occasionally dense, **open structured**, clayey silty sandy GRAVEL with abundant amounts of sub-rounded gravel to cobble of mixed origin with minor amounts of Fe concretions, **SLIGHTLY FERRUGINISED PEBBLE MARKER ZONE**.

1.00

Moist, reddish brown stained yellow, firm, **open structured to fissured to relict structured**, clayey silty SAND with minor amounts of angular completely weathered shale, **REWORKED RESIDUAL SHALE**.

1.20

Moist, light yellowish brown, firm to stiff, **open structured to fissured to relict** , silty sandy silt with patches of completely weathered shale, **RESIDUAL SHALE**.

1.20

Moist, yellow light brown stained black, fine grained, **completely weathered**, laminated, firm to stiff, extremely weak rock (ISRM 1981 rock class, <R0), **SHALE**.

Material properties:

Clay = 15%, Silt = 61%, Sand = 12%, Gravel = 12%, PI = 9.1%, LL = 34.1%, LS = 5.0%, GM = 0.49, SG = 2.627, VDM Heave Potential = Low, HRB class = A-4(7), USCS = ML, MDD = 1 751kgm3, OMC = 16.9%.

3.00

As above.

NOTES

1) Easy excavation conditions down to termination depth.

2) Soft excavation conditions at termination depth.

3) Hole terminated close to maximum reach of TLB.

4) Water seepage at 2.60m encountered during time of investigation.

5) No sidewall instability during time of investigation.

6) Two disturbed samples retrieved at 1.20--3.00m.

7) Undisturbed sample retrieved at 2.00m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1478m
X-COORD : 28.91268
Y-COORD : -25.95213

HOLE No: TP21

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

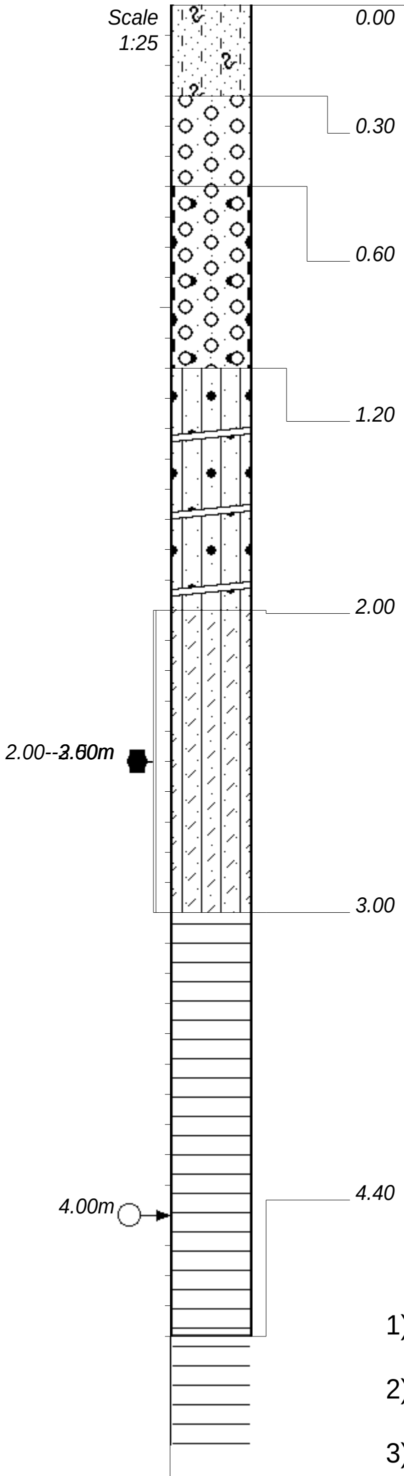
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dot PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP22
Sheet 1 of 1

JOB NUMBER: RS22025



Moist, dark brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel to cobbles of mixed origin, **TOPSOIL**. Scattered roots.

Moist, brown, loose to occasionally dense, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded to rounded gravel to occasional cobbles of mixed origin, **PEBBLE MARKER ZONE**.

Moist, red brown mottled black to orange, dense, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded gravel to cobble of mixed origin with minor amounts of weakly to strongly cemented Fe concretions, **SLIGHTLY FERRUGINISED PEBBLE MARKER ZONE**.

Moist, yellow mottled red to black, firm, **open structured to fissured to relict structured**, clayey gravelly sandy SILT with prominent slightly weathered to unweathered shale bands with moderately cemented Fe concretions, **REWORKED RESIDUAL SHALE**.

Note: medium dense to dense in places.

Moist, orange streaked red to grey blotched brown, firm to stiff, **relict to laminated to open structure**, clayey sandy SILT, **RESIDUAL SHALE**.

Note: Occasional slightly to unweathered shale bands.

Material properties:

Clay = 19%, Silt = 54%, Sand = 14%, Gravel = 12%, PI = 12.3%, LL = 39.5%, LS = 23.7%, GM = 0.55, SG = 2.631, VDM Heave Potential = Low, HRB class = A-6 (9), USCS = ML.

Moist to very wet, pink to yellow brown mottled grey, fine grained, **completely weathered**, massive to laminated, firm to stiff, extremely weak rock (ISRM 1981 rock class, <R0), **SHALE**.

Note: Unweathered shale band at 4.00 â ô 4.15m.

Note: Seepage water flow rate at approximately 5l/min.

As above.

NOTES

1) Hole terminated because of slow excavation down to confined refusal depth.

2) Soft excavation conditions at termination depth.

3) Water seepage encountered at 4.00m.

4) Sidewall stable during the time of inspection.

5) Disturbed sample retrieved at 2.00--3.00m.

6) Undisturbed sample retrieved at 2.50m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

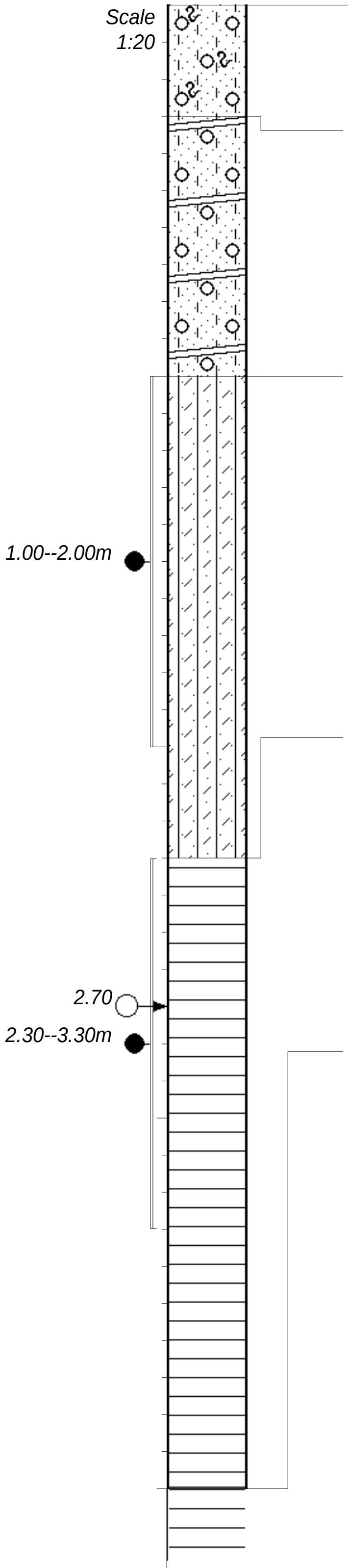
INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1470m
X-COORD : 28.91025
Y-COORD : -25.95198

HOLE No: TP22

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt



0.00

Moist, dark brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with occasional sub-angular to rounded gravel to cobbles of mixed origin, **TOPSOIL**. Scattered roots.

0.30

Moist, red brown mottled yellow, medium dense to dense to firm to stiff, **open to fissured to slightly shattered**, clayey gravelly silty SAND with purple banded slightly weathered shale in places, **REWORKED RESIDUAL SHALE**.

1.00

Moist, orange streaked red to grey blotched brown, firm to stiff, **relict to laminated to open structured**, clayey sandy SILT, **RESIDUAL SHALE**.

Material properties:

Clay = 9%, Silt = 46%, Sand = 25%, Gravel = 20%, PI = 12.8%, LL = 34.5%, LS = 6.1%, GM = 0.96, SG = 2.645, VDM Heave Potential = Low, HRB class = A-6 (5), USCS = CL, MDD = 1 698kgm3, OMC = 18.1%.

2.30

Moist to wet, brown to yellow streaked black to brown, fine grained, **completely weathered**, laminated and massive, stiff, extremely weak rock (ISRM 1981 rock class, S4), **SHALE.**

Note**: Seepage water flow rate at approximately 12l/min.**

Material properties**:**

Clay = 10%, Silt = 66%, Sand = 15%, Gravel = 9%, PI = 9.0%, LL = 82.9%, LS = 4.6%, GM = 0.43, SG = 2.659, VDM Heave Potential = Low, HRB class = A-4(6), USCS = ML, MDD = 1 680kgm3, OMC = 18.4%.

4.00

Moist to wet, brown to yellow streaked black to brown, fine grained, **highly weathered**, laminated and massive, stiff to very weak rock (ISRM 1981 rock class, <R1), **SHALE.**

NOTES

1) Confined refusal.
2) Slow excavation down to termination depth.
3) Intermediate excavation conditions at confined refusal depth.
4) Water seepage encountered at 2.70 during time of inspection.
5) No sidewall instability during time of investigation
6) Two disturbed sample retrieved at 1.00--2.00m.
7) Two disturbed sample retrieved at 2.30--3.30m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1469m
X-COORD : 28.90850
Y-COORD : -25.95326

HOLE No: TP23

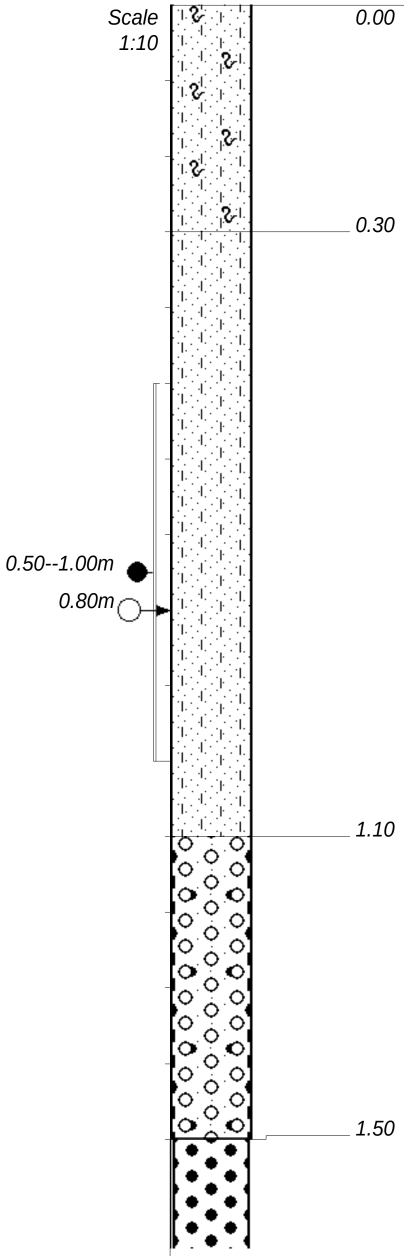
TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP25
Sheet 1 of 1

JOB NUMBER: RS22025



Moist, brown, loose to medium dense, **pinholed to voided**, silty SAND with occasional sub-rounded to rounded gravel to cobbles of mixed origin, **TOPSOIL**. Scattered roots.

Moist, yellow brown, very loose to loose, **pinholed to voided**, gravelly silty SAND with minor amounts of sub-rounded gravel of mixed origin, **COLLUVIUM**.

Note: Seepage water flow rate at approximately 30l/min.

Material properties:

Clay = 5%, Silt = 18%, Sand = 77%, Gravel = 0%, PI = 2.3%, LL = 17.3%, LS = 0.9%, GM = 0.95, SG = 2.625, VDM Heave Potential = Low, HRB class = A-2-4(0), USCS = SM, MDD = 2 026kgm3, OMC = 9.3%.

Wet, brown mottled black to orange, dense, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded gravel to cobble of mixed origin with minor amounts of weakly to strongly cemented Fe concretions, **SLIGHTLY FERRUGINISED PEBBLE MARKER ZONE**.

Very moist to wet, red mottled black to brown, dense to very, strongly cemented Fe concretions with a matrix of moist to wet, light brown, clayey silty SAND, **HARDPAN FERRICRETE**.

NOTES

1) Two buckets width excavation.

2) Refusal.

3) Intermediate to hard excavation conditions at termination depth.

4) Water seepage encountered at 0.80m during time of inspection.

5) Sidewall unstable, thus did not enter the test pit

6) Two disturbed sample retrieved at 0.50--1.00m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1464m
X-COORD : 28.90393
Y-COORD : -25.95224

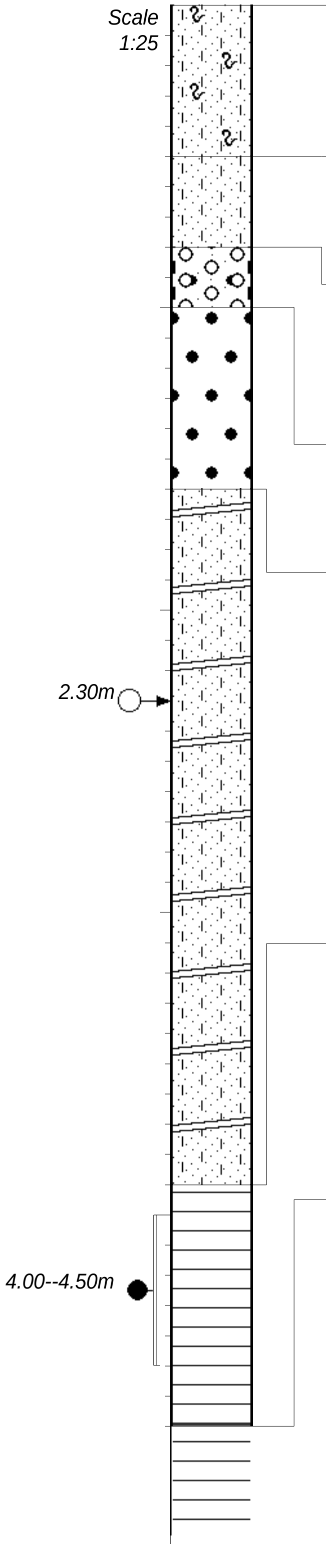
HOLE No: TP25

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP26
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Moist, brown, <u>loose to medium dense</u>, **pinholed to voided**, silty SAND with occasional sub-rounded to rounded gravel to cobbles of mixed origin, **TOPSOIL**. Scattered roots.

0.50

Moist, yellow brown, <u>very loose to loose</u>, **pinholed to voided**, gravelly silty SAND with minor amounts of sub-rounded gravel of mixed origin, **COLLUVIUM**.

0.80

Moist, brown mottled black to orange, <u>dense</u>, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded gravel to cobble of mixed origin with minor amounts of weakly to strongly cemented Fe concretions, **SLIGHTLY FERRUGINISED PEBBLE MARKER ZONE**.

1.00

Very moist to wet, red mottled black to brown, <u>dense to very dense</u>, moderately to strongly cemented Fe concretions with sub-rounded cobbles of mixed origin, **HONEYCOMB FERRICRETE**.

1.60

Very wet, orange grey brown stained yellow, <u>firm to stiff</u>, **open structured to fissured to relict structured**, gravelly clayey silty SAND with prominent highly weathered shale bands, **REWORKED RESIDUAL SHALE**.

<u>Note</u>: Seepage water flow rate at approximately 25l/min.

3.90

Moist to very moist, yellow mottled grey to red, <u>firm to stiff</u>, **shattered to fissured to more relict structured**, fine grained, **COMPLETELY WEATHERED SHALE**.

<u>Material properties</u>:

Clay = 15%, Silt = 16%, Sand = 17%, Gravel = 8%, PI = 14.5%, LL = 37.6%, LS = 7.0%, GM = 0.44, SG = 2.661, VDM Heave Potential = Medium, HRB class = A-6(10), USCS = CL.

4.70

As above.

NOTES

1) Two buckets width excavation.

2) Hole terminated close to maximum reach of excavator.

3) Soft to intermediate excavation conditions at termination depth.

4) Water seepage encountered at 2.30m during time of inspection.

5) Sidewall unstable, thus did not enter the test pit

6) Disturbed sample retrieved at 4.00--4.50m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1464m
X-COORD : 28.90280
Y-COORD : -25.95077

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

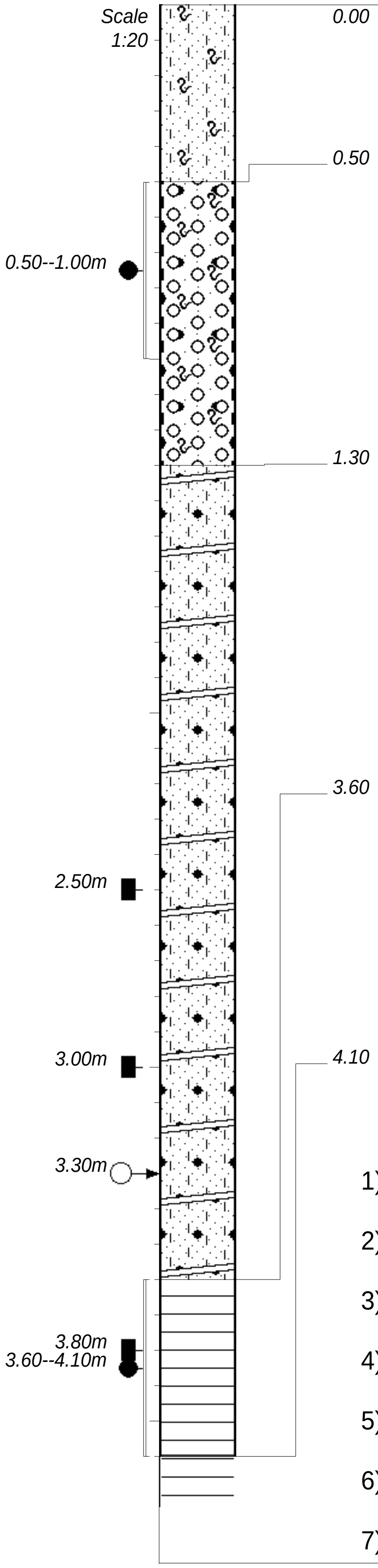
HOLE No: TP26

dot PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP29
Sheet 1 of 1

JOB NUMBER: RS22025



Moist, brown, loose, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel to cobbles of mixed origin, **TOPSOIL**. Scattered roots.

Moist, brown to orange brown speckled red, medium dense to dense, **open structured**, clayey silty sandy GRAVEL with abundant amounts of sub-rounded gravel to small boulders (<25cm) of mixed origin with weakly to strongly cemented Fe concretions, **FERRUGINISED PEBBLE MARKER ZONE**. Minor mounts of roots.

Material properties:

Clay = 6%, Silt = 21%, Sand = 52%, Gravel = 21%, PI = 8.0%, LL = 26.3%, LS = 4.4%, GM = 1.36, SG = 2.633, VDM Heave Potential = Low, HRB class = A-2-4(0), USCS = SC.

Moist, yellow mottled red to black, firm to stiff, **open structured to fissured to relict structured**, sandy clayey SILT to gravelly clayey silty SAND with prominent slightly weathered to unweathered shale bands with moderately cemented Fe concretions, **REWORKED RESIDUAL SHALE**.

Note: Seepage water flow rate at approximately 20l/min.

Material properties:

Clay = 23-31%, Silt = 29-42%, Sand = 21-25%, Gravel = 7-22%, PI = 15.9-18.6%, LL = 34-42.1%, LS = 7.5-10.0%, GM = 0.46-1.00, SG = 2.632-2.660, VDM Heave Potential = Medium, HRB class = A-7-6(7), USCS = CL.

Moist, white to yellow brown streaked red, firm to stiff **shattered to laminated to relict structured**, sandy clayey SILT, **RESIDUAL SHALE TO COMPLETELY WEATHERED SHALE**.

Material properties:

Clay = 26%, Silt = 61%, Sand = 12%, Gravel = 1%, PI = 18.5%, LL = 43.2%, LS = 8.8%, GM = 0.16, SG = 2.675, VDM Heave Potential = Medium, HRB class = A-7-6(18), USCS = CL, MDD = 1 677kgm3, OMC = 19.1%.

As above.

NOTES

1) Easy excavation down to termination depth.
2) Excavator terminated.
3) Water seepage encountered at 3.30m during time of inspection.
4) Sidewall unstable during the time of inspection.
5) Disturbed sample retrieved at 0.50--1.00m.
6) Two disturbed sample retrieved at 3.60--4.10m.
7) Undisturbed samples retrieved at 2.50m, 3.00m and 3.80m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1463m
X-COORD : 28.89899
Y-COORD : -25.94971

HOLE No: TP29

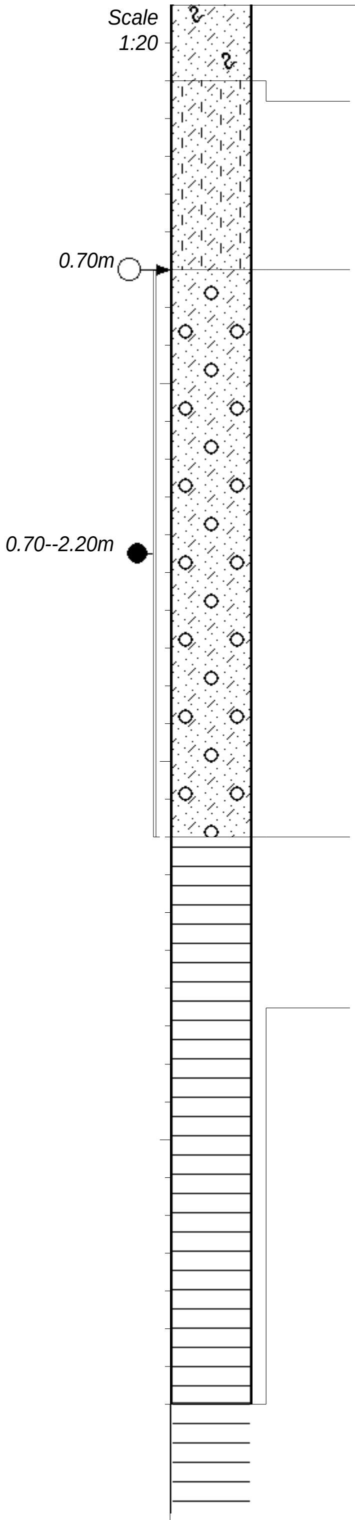
TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dot PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP42
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Moist to wet, dark brown, <u>very loose to loose</u>, **massive to open structured**, clayey SAND, **TOPSOIL**. Roots present.

0.20

Moist to wet, greyish brown, <u>very loose to loose</u>, **open structured**, clayey silty SAND, **ALLUVIUM**.

0.70

Slightly moist, grey stained yellow and occasionally red, <u>firm</u>, **micro-shattered**, clayey gravelly sand, **RESIDUAL TO COMPLETELY WEATHERED SHALE**.

<u>Note</u>: Seepage present in this horizon at 1l/min.

<u>Material properties</u>:

Clay = 16%, Silt = 17%, Sand = 37%, Gravel = 30%, PI = 14.4%, LL = 34.2%, LS = 7.8%, GM = 1.73, SG = 2.686, VDM Heave Potential = Low, HRB class = A-2-6(1), USCS = SC.

2.20

Slightly moist, purple brown, **highly weathered**, fine grained, laminated, weak (ISRM 1981 rock class, R2), **SHALE**.

<u>Note</u>: Black staining on joints.

3.70

As above.

NOTES

1) Excavator confined refusal at 3.70 mbgl.
2) Soft excavation conditions down to refusal depth.
3) Intermediate excavation conditions at refusal depth.
4) Water seepage present at 0.70m.
5) Disturbed samples retrieved at 0.70--2.20m.
6) Sidewalls unstable during time of investigation.
7) Sidewall collapsed during time of investigation.
8) Soil profile is undulating.
9) Test pit situated south west of a drainage feature.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022-05-25

ELEVATION : 1451m
X-COORD : 28.90223
Y-COORD : -25.94469

HOLE No: TP42

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

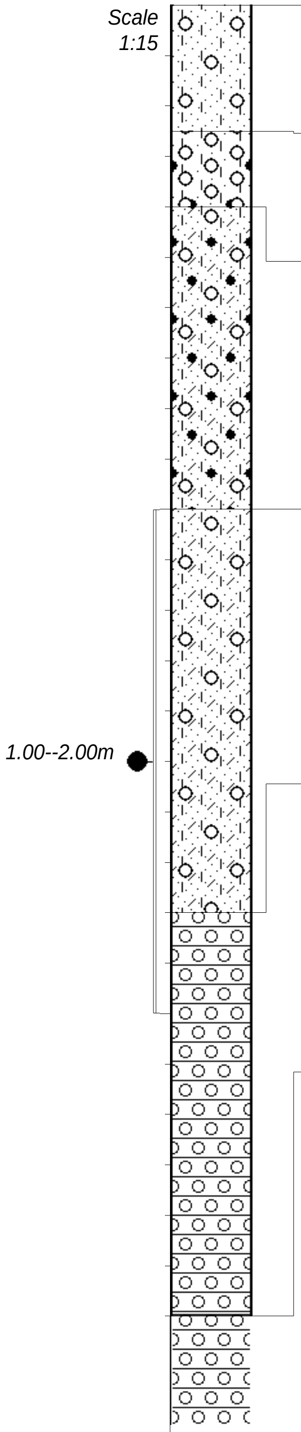
dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: **TP53**
Sheet 1 of 1

JOB NUMBER: **RS22025**

Scale
1:15



0.00

Slightly moist, greyish brown, medium dense, **pinholed to voided**, silty gravelly SAND with occasional subangular to rounded gravel to boulders of quartzite, **TOPSOIL**.

0.25

Slightly moist, greyish brown, medium dense, **pinholed to voided**, silty sandy GRAVEL with occasional subangular to rounded gravel to boulders of quartzite, **PEBBLE MARKER ZONE**.

0.40

Moist, brown mottled orange to yellow, firm to stiff, **open to fissured**, clayey gravelly silty fine SAND with subangular to rounded gravel of mixed origin to completely weathered diamictite, **REWORKED RESIDUAL DIAMICTITE**.

Note: Occasional Fe concretions present.

1.00

Moist, light brown mottled red to black and light brown, stiff, **open to slightly shattered and fissured**, gravelly clayey silty fine SAND with subrounded to rounded gravel of mixed origin, **REWORKED RESIDUAL DIAMICTITE**.

1.80

Moist, grey speckled black to white to red, **completely weathered**, fine to coarse grained, massive to relict structure, dense to very dense, extremely weak (ISRM 1981 rock class, <R0), **DIAMICTITE**.

Material properties:

Clay = 16%, Silt = 18%, Sand = 30%, Gravel = 36%, PI = 18.1%, LL = 41.9%, LS = 8.3%, GM = 1.45, SG = 2.597, VDM Heave Potential = Low, HRB class = A-2-7(2), USCS = SC, MDD = 1 766kgm3, OMC = 17.1%.

2.60

As above.

NOTES

1) TLB terminated near maximum depth.

2) Slow excavation down to terminated depth.

3) Soft excavation conditions at terminated depth.

4) Excavation started with 2 bucket widths and ended with 1 bucket width.

5) No seepage encountered during time of investigation.

6) No sidewall instability during time of investigation.

7) Disturbed sample retrieved at 1.00--2.00m.

8) Test pit near pollution control dam.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06

ELEVATION : 1465m
X-COORD : 28.89806
Y-COORD : -25.93560

HOLE No: **TP53**

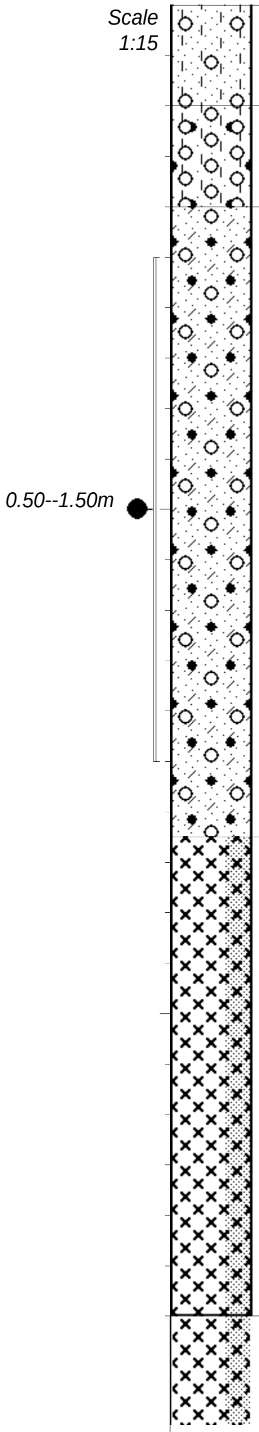
TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP54
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Slightly moist, greyish brown, <u>medium dense</u>, **pinholed to voided**, silty gravelly SAND with occasional subangular to rounded gravel to cobbles of quartzite, **TOPSOIL**.

0.20

Slightly moist, greyish brown, <u>medium dense</u>, **pinholed to voided**, silty sandy GRAVEL with occasional subangular to rounded gravel to cobbles of quartzite, **PEBBLE MARKER ZONE**.

0.40

Moist, red mottled black to brown, <u>firm to stiff</u>, **open to fissured and slightly shattered**, gravelly clayey SAND with subrounded gravel of mixed origin and highly weathered to unweathered corestones, **REWORKED RESIDUAL DIABASE** .

<u>Note</u>: Occasional Fe concretions present.

<u>Material properties</u>:

Clay = 10%, Silt = 27%, Sand = 35%, Gravel = 28%, PI = 18.4%, LL = 41.6%, LS = 8.9%, GM = 1.30, SG = 2.830, VDM Heave Potential = Low, HRB class = A-7-6(3), USCS = SC, MDD = 1 596kgm3, OMC = 23.2%.

1.65

Dry to slightly moist, light brown to grey speckled white, **highly weathered to unweathered**, fine grained, very closely to closely jointed, massive, very weak to moderately strong (ISRM 1981 rock class, R1 to R3), **DIABASE**.

<u>Note</u>: Brown, sandy silt gauge on joints up to 1cm thick.

2.60

As above.

NOTES

1) TLB confined refusal at 2.60m.
2) Intermediate excavation conditions at refusal depth.
3) No seepage encountered during time of investigation.
4) No sidewall instability during time of investigation.
5) Disturbed sample retrieved at 0.50--1.50m.
6) Soil profile highly undulating.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06

ELEVATION : 1461m
X-COORD : 28.89734
Y-COORD : -25.93532

HOLE No: TP54

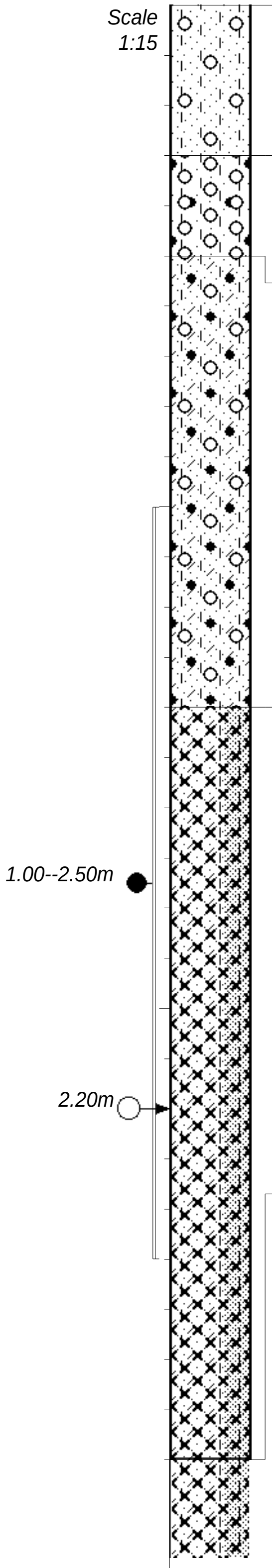
TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP55
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Slightly moist, greyish brown, medium dense, **pinholed to voided**, silty gravelly SAND with occasional subangular to rounded gravel to boulders of quartzite, **TOPSOIL**.

0.30

Slightly moist, greyish brown, medium dense, **pinholed to voided**, silty sandy GRAVEL with occasional subangular to rounded gravel to cobbles of quartzite, **PEBBLE MARKER ZONE**.

0.50

Moist, brown speckled black to orange, firm, **open to fissured**, silty clayey gravelly SAND with subrounded gravel of mixed origin with Fe concretions, **SLIGHTLY FERRUGINISED REWORKED RESIDUAL DIABASE**.

1.40

Moist, light brown speckled black to white, **completely weathered**, medium grained, relict structure, dense, clayey silty SAND, extremely weak (ISRM 1981 rock class, <R0), **DIABASE**.

Material properties:

Clay = 1%, Silt = 21%, Sand = 63%, Gravel = 15%, PI = 17.1%, LL = 43.7%, LS = 9.1%, GM = 1.40, SG = 2.793, VDM Heave Potential = Low, HRB class = A-2-7(1), USCS = SM, MDD = 1 730kgm3, OMC = 15.8%.

2.90

As above

NOTES

1) TLB terminated near maximum depth.

2) Soft excavation conditions at refusal depth.

3) Water seepage encountered at 2.20m.

4) No sidewall instability during time of investigation.

5) Disturbed sample retrieved at 1.00--2.50m.

6) Diabase boulders near test pit.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06

ELEVATION : 1452m
X-COORD : 28.89606
Y-COORD : -25.93564

HOLE No: TP55

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

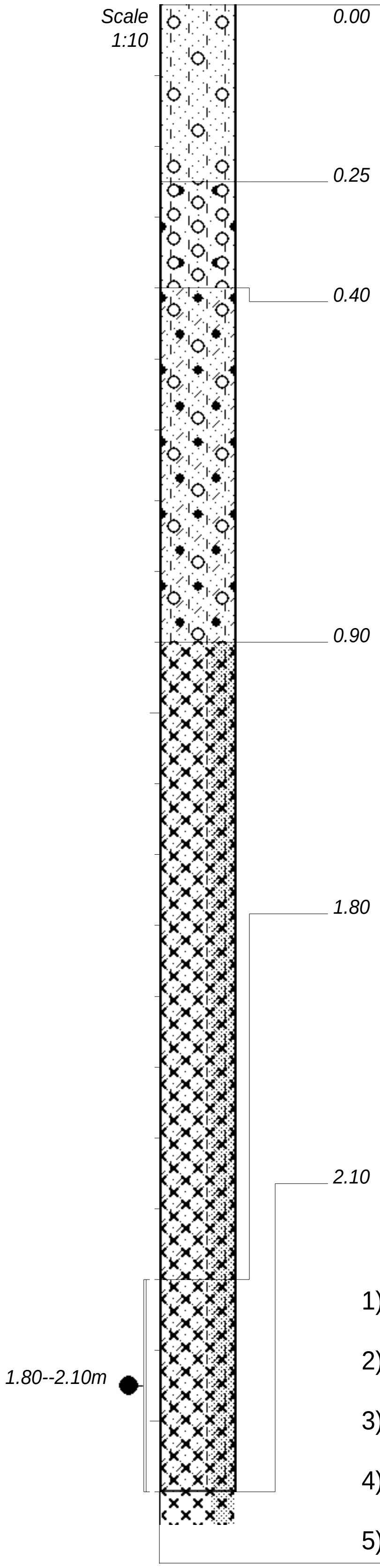
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dot PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP56
Sheet 1 of 1

JOB NUMBER: RS22025



Slightly moist, greyish brown, <u>medium dense</u>, **pinholed to voided**, silty gravelly SAND with occasional subangular to rounded gravel of quartzite, **TOPSOIL**.

Slightly moist, greyish brown, <u>medium dense</u>, **pinholed to voided**, silty sandy GRAVEL with occasional subangular to rounded gravel to cobbles of quartzite, **PEBBLE MARKER ZONE**.

Moist, dark red brown speckled black, <u>firm</u>, **open to fissured**, silty clayey gravelly SAND with subrounded gravel of mixed origin with Fe concretions, **SLIGHTLY FERRUGINISED REWORKED RESIDUAL DIABASE**.

Moist, light brown speckled black to white, **completely weathered**, medium grained, relict structure, <u>dense</u>, clayey silty SAND, extremely weak (ISRM 1981 rock class, <R0), **DIABASE**.

<u>Note</u>: Diabase corestones present in horizon.

Moist, light brown speckled black to white, **completely to highly weathered**, medium grained, relict structure, <u>dense</u>, clayey silty SAND, very weak (ISRM 1981 rock class, <R1), **DIABASE**.

<u>Material properties</u>:

Clay = 2%, Silt = 7%, Sand = 52%, Gravel = 39%, PI = 17.7%, LL = 48.9%, LS = 8.3%, GM = 2.01, SG = 2.695, VDM Heave Potential = Low, HRB class = A-2-7(0), USCS = SP - SM, MDD = 1 914kgm3, OMC = 15.5%.

Refusal seeming on large >1m diameter diabase corestone.

NOTES

1) TLB confined refusal at 2.10m.
2) Intermediate excavation conditions at refusal depth.
3) No seepage encountered during time of investigation.
4) No sidewall instability during time of investigation.
5) Disturbed sample retrieved at 1.80--2.10m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06

ELEVATION : 1453m
X-COORD : 28.89545
Y-COORD : -25.93489

HOLE No: TP56

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

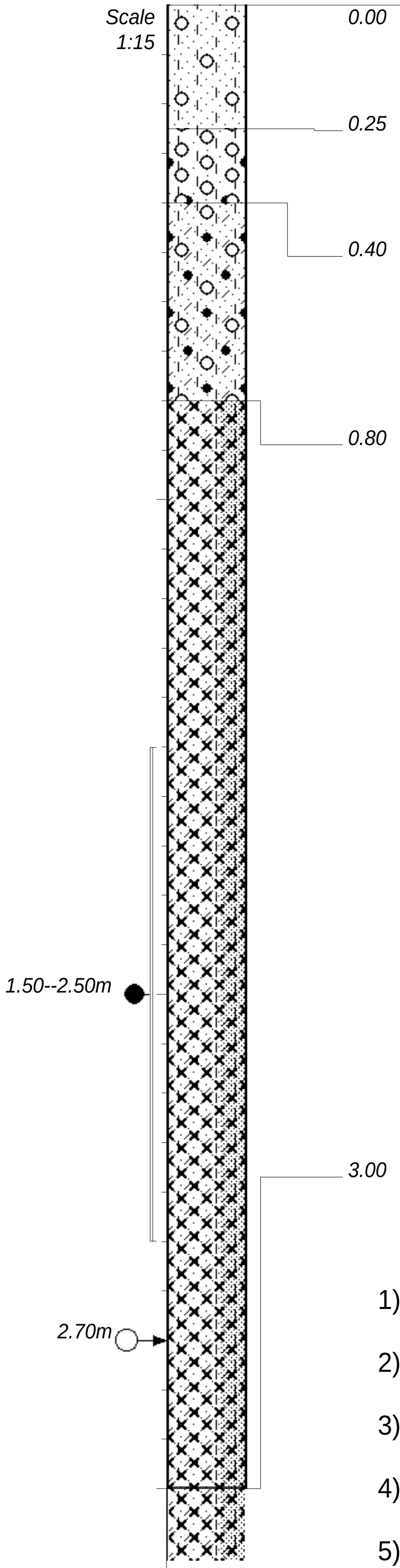
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP57
Sheet 1 of 1

JOB NUMBER: RS22025



Slightly moist, greyish brown, medium dense, **pinholed to voided**, silty gravelly SAND with occasional subangular to rounded gravel of quartzite, **TOPSOIL**.

Slightly moist, greyish brown, medium dense, **pinholed to voided**, silty sandy GRAVEL with occasional subangular to rounded gravel to small boulders of quartzite, **PEBBLE MARKER ZONE**.

Moist, dark red brown speckled black, firm, **open to fissured**, silty clayey gravelly SAND with subrounded gravel of mixed origin with Fe concretions, **SLIGHTLY FERRUGINISED REWORKED RESIDUAL DIABASE**.

Note: Diabase corestones present in horizon.

Moist, light brown speckled black to white, **completely weathered**, medium grained, relict structure, dense, clayey silty SAND, extremely weak (ISRM 1981 rock class, <R0), **DIABASE**.

Note: Diabase corestone present in horizon. Occasional residual grey sandy silty gauge on joints with depth.

Material properties:

Clay = 5%, Silt = 13%, Sand = 57%, Gravel = 26%, PI = 17.0%, LL = 41.5%, LS = 7.2%, GM = 1.62, SG = 2.760, VDM Heave Potential = Low, HRB class = A-2-7(0), USCS = SC, MDD = 1 792kgm3, OMC = 20.7%.

As above, with very closely to closely spaced joints.

NOTES

1) TLB terminated near maximum depth.
2) Soft excavation conditions at refusal depth.
3) Water seepage encountered at 2.70m.
4) No sidewall instability during time of investigation.
5) Disturbed sample retrieved at 1.50--2.50m.
6) Diabase outcrop and boulders near test pit.

CONTRACTOR : EPCM
INCLINATION :
ELEVATION : 1458m
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DIAM :
X-COORD : 28.89840
DRILLED BY : RockSoilConsult (Pty) Ltd
DATE : 2022-05-25
Y-COORD : -25.93702
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff
DATE : 2022/06/06

HOLE No: TP57

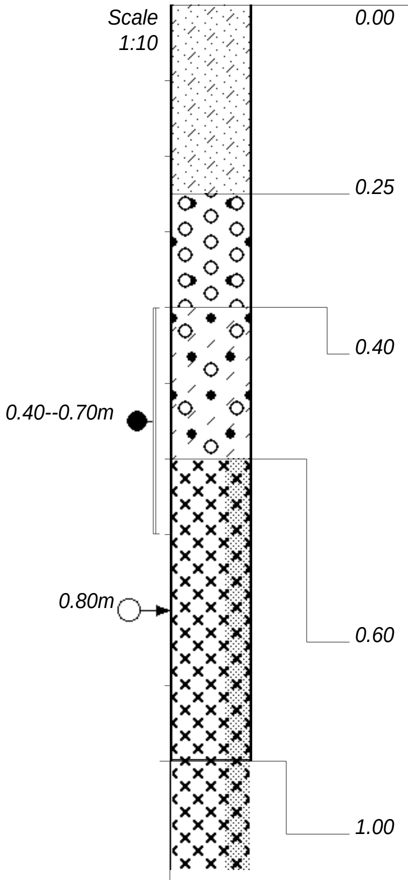
TYPE SET BY : JI Roux
DATE : 08/11/2022 14:42
SETUP FILE : STANDARD.SET
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dot●PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP58
Sheet 1 of 1

JOB NUMBER: RS22025



Slightly moist, dark brown, medium dense, **open to voided to slightly shattered**, clayey silty SAND with occasional gravel of quartzite, **TOPSOIL**.

Moist, dark brown mottled purple to black, dense to stiff, **open to slightly fissured**, clayey silty sandy GRAVEL with subrounded gravel to cobbles of mixed origin, **SLIGHTLY FERRUGINISED PEBBLE MARKER ZONE**.
Note: Fe concretions present in horizon.

Moist to very moist, dark brown mottled black to orange, firm to stiff, silty clayey gravelly with subrounded gravel of mixed origin and Fe concretions, **REWORKED RESIDUAL DIABASE**.
Note: Slightly ferruginised corestones in horizon.
Material properties:
Clay = 5%, Silt = 16%, Sand = 3534%, Gravel = 26%, PI = 16.7%, LL = 55.5%, LS = 9.0%, GM = 1.55, SG = 2.689, VDM Heave Potential = Low, HRB class = A-2-7(0), USCS = SM.

Moist, orange brown to khaki speckled black, **completely weathered to highly weathered**, medium grained, massive to jointed occasionally closely jointed, dense, extremely weak (ISRM 1981 rock class, <R0), **DIABASE**.
Note: Residual diabase zones present.

Moist, orange brown to khaki speckled black, **highly weathered to unweathered**, medium grained, massive to jointed occasionally closely jointed, dense, very weak to medium strong (ISRM 1981 rock class, R1 to R3), **DIABASE**.
Note: Residual diabase zones present.

NOTES

1) TLB refusal at 1.00m.
2) Intermediate to hard excavation conditions at refusal depth.
3) 2 Bucket width excavation.
4) Water seepage encountered at 0.80m.
5) No sidewall instability during time of investigation.
6) Disturbed sample retrieved at 0.40--0.70m.
7) Highly undulating horizon.
8) Diabase boulders to outcrop near test pit.

CONTRACTOR : EPCM
INCLINATION :
ELEVATION : 1451m
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DIAM :
X-COORD : 28.89853
DRILLED BY : RockSoilConsult (Pty) Ltd
DATE : 2022-05-25
Y-COORD : -25.93832
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff
DATE : 2022/06/06

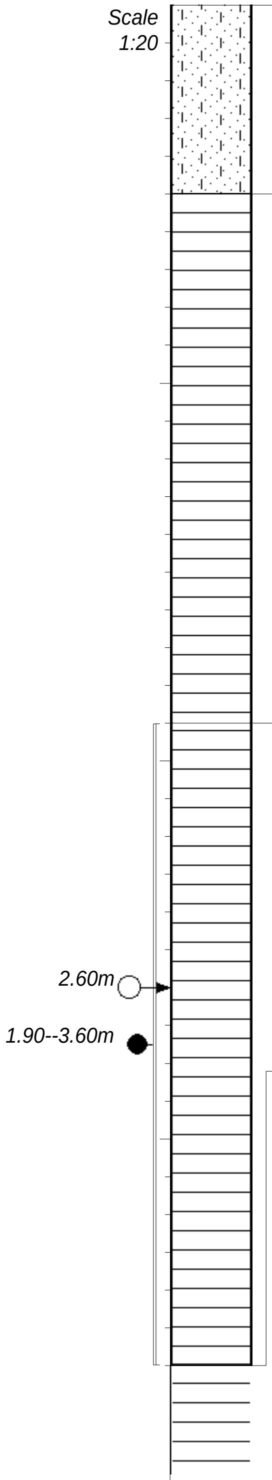
HOLE No: TP58

TYPE SET BY : JI Roux
DATE : 08/11/2022 14:42
SETUP FILE : STANDARD.SET
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP59
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Moist, reddish brown, loose to medium dense, **open structured**, silty SAND, **TOPSOIL**.

0.50

Moist, yellowish brown stained red and black, **completely weathered to highly weathered**, fine grained, laminated, very weak to weak (ISRM 1981 rock class, R1 to R2), **SHALE**.
Note: Moist, reddish brown, silty sand, infill.

1.90

Slightly moist to moist, greyish brown, **slightly weathered to unweathered**, fine grained, laminated, moderately strong to strong (ISRM 1981 rock class, R3 to R4), **SHALE**.
Note: Moist, reddish brown, silty sand, infill.
Material properties:
Clay = 6%, Silt = 13%, Sand = 11%, Gravel = 70%, PI = 19.7%, LL = 47.9%, LS = 9.8%, GM = 2.27, SG = 2.737, VDM Heave Potential = Low, HRB class = A-2-7(0), USCS = GM.

3.60

As above.

NOTES
1) TLB confined termination near maximum depth.
2) Soft excavation conditions down to termination depth.
3) Intermediate excavation conditions at termination depth.
4) Shale is rippable.
5) Water seepage present at 2.60m.
6) Disturbed sample retrieved at 1.90--3.60m.
7) Sidewalls stable during time of investigation.
8) Undulating soil profile.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06

ELEVATION : 1448m
X-COORD : 28.89862
Y-COORD : -25.93933

HOLE No: TP59

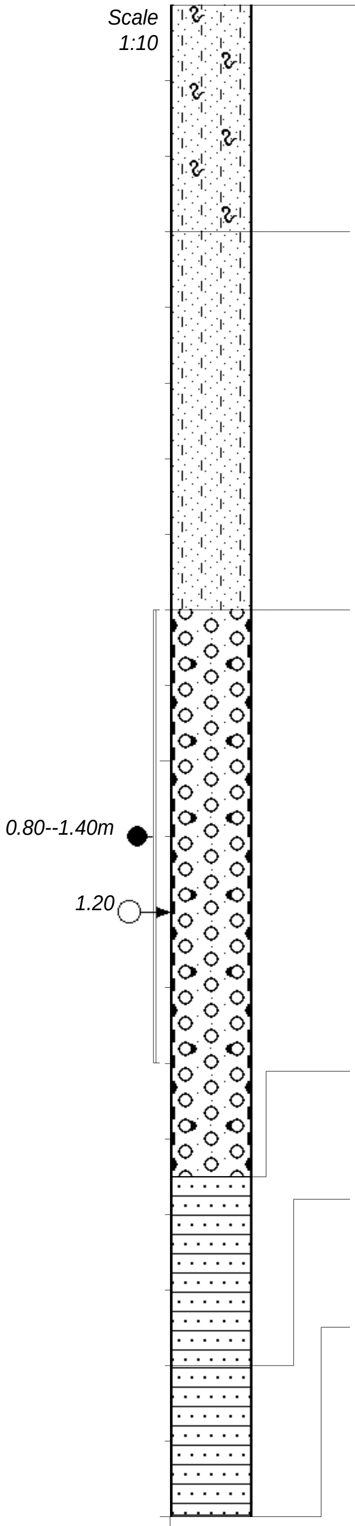
TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP65
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Moist, dark brown, loose to medium dense, **pinholed to voided**, gravelly silty SAND with occasional sub-rounded to rounded gravel to cobbles of mixed origin, **TOPSOIL**. Scattered roots.

0.30

Moist, brown speckled black, loose to medium dense, **pinholed to voided**, gravelly silty SAND with minor amounts of sub-rounded gravel of mixed origin, **COLLUVIUM**.

0.80

Moist, brown, medium dense, **open structured**, clayey silty sandy GRAVEL abundant amounts of sub-rounded gravel to cobble of mixed origin with minor amounts of weakly to strongly cemented Fe concretions, **SLIGHTLY FERRUGINISED PEBBLE MARKER ZONE**. Minor mounts of roots.

Note: Occasional honeycomb to hardpan ferricrete zones.

Note: Seepage water flow rate at approximately 30l/min.

Material properties:

Clay = 5%, Silt = 16%, Sand = 38%, Gravel = 41%, PI = 6.7%, LL = 24.1%, LS = 2.5%, GM = 0.31, SG = 1.75, VDM Heave Potential = Low, HRB class = A-2-4(0), USCS = SM & SC.

1.55

Moist to very wet, pink to yellow brown mottled grey streaked brown, fine grained, **completely weathered**, massive, dense, extremely weak rock (ISRM 1981 rock class, <R0), **ARGILLACEOUS SANDSTONE**.

1.80

Moist to very wet, grey streaked brown, fine grained, **completely weathered to highly weathered**, massive, dense, very weak rock (ISRM 1981 rock class, <R1), **ARGILLACEOUS SANDSTONE**.

2.00

NOTES

1) Confined refusal with excavator.
2) Intermediate excavation conditions at termination depth.
3) Water seepage encountered at 1.20 during time of inspection.
4) No sidewall instability during time of investigation
5) Disturbed sample retrieved at 0.80--1.40m.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06

ELEVATION : 1477m
X-COORD : 28.89997
Y-COORD : -25.95301

HOLE No: TP65

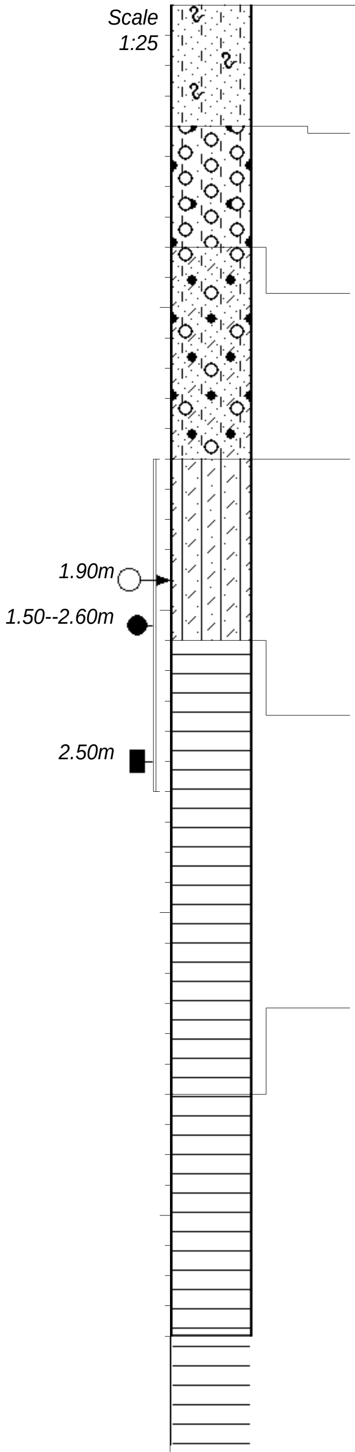
TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

dot●PLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP67
Sheet 1 of 1

JOB NUMBER: RS22025



0.00

Moist, light brown, loose to medium dense, **pinholed to voided**, gravelly clayey silty SAND with abundant amounts of sub-rounded pebbles of mixed origin, **TOPSOIL**. Scattered roots.

0.40

Moist, brown speckled black, loose to medium dense, **open structured to pinholed**, silty gravelly SAND to silty sandy GRAVEL with abundant amounts of sub-rounded gravel of mixed origin with minor amounts of Fe concretions, **PEBBLE MARKER ZONE**.

0.80

Moist to very moist, light yellow brown mottled red, firm, **shattered to massive**, clayey sandy SILT to gravelly clayey silty SAND with occasionally sub-rounded pebbles to cobbles of mixed origin with minor amounts of Fe concretions, **REWORKED RESUDIAL DIAMICTITE**.

1.50

Wet, purple stained grey to yellow, stiff to firm, **laminated**, sandy clayey SILT, **RESIDUAL SHALE**.

Note: Seepage water flow rate at approximately 10-20l/min.

Material properties:

Clay = 45%, Silt = 30%, Sand = 17%, Gravel = 8%, PI = 15.9%, LL = 54.4%, LS = 8.9%, GM = 0.45, SG = 2.731, VDM Heave Potential = Low, HRB class = A-7-5(16), USCS = MH.

2.10

Wet, purple stained grey to yellow to orange, fine grained, **highly weathered**, massive to bedded/laminated, stiff to extremely weak rock (ISRM 1981 rock class, <R0), **SHALE**.

3.60

As above but completely weathered to highly weathered shale (ISRM 1981 rock class, <R1).

NOTES

1) Confined refusal of excavator.

2) Intermediate excavation conditions at refusal depth.

3) Water seepage encountered at approximately 10-20 liters/min at 1.90m during time of investigation.

4) Sidewall stable during the time of inspection.

5) Disturbed sample retrieved at 1.50--2.60m.

6) Undisturbed sample retrieved at 2.50m.

7) Near drainage feature.

8) Undulating profiles.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06

ELEVATION : 1468m
X-COORD : 28.90718
Y-COORD : -25.95991

HOLE No: TP67

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

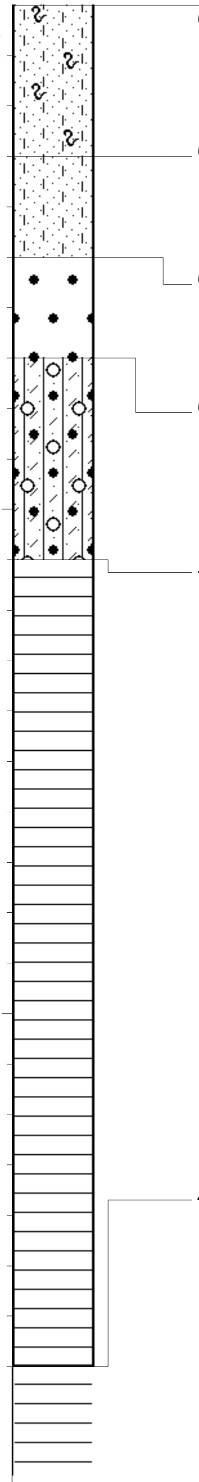
dotPLOT

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP68
Sheet 1 of 1

JOB NUMBER: RS22025

Scale 1:15



0.00	Moist, dark brown, <u>loose to medium dense</u>, **pinholed to voided**, clayey silty SAND with minor amounts of sub-angular to gravel to cobbles of mixed origin, **TOPSOIL**. Scattered roots.
0.30	Moist to very moist, orange yellow brown, <u>medium dense</u>, **pinholed to voided**, silty SAND to silty clayey sand moving down in profile, **COLLUVIUM**.
0.50	Moist, orange yellow brown, <u>medium dense</u>, **open structured**, gravelly silty SAND with abundant amounts of sub-rounded cobbles to boulders of mixed origin, **PEBBLE MARKER ZONE**.
0.70	Moist, yellow orange stained black, <u>firm to stiff</u>, **open to massive to fissured structure**, clayey gravelly sandy SILT with minor amounts of sub-rounded to rounded gravel to cobbles of mixed origin, **REWORKED RESIDUAL SHALE**.
1.10	Moist to wet, orange yellow grey stained black, fine grained, **completely weathered**, laminated and massive, <u>stiff to very stiff</u>, extremely weak (ISRM 1981 rock class, <R0), **SHALE.**
2.70	Same as above but more highly weathered (ISRM 1981 rock class, R1).

NOTES

1) Excavator confined refusal.

2) Soft to intermediate excavation conditions at refual depth.

3) No water seepage encountered during time of investigation.

4) No sidewall instability during the time of investigation.

5) No sample retrieved.

6) Near drainage feature.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06

ELEVATION : 1465m
X-COORD : 28.91060
Y-COORD : -25.95920

HOLE No: TP68

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

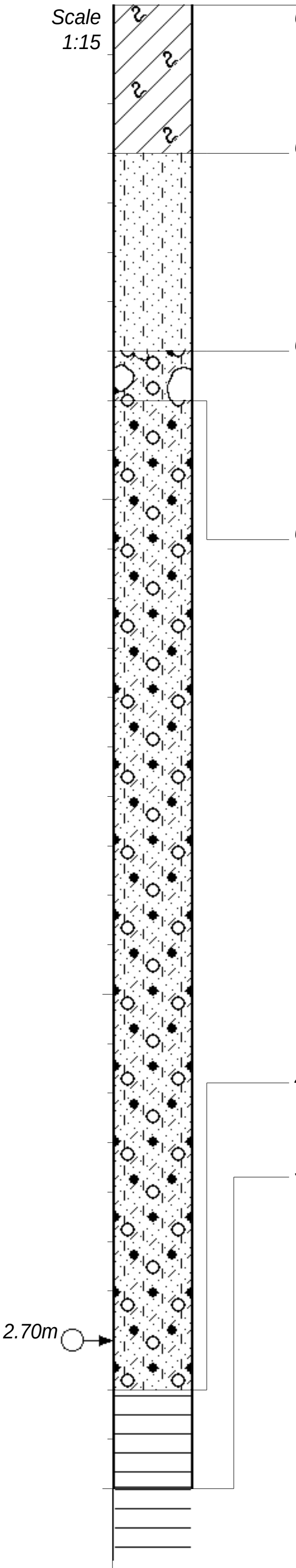
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP69
Sheet 1 of 1

JOB NUMBER: RS22025

Scale 1:15



0.00	Moist, brown to black mottled orange, medium dense, **open structured to shattered**, silty sandy CLAY with occasional sub-rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.
0.30	Moist, yellow brown, medium dense, **pinholed to voided**, clayey silty SAND with minor amounts of sub-rounded gravel of mixed origin, **COLLUVIUM**.
0.70	Moist, orange mottled red to black, moderately to strongly cemented Fe concretions with sub-rounded to rounded gravel to cobbles of mixed origin in a very moist, grey, soft, silty clayey SAND matrix, nodular to honeycombed ferricrete, **SLIGHTLY FERRUGINISED PEBBLE MARKER ZONE**.
0.80	Moist, light brown to grey, stiff to dense, **open structured to slightly fissured**, gravelly silty clayey SAND to clayey gravelly silty SAND with sub-rounded gravel to cobbles of mixed origin with moderately to strongly commented Fe concretions, **SLIGHTLY FERRUGINISED REWORKED RESUDAL DIAMICTITE**. **Note**: Seepage water flow rate at approximately 0.5l/min.
2.80	Moist, purple, completely weathered, firm to stiff, **laminated**, fine grained, extremely weak rock (ISRM 1981 rock class, <R0) **SHALE**.
3.00	As above but highly weathered (ISRM 1981 rock class, R1).

NOTES

1) Confined refusal with excavator.

2) Intermediate excavation conditions at refusal depth.

3) Water seepage encountered at 2.70m during time of investigation.

4) Sidewall stable during the time of inspection.

5) No sample retrieved.

6) Near drainage feature.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06

ELEVATION : 1469m
X-COORD : 28.91179
Y-COORD : -25.96108

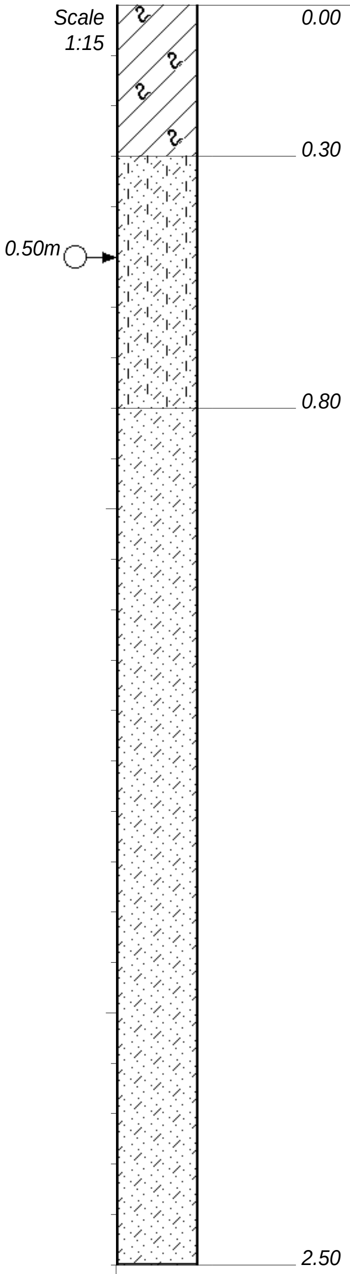
HOLE No: TP69

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET
DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

Eco Elementum Engineering
Kusile ADF Preliminary Profiles

HOLE No: TP86
Sheet 1 of 1

JOB NUMBER: RS22025



0.00 – Very moist, dark brown to black, firm, **shattered**, silty sandy CLAY with occasional sub-rounded gravel of mixed origin, **TOPSOIL**. Scattered roots.

0.30 – Moist to wet, grey brown mottled orange, soft to firm, **shattered to fissured**, silty clayey SAND with occasional sub-rounded gravel to cobbles of mixed origin, **ALLUVIUM**.

Note: Seepage water flow rate at approximately 50-70l/min.

0.80 – Wet, grey mottled orange, soft, **open structured to shattered to fissured**, gravelly silty clayey SAND with occasional sub-rounded gravel to cobbles of mixed origin, **ALLUVIUM**.

2.50

NOTES

1) Hole terminated, due to collapse.

2) Soft excavation conditions at termination depth.

3) Water seepage encountered at 0.50m during time of investigation.

4) No sample retrieved.

5) Close to drainage feature.

CONTRACTOR : EPCM
MACHINE : JCB 205 LC Excavator & JCB 3XC TLB
DRILLED BY : RockSoilConsult (Pty) Ltd
PROFILED BY : JI Roux, CJ Homan & JHL de Graaff
TYPE SET BY : *JI Roux*
SETUP FILE : *STANDARD.SET*

INCLINATION :
DIAM :
DATE : 2022-05-25
DATE : 2022/06/06
DATE : *08/11/2022 14:42*
TEXT : *..s\RS22025SP(V1.4)Lab.txt*

ELEVATION : 1453m
X-COORD : 28.90791
Y-COORD : -25.95731

HOLE No: TP86

dot PLOT

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Kusile ADF Preliminary Profiles

LEGEND

Sheet 1 of 2

JOB NUMBER: RS22025

SCATTERED BOULDERS/occasional boulders	{SA48}
GRAVEL	{SA02}
GRAVELLY	{SA03}
SAND	{SA04}
SANDY	{SA05}
SILT	{SA06}
SILTY	{SA07}
CLAY	{SA08}
CLAYEY	{SA09}
CONGLOMERATE/agglomerate/tillite	{SA10}
SANDSTONE	{SA11}
MUDROCK/mudstone/siltstone/shale	{SA12}
HYPABYSSAL/anorthosite/syenite aplite	{SA18}
DIORITE FAMILY	{SA41}
DIABASE	{SA18}{SA41}
HARDPAN FERRICRETE	{SA23}{SA29}
HONEYCOMB FERRICRETE/ferricrete/nodular ferricrete	{SA24}
SPARSE FERRICRETE NODULES/occasional ferricrete nodu....	{SA25}
WELL CEMENTED/well stabilised/well ferruginised	{SA29}
PARTIALLY CEMENTED/partially stabilised/partially ferruginised	{SA30}
FILL	{SA32}
Name ▮ UNDISTURBED SAMPLE	{SA37}

dot●PLOT

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LEGEND
Sheet 2 of 2

JOB NUMBER: RS22025

Name ●	DISTURBED SAMPLE	{SA38}
	ROOTS	{SA40}
27.09 ○→	WATER SEEPAGE/water strike	{CH50}
	BAND	{SA53}
	COBBLES	{SA58}

CONTRACTOR :
MACHINE :
DRILLED BY :
PROFILED BY :

INCLINATION :
DIAM :
DATE :
DATE :

ELEVATION :
X-COORD :
Y-COORD :

TYPE SET BY : JI Roux
SETUP FILE : STANDARD.SET

DATE : 08/11/2022 14:42
TEXT : ..s\RS22025SP(V1.4)Lab.txt

LEGEND
SUMMARY OF SYMBOLS

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

BH No.	BH03		
Box No.	1	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

BH No.	BH03		
Box No.	2	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

BH No.	BH03		
Box No.	3	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:

1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

ROCKSOIL
CONSULT

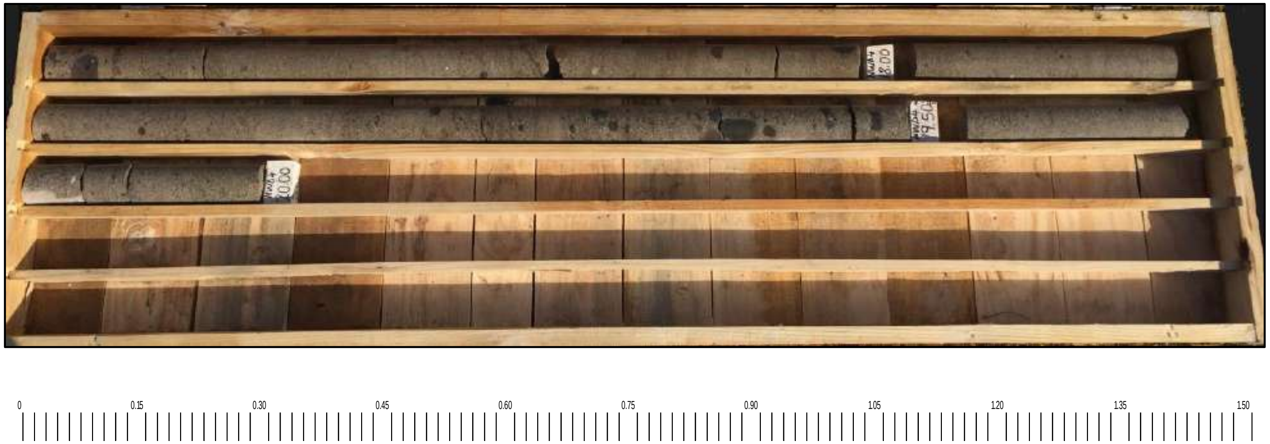
BH No.	BH04		
Box No.	1	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

BH No.	BH04		
Box No.	2	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation



BH No.	BH04		
Box No.	3	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

BH No.	BH06		
Box No.	1	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

BH No.	BH06		
Box No.	2	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

BH No.	BH06		
Box No.	3	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

BH No.	BH07		
Box No.	1	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

BH No.	BH07		
Box No.	2	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

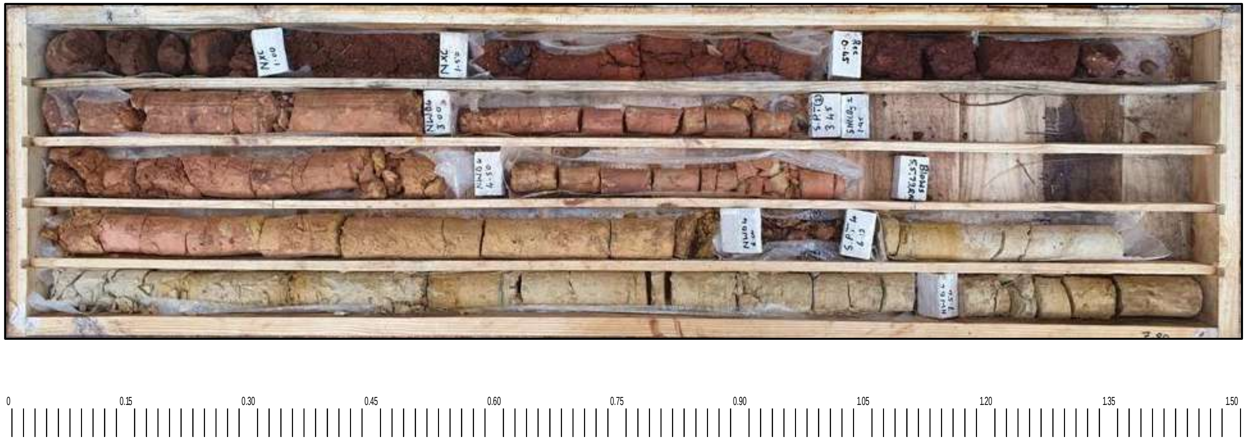
Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

BH No.	BH07		
Box No.	3	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation



BH No.	BH08		
Box No.	1	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

BH No.	BH08		
Box No.	2	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

BH No.	BH08		
Box No.	3	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

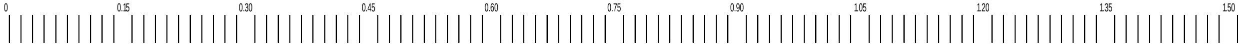
BH No.	BH09		
Box No.	1	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

BH No.	BH09		
Box No.	2	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation



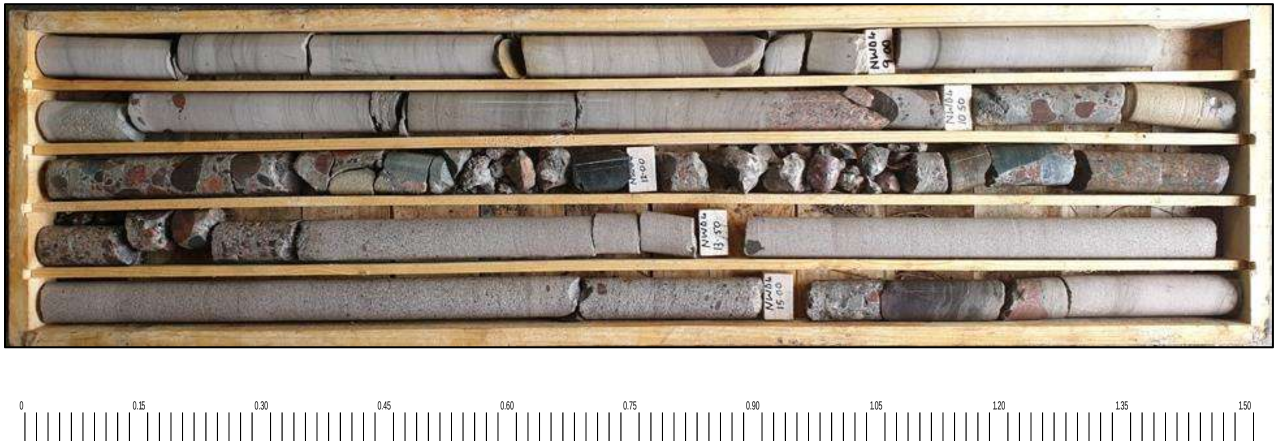
BH No.	BH09		
Box No.	3	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

BH No.	BH10		
Box No.	1	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.



BH No.	BH10		
Box No.	2	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Project reference number: RS22025
Project name: Kusile ADF Geotechnical Investigation

BH No.	BH10		
Box No.	3	of	3
Core Size:	NWD4		
BH Depth (m):	20.00		
Piezo Installed:			
Water Level:	Refer to report		

Notes:
1) Refer to final logs and final report.

Appendix C

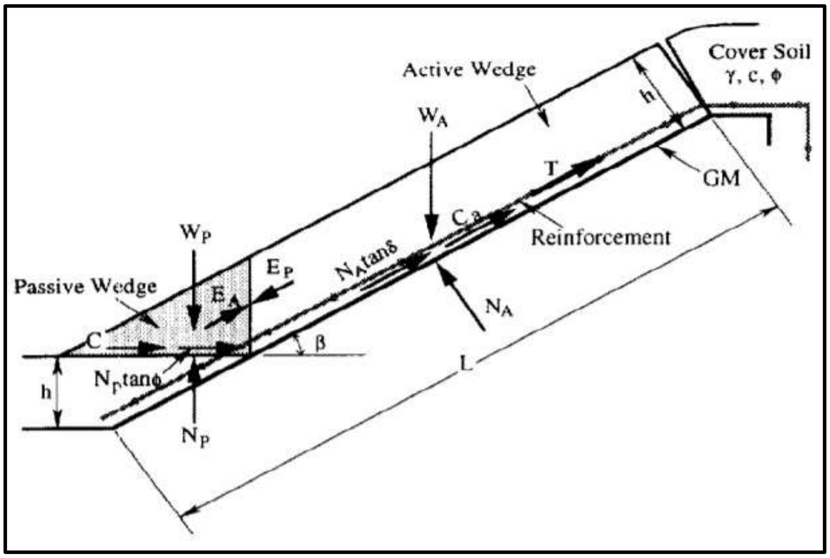
Calculations and Supporting Documentation

COVER SOIL STABILITY ANALYSIS WORKSHEET

Uniform Cover Soil Thickness with Overlying Geosynthetics

Client: EPCM - Eskom Holdings
Project: Kusile 60yr ADF - Phase 1 ADF
Description: Slope Stability Analysis - Cell Slopes
Project No.: ES-816-2023
Date: Envitech Solutions (Pty) Ltd
Author: KP Matulovich

(Based on TY Soong and RM Koerner, 2005)



Calculation of FS

Active Wedge:

Wa= 29.6 kN
Na= 28.1 kN

Passive Wedge:

Wp= 2.8 kN

$$FS = \frac{-b+\sqrt{b^2-4ac}}{2a}$$

a= 2.80
b= -4.21
c= 0.70

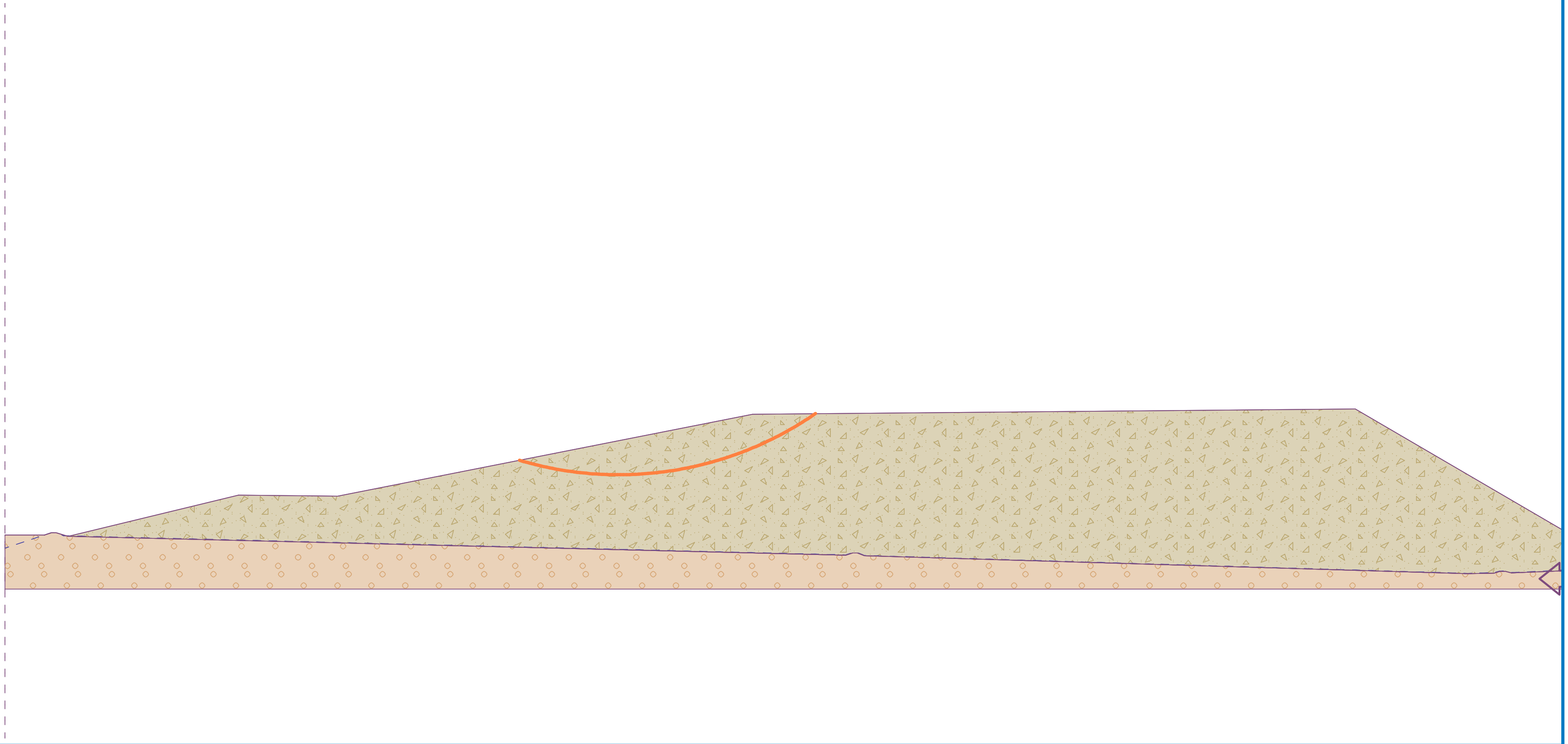
FS= 1.31

thickness of cover sand layer = h = 0.30 m
soil slope angle beneath the geomembrane = β = 18.43 ° *= 0.322 (rad.)*
length of slope measured along the geomembrane = L = 6.30 m
unit weight of the soil layer= γ = 18.6 kN/m^3
friction angle of the soil layer = φ = 36.0 ° *= 0.628 (rad.)*
cohesion of the soil layer = c = 0.0 kN/m^2 *C= 0 kN*
critical interface friction angle = δ = 8.7 ° *= 0.152 (rad.)*
critical interface adhesion = ca = 1.0 kN/m^2 *Ca= 5.3511 kN*

Note: numbers in boxes are input values

numbers in Italics are calculated values

Name : **Stage - analysis : 1 - 2**

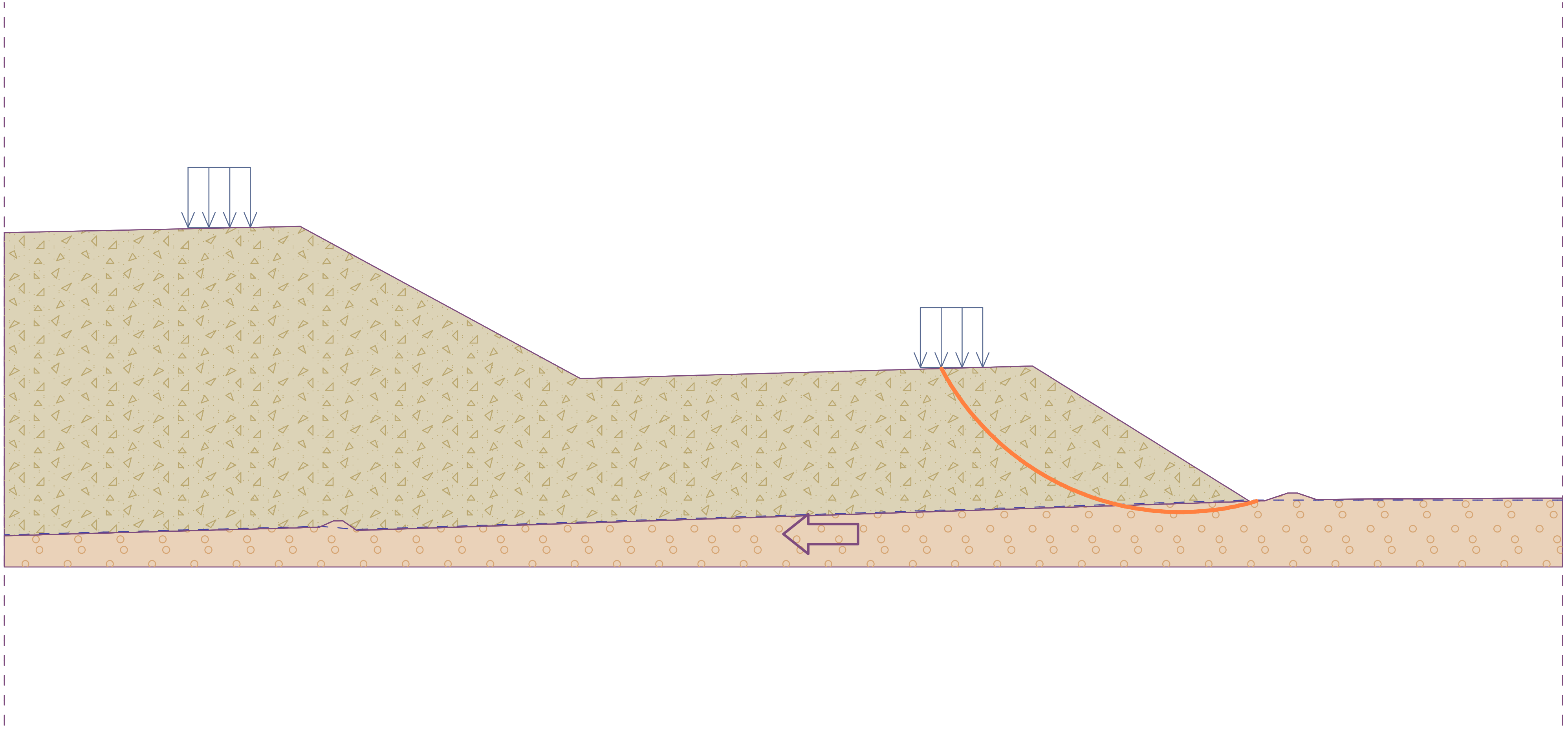


Analysis of the slip surface without optimization.

Slope stability verification (all methods)

Bishop :	FS = 3.41 > 1.50	**ACCEPTABLE**
Fellenius / Petterson :	FS = 3.27 > 1.50	**ACCEPTABLE**
Spencer :	FS = 3.41 > 1.50	**ACCEPTABLE**
Janbu :	FS = 3.41 > 1.50	**ACCEPTABLE**
Morgenstern-Price :	FS = 3.41 > 1.50	**ACCEPTABLE**

Name : **Stage - analysis : 1 - 2**

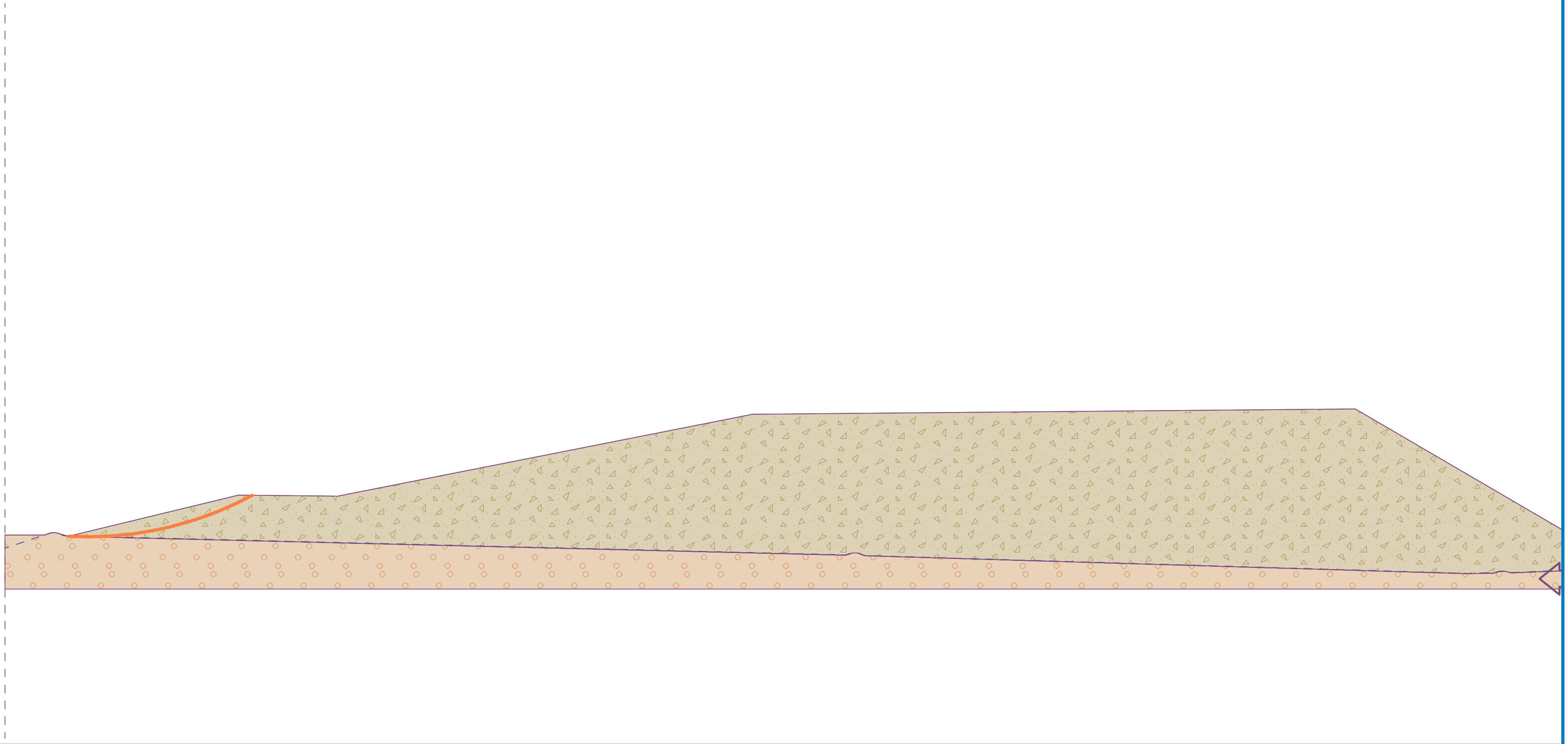


Analysis of the slip surface without optimization.

Slope stability verification (all methods)

Bishop : FS = 1.48 < 1.50 **NOT ACCEPTABLE**
Fellenius / Petterson : FS = 1.32 < 1.50 **NOT ACCEPTABLE**
Spencer : FS = 1.46 < 1.50 **NOT ACCEPTABLE**
Janbu : FS = 1.46 < 1.50 **NOT ACCEPTABLE**
Morgenstern-Price : FS = 1.45 < 1.50 **NOT ACCEPTABLE**

Name : **Stage - analysis : 1 - 2**

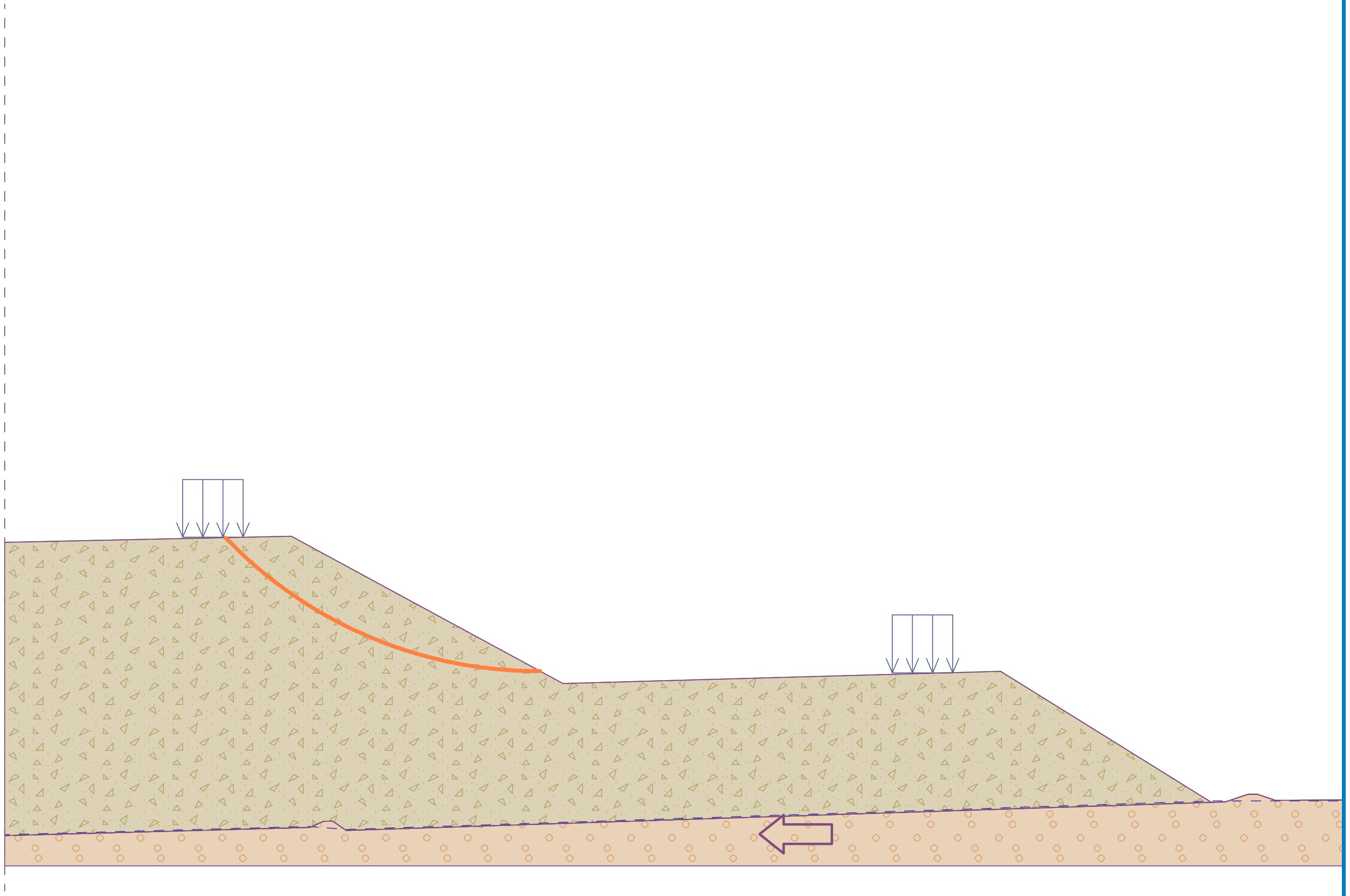


Analysis of the slip surface without optimization.

Slope stability verification (all methods)

Bishop :	FS = 2.50 > 1.50	**ACCEPTABLE**
Fellenius / Petterson :	FS = 2.46 > 1.50	**ACCEPTABLE**
Spencer :	FS = 2.50 > 1.50	**ACCEPTABLE**
Janbu :	FS = 2.50 > 1.50	**ACCEPTABLE**
Morgenstern-Price :	FS = 2.50 > 1.50	**ACCEPTABLE**

Name : **Stage - analysis : 1 - 2**

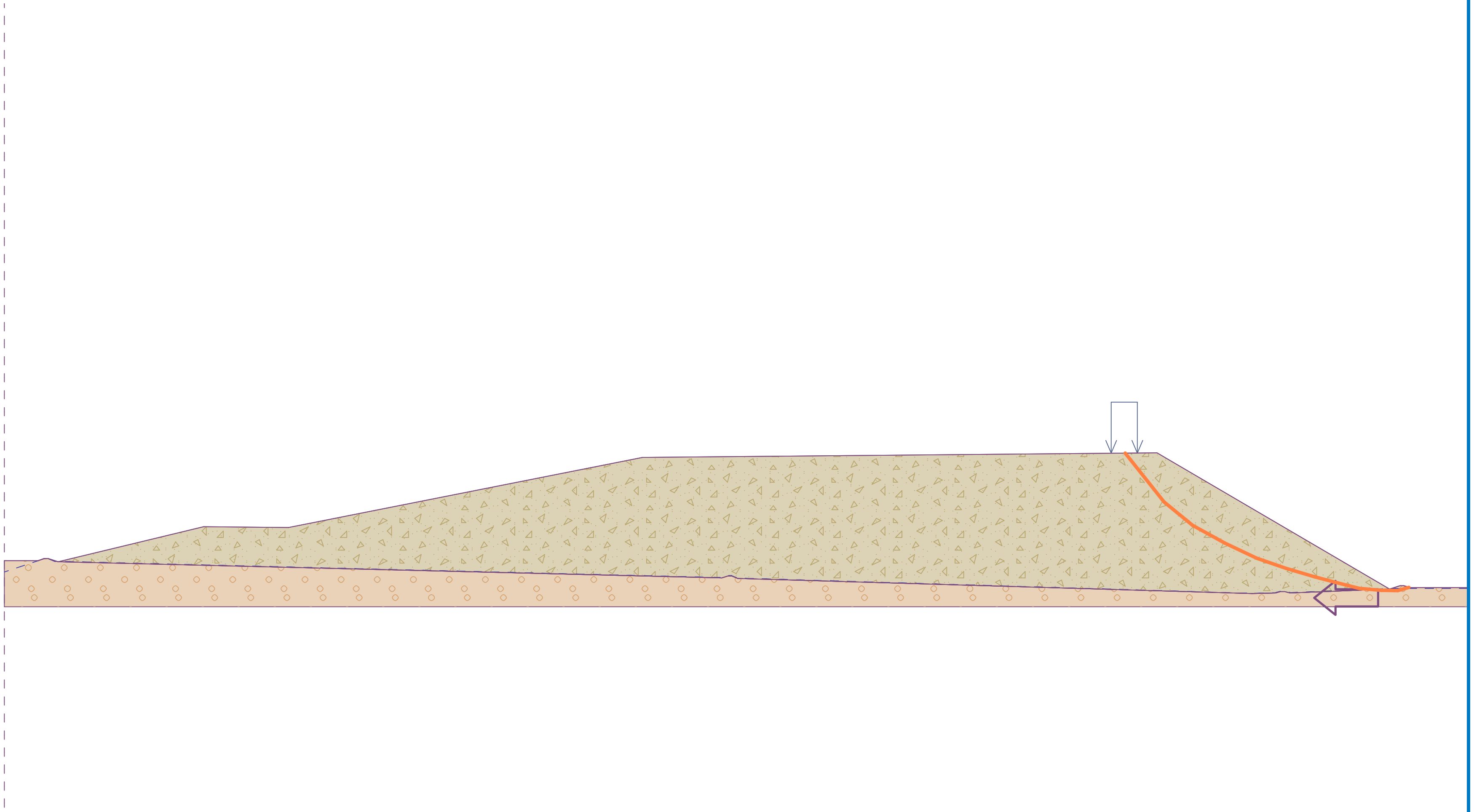


Analysis of the slip surface without optimization.

Slope stability verification (all methods)

Bishop : FS = 1.43 < 1.50 **NOT ACCEPTABLE**
Fellenius / Petterson : FS = 1.37 < 1.50 **NOT ACCEPTABLE**
Spencer : FS = 1.43 < 1.50 **NOT ACCEPTABLE**
Janbu : FS = 1.43 < 1.50 **NOT ACCEPTABLE**
Morgenstern-Price : FS = 1.43 < 1.50 **NOT ACCEPTABLE**

Name : **Stage - analysis : 1 - 2**



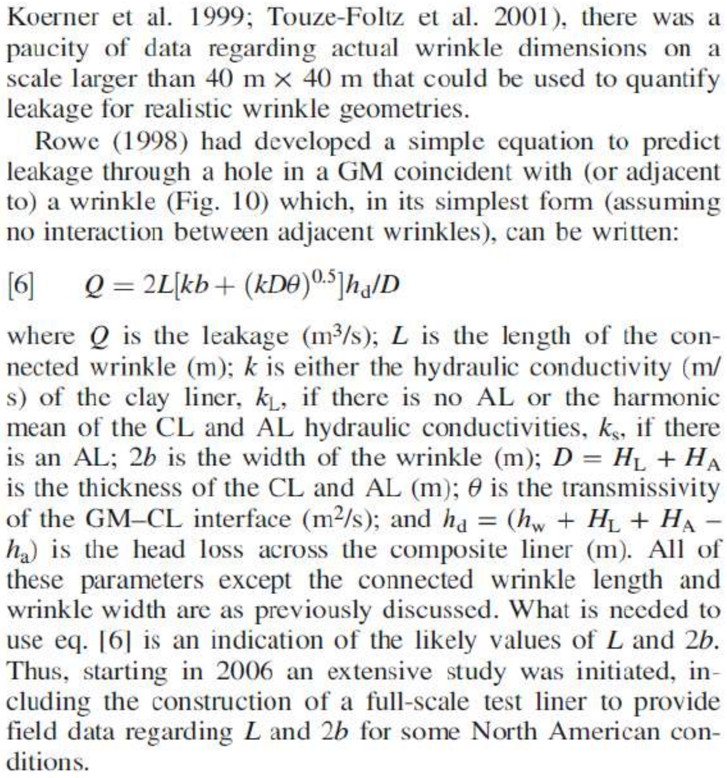
Analysis of the slip surface without optimization.

Slope stability verification (all methods)

Sarma :	FS = 1.29 < 1.50 **NOT ACCEPTABLE**
Spencer :	FS = 1.30 < 1.50 **NOT ACCEPTABLE**
Janbu :	FS = 1.30 < 1.50 **NOT ACCEPTABLE**
Morgenstern-Price :	FS = 1.29 < 1.50 **NOT ACCEPTABLE**

KUSILE 60YR ADF - PHASE 1
ADF - POTENTIAL LEAKAGE FLOW RATE CALCULATION

Length of connected wrinkle	L	14	m
Hydraulic Conductivity	k	1.00E-07	m/s
Width of the wrinkle	2b	0.15	m
Thickness of Clay and Atenuation layer	D	0.1	m
Transmissivity of GM-GCL interface	Theta	1.60E-08	m^2/s
Head loss across the composite liner	h_d	0.16	m
Leakage	Q	1.23868E-06	m^3/s
Leakage	Q	107.02	litres per wrinkle per day
Assuming 5No wrinkles with holes per hectare	Q	535.11	litres per hectare per day

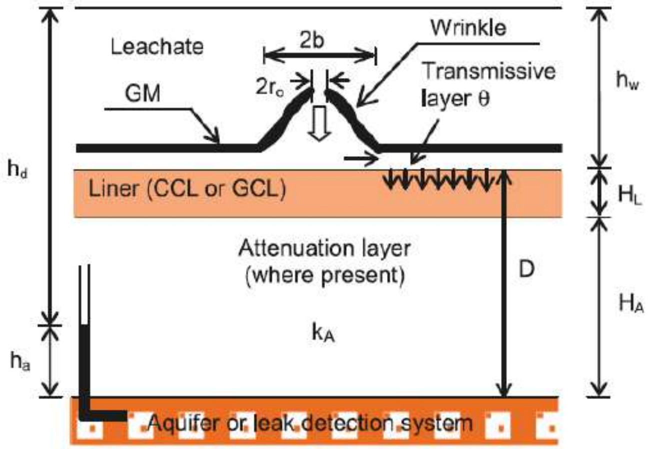


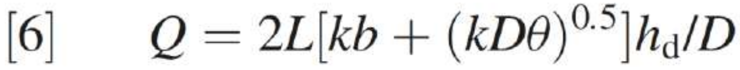
Koerner et al. 1999; Touze-Foltz et al. 2001), there was a paucity of data regarding actual wrinkle dimensions on a scale larger than 40 m × 40 m that could be used to quantify leakage for realistic wrinkle geometries.

Rowe (1998) had developed a simple equation to predict leakage through a hole in a GM coincident with (or adjacent to) a wrinkle (Fig. 10) which, in its simplest form (assuming no interaction between adjacent wrinkles), can be written:

$$Q = 2L[kb + (kD\theta)^{0.5}]h_d/D \quad [6]$$

where Q is the leakage (m^3/s); L is the length of the connected wrinkle (m); k is either the hydraulic conductivity (m/s) of the clay liner, k_L, if there is no AL or the harmonic mean of the CL and AL hydraulic conductivities, k_s, if there is an AL; $2b$ is the width of the wrinkle (m); $D = H_L + H_A$ is the thickness of the CL and AL (m); θ is the transmissivity of the GM–CL interface (m^2/s); and $h_d = (h_w + H_L + H_A - h_a)$ is the head loss across the composite liner (m). All of these parameters except the connected wrinkle length and wrinkle width are as previously discussed. What is needed to use eq. [6] is an indication of the likely values of L and $2b$. Thus, starting in 2006 an extensive study was initiated, including the construction of a full-scale test liner to provide field data regarding L and $2b$ for some North American conditions.





$$Q = 2L[kb + (kD\theta)^{0.5}]h_d/D \quad [6]$$

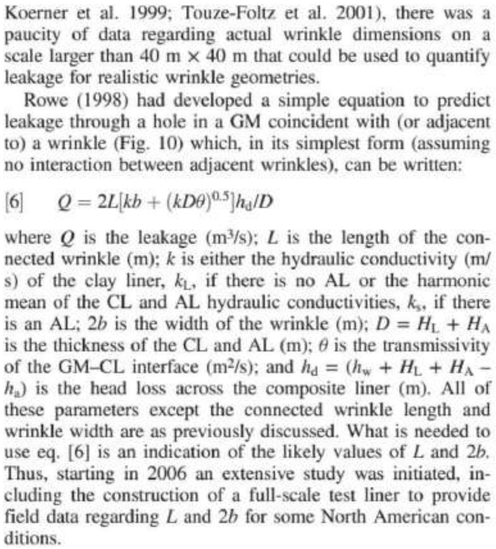
KUSILE 60yr ADF POLLUTION CONTROL AND CLEAN WATER DAMS
POTENTIAL LEAKAGE FLOW RATE CALCULATION

Outside Wrinkle

Length of connected wrinkle	L	50	m
Hydraulic Conductivity	k	5.00E-11	m/s
Width of the wrinkle	2b	0.1	m
Thickness of Clay and Atenuation layer	D	0.01	m
Transmissivity of GM-GCL interface	Theta	2.00E-11	m^2/s
Head loss across the composite liner	h_d	6.01	m
Leakage	Q	4.91E-07	m^3/s
Leakage	Q	42.38	litres per wrinkle per day
Assuming 5No wrinkles with holes per hectare	Q	211.92	litres per hectare per day

Below Wrinkle

Length of connected wrinkle	L	50	m
Hydraulic Conductivity	k	2.00E-10	m/s
Width of the wrinkle	2b	0.1	m
Thickness of Clay and Atenuation layer	D	0.01	m
Transmissivity of GM-GCL interface	Theta	2.00E-11	m^2/s
Head loss across the composite liner	h_d	6.01	m
Leakage	Q	1.58E-06	m^3/s
Leakage	Q	136.69	litres per wrinkle per day
Assuming 5No wrinkles with holes per hectare	Q	683.47	litres per hectare per day

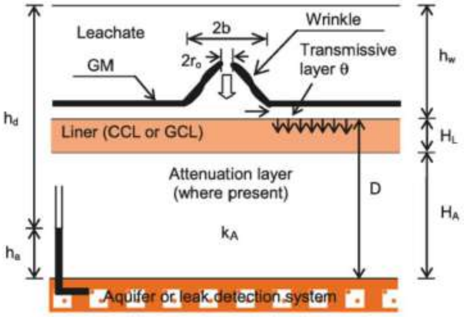


Koerner et al. 1999; Touze-Foltz et al. 2001), there was a paucity of data regarding actual wrinkle dimensions on a scale larger than 40 m × 40 m that could be used to quantify leakage for realistic wrinkle geometries.

Rowe (1998) had developed a simple equation to predict leakage through a hole in a GM coincident with (or adjacent to) a wrinkle (Fig. 10) which, in its simplest form (assuming no interaction between adjacent wrinkles), can be written:

$$Q = 2L[kb + (kD\theta)^{0.5}]h_d/D \quad [6]$$

where Q is the leakage (m^3/s); L is the length of the connected wrinkle (m); k is either the hydraulic conductivity (m/s) of the clay liner, k_L, if there is no AL or the harmonic mean of the CL and AL hydraulic conductivities, k_s, if there is an AL; $2b$ is the width of the wrinkle (m); $D = H_L + H_A$ is the thickness of the CL and AL (m); θ is the transmissivity of the GM–CL interface (m^2/s); and $h_d = (h_w + H_L + H_A - h_a)$ is the head loss across the composite liner (m). All of these parameters except the connected wrinkle length and wrinkle width are as previously discussed. What is needed to use eq. [6] is an indication of the likely values of L and $2b$. Thus, starting in 2006 an extensive study was initiated, including the construction of a full-scale test liner to provide field data regarding L and $2b$ for some North American conditions.



$$Q = 2L[kb + (kD\theta)^{0.5}]h_d/D \quad [6]$$

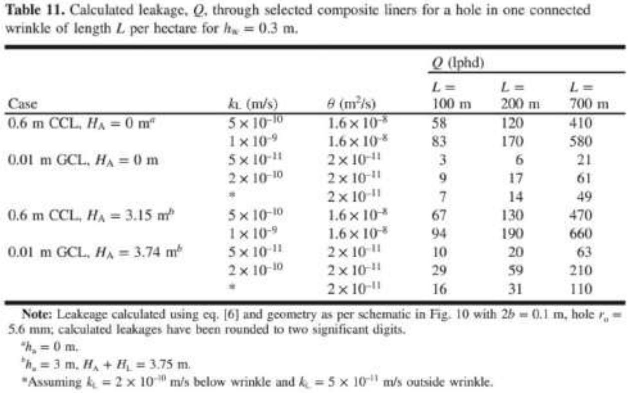


Table 11. Calculated leakage, Q, through selected composite liners for a hole in one connected wrinkle of length L per hectare for $h_w = 0.3$ m.

			Q (lphd)		
Case	k_L (m/s)	θ (m^2/s)	$L =$ 100 m	$L =$ 200 m	$L =$ 700 m
0.6 m CCL, $H_A = 0$ m^a	5×10^{-10}	1.6×10^{-8}	58	120	410
	1×10^{-9}	1.6×10^{-8}	83	170	580
0.01 m GCL, $H_A = 0$ m	5×10^{-11}	2×10^{-11}	3	6	21
	2×10^{-10}	2×10^{-11}	9	17	61
	*	2×10^{-11}	7	14	49
0.6 m CCL, $H_A = 3.15$ m^b	5×10^{-10}	1.6×10^{-8}	67	130	470
	1×10^{-9}	1.6×10^{-8}	94	190	660
0.01 m GCL, $H_A = 3.74$ m^b	5×10^{-11}	2×10^{-11}	10	20	63
	2×10^{-10}	2×10^{-11}	29	59	210
	*	2×10^{-11}	16	31	110

Note: Leakeage calculated using eq. [6] and geometry as per schematic in Fig. 10 with $2b = 0.1$ m, hole $r_o =$ 5.6 mm; calculated leakages have been rounded to two significant digits.

[a] $h_a = 0$ m.

[b] $h_a = 3$ m, $H_A + H_L = 3.75$ m.

*Assuming $k_L = 2 \times 10^{-10}$ m/s below wrinkle and $k_L = 5 \times 10^{-11}$ m/s outside wrinkle.

Document Title:	Stormwater Management Plan – Basis of Design
Eskom document no.:	366-511877
Document type:	Technical Report
Contractor Name:	EPCM
Revision no.:	0
Prepared by	K Matulovich
Package/System name:	60 Year Ash Disposal Facility
Contractor Name:	EPCM Bonisana

Rev	Date	Document Status	EPCM Reviewed	EPCM Approved	Client Review/ Approval
A	11-10-23	For Internal Review	M McLennan	K Matulovich	
				Signature	Signature
B		For External Review	M McLennan	K Matulovich	
				Signature	Signature
0	14-12-23	Issued For Use	M McLennan	K Matulovich	
				Signature	Signature

Table of Contents

Table of Tables

List of Figures

Appendices

Abbreviations

EPCM	EPCM Bonisana (Pty) Ld
SoW	Scope of Work
WUL	Water Use Licence
WULA	Water Use Licence Application
Eskom	Eskom Holdings SOC Ltd
Envitech	Envitech Solutions (Pty) Ltd
ADF	Ash Disposal Facility
PCD	Polution control Dam
CWD	Clean Water Dam
DWS	Department of Water and Sanitation
DEA	Department of Environment
DWAF	Department of Water Affairs
ROD	Record of Decision
MAP	Mean Annual Precipitation
SMP	Stormwater Management Plan

1. TERMS OF REFERENCE

Envitech Solutions (Pty) Ltd was appointed by EPCM Bonisana (Pty) Ltd in September 2023 to review existing design, develop Detailed Design for Phase 1 and consolidate all required information for the 60-year Ash Disposal Facility Design Report, to support the amended Water Use Licence Application.

2. BACKGROUND INFORMATION

2.1 Project Background

Kusile is a coal fired power station, owned and operated by Eskom Holdings SOC (Ltd). Construction of the power station began in 2008, and the first power production started in 2017. Currently 4 units of the planned 6 units have been constructed and have been connected to the national grid. Coal is burned in large boilers, and the heat generated in the burning of the coal drives the electricity generation process. The residual material from the coal-burning is ash.

The current ash and gypsum co-disposal facility has inadequate capacity to accommodate the expected volume of ash that will be generated over the life of the station. A new ADF is therefore required to accommodate ash disposal for the 60-year design life of the station. Dry-ashing facilities are required for the disposal of coal ash

The scope of new infrastructure that will be required includes the following:

- Conveyor system for the transportation of ash from the power station to the new ADF.
- Barrier containment / liner system for the ADF.
- Stormwater, contaminated water, and clean water management systems.
- Site services.
- Office facilities for a contractor to operate and maintain the facility.
- Support systems for dust suppression, irrigation, electrical, control and instrumentation.

The ADF is to be developed in ten phases, with the extension of the ADF at approximately five-year intervals. Eskom's objective is to construct and commission the proposed ADF so that ash produced by the power station over its design life of 60 years can be disposed in a safe and responsible manner.

An authorisation for the Ash Disposal Facility (ADF) was issued in 2015. In order to commence with the development, Eskom Holdings SOC Limited applied for a Water Use Licence for the applicable water uses, in terms of the National Water Act, 36 of 1998, as amended.

2.2 Scope of report

This report covers the design philosophy of the over-arching stormwater management system, and details of Phase 1 clean and dirty water management systems. The same philosophy would be applied to all following phases. This information will form part of the 60-year ADF Design Report, which will document information needed for project records and for the application for an amended Water Use Licence (WUL). The scope of this report includes the following:

- Background information on site conditions,
- General arrangement of the stormwater management system,
- Phase 1 ADF General arrangement,
- ADF Surface Drainage,
- Pollution Control and Clean Water Dams (PCD’s and CWD’s),
- River Diversion,
- Catchment analyses for runoff volumes and peak flows.

The scope of this report does not cover the mechanical and electrical aspects of the proposed facilities. Envitech drawings will indicate the mechanical interface points where applicable. A floodline analysis has been undertaken previously by other consultants and will be accepted and included **in the design**. Floodlines will not be re-calculated. The scope also does not cover design of the final landform surface water management system, which is understood to include final landform shaping, top platform contour drains, bench drains and down-chutes. The runoff from the final landform has been allowed for in the sizing and routing of stormwater channels.

2.3 Objectives of this Report

The objective of this Design Report is to document the design criteria and preliminary details of the proposed Stormwater Management System of the ADF.

This report precedes the Detailed Design Report for Phase 1 and the information will be carried forward into the final ADF Design Report.

The National Department of Water and Sanitation (DWS) issued a revised document in November 2020 pertaining to waste disposal applications, namely, "*National Environmental Management Waste Act Regulations 2013: Basal Barrier System Checklist for the Lead authority (National or Provincial Government) in Advance of Document Submission to Commenting Authority"*. Information in the Design Report will seek to fulfil these checklist requirements. The objective of this report, with respect to the checklist, is to address the required information relevant to stormwater management such as site geohydrology, catchment details and drainage design.

2.4 Design Engineer, Peer Review

The project Design Engineer will be Mr. Kris Matulovich, Professional Engineer with ECSA Registration number 20190729.

Contact Details: 011 425-2810, Email: kris@envitech.co.za

An internal peer review will be undertaken on the final design report.

2.5 The Developer's Representative

The Developer's Representative will be Mr Samuel Mahlangu of Eskom Holding SOC (Ltd).

Contact Details: Cell: -- Tel: 013 699 7271 Email: MahlaSaM@eskom.co.za

2.6 Description of the Waste Stream for Disposal

The waste stream will be both coarse and fine ash. The current disposal facility receives gypsum as well. Once the first phase of the ADF is commissioned, the waste streams will be separated and the 60-year ADF will receive ash only.

Whilst initial ADF design was based on similar ash from Kendal Power Station, testing on site-specific ash has since been undertaken. In addition to this, Envitech are currently testing ash samples for confirmation of foundation indicators, grading and permeability of both fine and coarse ash, as well as a mixture of both. The ADF will be a "dry-ash" facility. There will be no leachate re-circulation and no liquid co-disposal. In accordance with the proposed new *National Norms and Standards for Disposal of Waste to Landfill* (R636 of August 2013), the ADF will be designed to receive "Type 3" waste. The basal liner will be designed as a "Class C" barrier system.

2.7 Design History to Date

A conceptual design was prepared by Jones and Wagener Consulting Engineers in 2013 in support of the Environmental Authorisation application for the ADF. Thereafter a Zitholele JV project team developed the basic detailed design in 2018, which is deemed to be an approved concept. An Environmental Authorisation was granted by the DEA in July 2015.

EPCM have been tasked to review the design and oversee the ADF Detailed Design, which will also support an amended Water Use Licence Application. Envitech have in turn been appointed to carry out the Detailed ADF design for Phase 1, for EPCM.

Comments from the Record of Decision (ROD), together with comments from the EPCM review have been taken into account by Envitech. Some optimizations of the proposed stormwater management layout have been tabled, in consultation with Eskom and EPCM. These are outlined in Section 7.

The upslope stormwater cut-off drain is currently under construction under the "Early Works" Package.

3. RELEVANT LEGISLATION

The legislation applicable to the proposed landfill development includes, but is not limited to, the following:

- Constitution of the Republic of South Africa (No 108 of 1996)
- National Environmental Management Act (No 107 of 1998) - NEMA
- Environment Conservation Act (No 73 of 1989)

- National Environmental Management: Waste Act (No 59 of 2008)
- National Waste Management Strategy (NWMS)
- National Norms and Standards as published in Government Gazette Notices 634, 635 and 636 of 2013
- National Water Act (No 36 of 1998) – NWA
- National Heritage Resources Act (No 25 of 1999)
- National Environmental Management: Air Quality Act (No 39 of 2004)
- National Forests Act of 1998 (Act No. 84 of 1998) (DWAF, 2007)

4. SITE DESCRIPTION

4.1 Location and access

The Kusile Power Station is located 40 km from Bronkhorstspruit in Mpumalanga Province of South Africa. Access is off the R686 regional road that runs to the west and then north of the Site. The ADF site is currently open land that was under crop farming and animal grazing until relatively recently. The location co-ordinates of the centre of the ADF are tabled below. The ADF covers approximately 740 hectares, with the existing Power Station facilities located to the north.

Table 1: Site Location Co-ordinates

POINT	LATITUDE	LONGITUDE
Centre of ADF	25^0 57′ 39″ South	28^0 54′47″ East

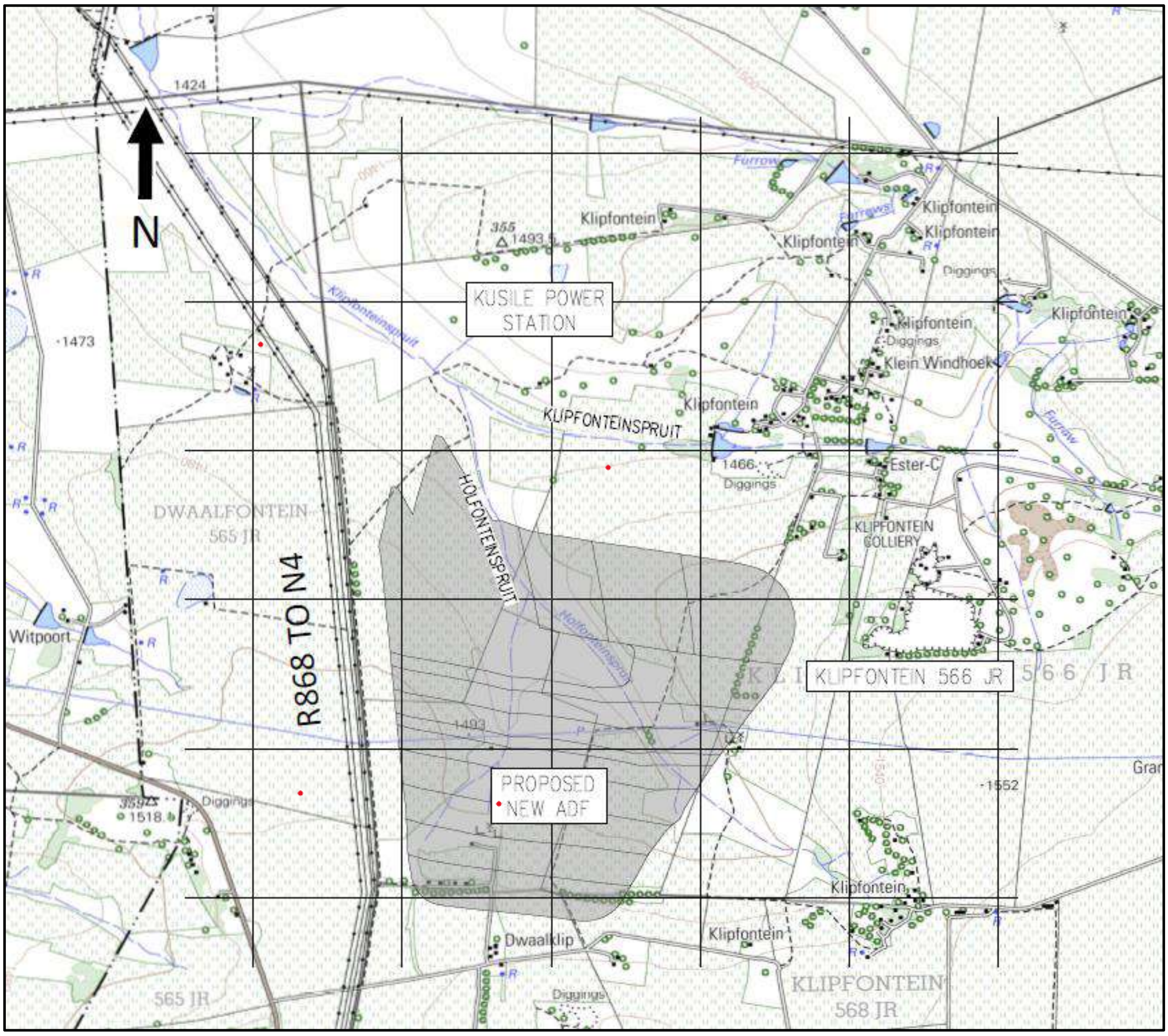


Figure 1: Locality Plan, Extract of 1:50000 Topographical Map No.2528DD

4.2 Topography

The site is characterised by slightly undulating topography, with elevations ranging from 1441 m.a.m.s.l. in the north to 1515 m.a.m.s.l. in the south. The main drainage channel on the site is the Klipfonteinspruit. This stream is fed by the Holfonteinspruit, a perennial stream from the south-west, and it's tributary from the south that run diagonally across the proposed ADF footprint. The average slopes are about 3% with a dominant slope toward the Klipfonteinspruit.

4.3 Quaternary catchment

The Kusile Power Station and ADF facilities fall within the Olifants Water Management Area. Within this area, the facilities fall within the B20F quaternary catchment. The Klipfonteinspruit passes between the Power Station and the ADF, flowing in a westerly direction, ultimately joining the Wilge River system.

The Holfonteinspruit and a tributary flow northwards to join the Klipfonteinspruit. The proposed ADF will be positioned over the Holfonteinspruit and it's tributary, as shown in Figure 1, Figure 2 below and drawings.

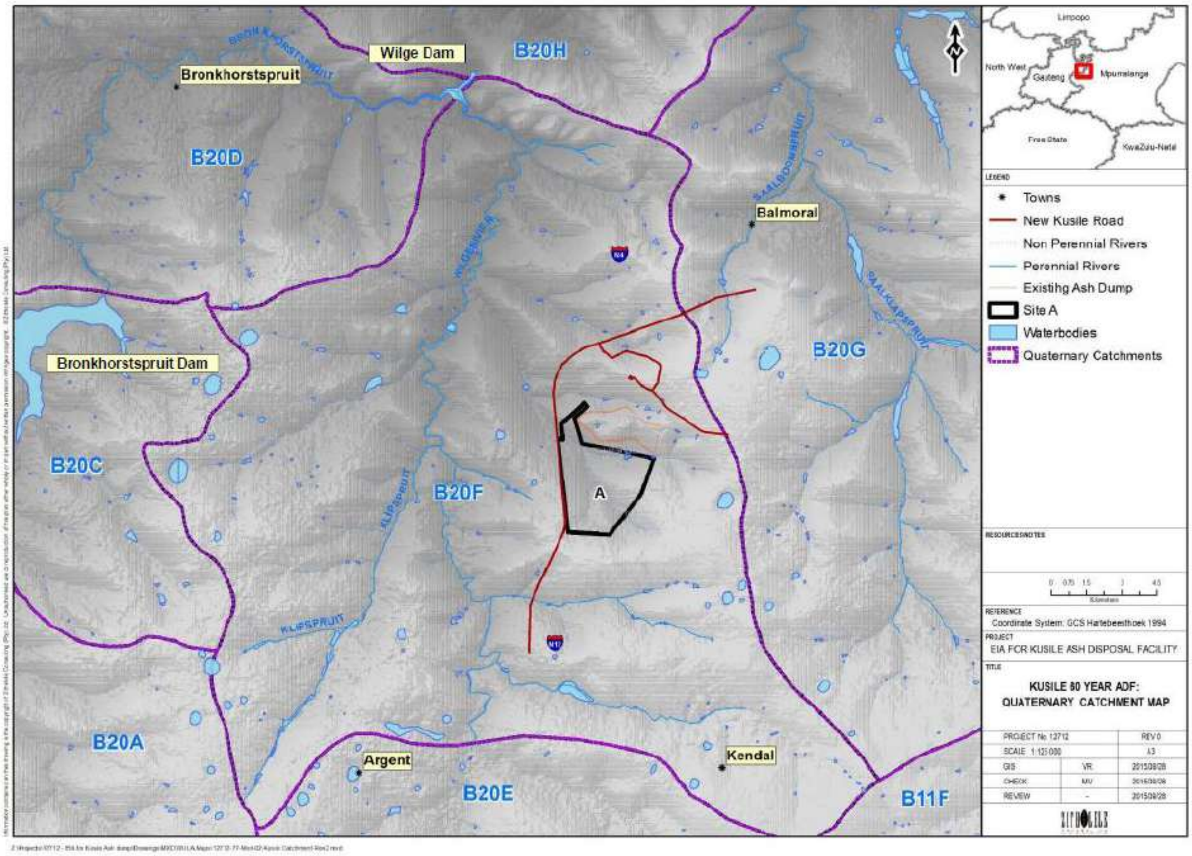


Figure 2: Quatenary Catchment, Zitholele 2014

4.4 Geology & Hydrogeology

The site geology and hydrogeology sections that follow serve only as an overview to provide context to the design approach. Site data will be described in more detail in the Design Report. There is extensive information documented in historical reports, and some of these are included under the References at the end of this report.

4.4.1 Regional Geology

According to the 1:250 000 geological map 2528 Pretoria the site is underlain by shale, shale sandstone, grit sandstone, conglomerate with coal in places near the base and top from the Ecca Group with tillite and shale from the Dwyka Group of the Karoo Super Group, with diabase intrusions that is of Vaalian and post-Mogolian age. Underlying these formations is the Silverton Formation shale's that are carbonaceous in places with hornfels and chert from the Pretoria Group. The region is not underlain by dolomite.

4.4.2 Site Geology

An area-specific generalised description of the geology is given below. This is a very generalised summary due to the variability of the tillite composition.

Stream bed where pipeline to be laid:

0.8 – 2.4m	Clayey Silty Sand - Colluvium & Alluvium
3.0 – 4.9m	Silty Clayey Sandy Gravel but varies - Residual Tillite/Shale

Groundwater seepage between 0.7 to 3.4m below NGL.

Excavatability – Soft to Intermediate

The PCDs area close to the Klipfonteinspruit:

0.5 – 1.6m	Sandy Gravel - Colluvium & Alluvium
3.0 – 4.9m	Silty Clayey Sandy Gravel but varies - Residual Tillite/Shale

Groundwater seepage between 0.7 to 3.4m below NGL.

Excavatability – Soft to Intermediate

The ADF Phase 1 area:

0.8 – 3.0m	Clayey Silty Sand - Colluvium & Alluvium

2.0 – 5.2m Clayey Sandy gravelly Silt but varies - Residual Tillite/Shale

No Groundwater seepage however can be present due the presence of the ferricrete layer.

Excavatability – Soft to Intermediate

4.4.3 Ground Water Levels

The water table on-site is relatively shallow and saturated conditions can be expected in low-lying areas. Groundwater seepage was recorded in various testpits over the site, at a depths varying from 0,7m to about 3,5m, which is generally the depth of weathered rock upper surface. The presence of ferruginisation in portions of the site is indicative of seasonal fluctuations in ground water seepage levels, which should be taken into account where sub-soil drains may be required. Groundwater flow is found to generally follow surface topography.

4.4.4 Topsoil

A loose structured topsoil layer of between 100mm and 300m was observed on the site. This material is to be set aside for final rehabilitation in the progressive closure of the ADF stages. The area north-east of the ADF could be considered for a topsoil stockpile area and / or the area south-west of Disaposal Phase 1 within the footprint of Disposal Phase 6 or further southwards. Toe drains around the upslope flanks should control runoff and prevent ponding against the stockpiles. Stockpiles should be shaped with a smooth, sloping platform to allow runoff, and smooth side slope at natural angle of repose. Topsoil stockpiles should not exceed 3m in height.

4.5 Seismicity

According to the SANS Standard SANS 10160-4:2017 the Kusile site falls within Seismic Risk Zone 1 with a value of 0.100 m/s^2.

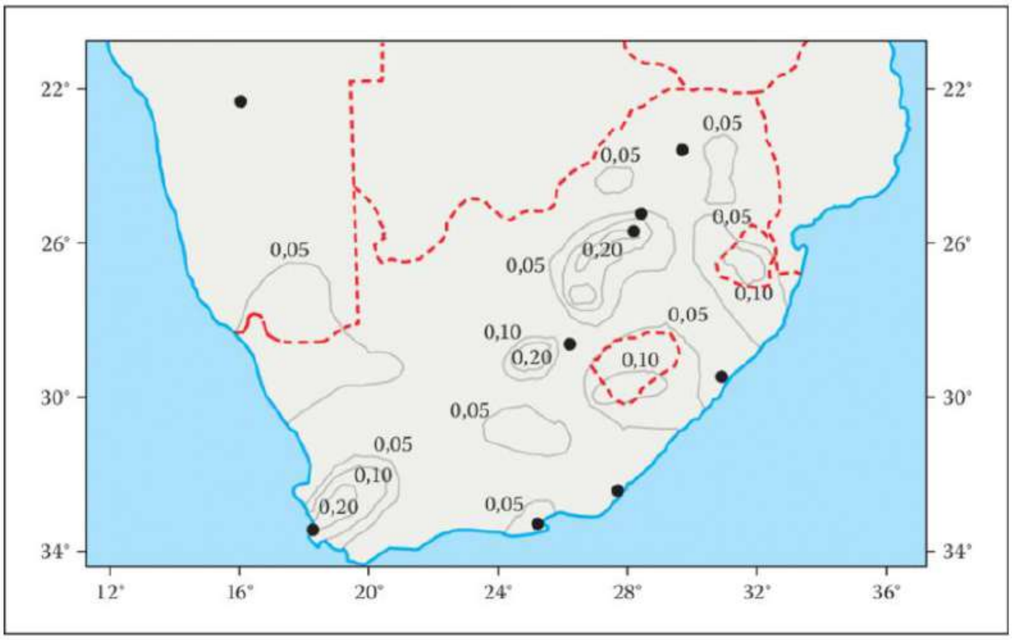


Figure 3: Seismic Risk Values

4.6 Existing Services and Structures

There are a few remnants of farm buildings, but no significant buildings on the property. There are old stock fences here and there on the ADF site and alongside the R686 road running along the western boundary. Security fences surround the Power Station on the northern bank of the Holfonteinspruit. A boundary fence runs on most the eastern side of the ADF property, on the boundary with New Largo Mine and the Phola conveyor.

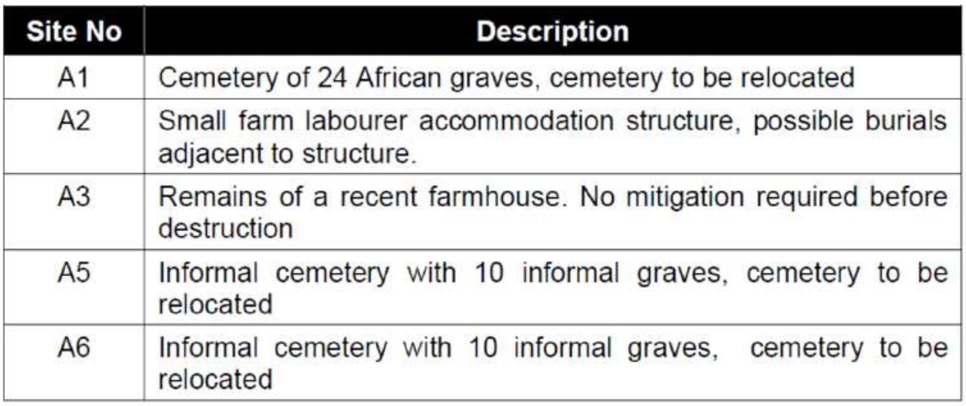
A 400 kV powerline runs alongside the R686 road to the west and a 22 kV overhead powerline traverses the site east-to-west about midway across the ADF site, which will have to be re-routed. The planned new route powerline route is shown on Drawing 366-511891.

There are a few gravel farm roads across the site and around the cut-off drain construction works.

4.7 Historical Graves

Graves have been found on site, shown in the figure below.

Table 2: Graves and Farm Structures



Site No	Description
A1	Cemetery of 24 African graves, cemetery to be relocated
A2	Small farm labourer accommodation structure, possible burials adjacent to structure.
A3	Remains of a recent farmhouse. No mitigation required before destruction
A5	Informal cemetery with 10 informal graves, cemetery to be relocated
A6	Informal cemetery with 10 informal graves, cemetery to be relocated

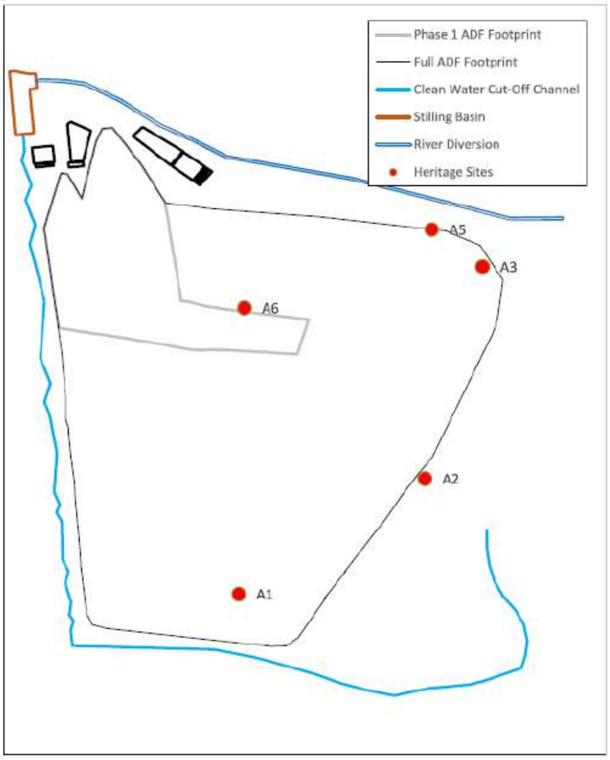


Figure 4: Location of Graves

5. REGIONAL CLIMATE

The region falls withing the Highveld sub-tropical climate zone. The summers are characterised by hot, humid and wet conditions whilst the winters are characterised by lower tempeartures and Monthly precipitation levels are highest in January at approximately 124mm, whilst June is the driest month with about 7mm average precipitation.

5.1 Mean Annual Precipitation

The area receives about 697mm per annum.

5.2 Rainfall and Evaporation Figures

The evaporation far exceeds precipitation and is expected to be around 1500 mm per annum. Mean monthly figures are provided below.

Table 3: Monthly Rainfall and Evaporation

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEPT	OCT	NOV	DEC	TOTAL
RAIN (mm)	**127,8**	**98,8**	**88,3**	**41,7**	**17,7**	**7,5**	**8,0**	**9,1**	**18,5**	**69,5**	**110,9**	**114,7**	**697**
S-Pan Evaporation	**166,1**	**143,5**	**135,4**	**105**	**85,3**	**67,4**	**74,6**	**102,4**	**139,5**	**163,1**	**160,3**	**174,9**	**1524**

5.3 Weather stations

The closest South African Weather Service (SAWS) weather stations are as follows in Table 3 below. The data from these stations are sourced automatically by the hydrology software and are also input into manual calculations such as the Rational Method.

Table 4: Weather Station Data

Station	Year of Records	Co-ordinates	Distance from ADF
Wilgerivier	94	25^{O} 49′ S; 28^{O} 51′	< 1 km
Bronkhorstspruit	92	25^{O} 48′ S; 28^{O} 44′	12.7 km

Blesbokfontein	35	25^{O} 57′ S; 28^{O} 48′	15.4 km
Kleinwater	39	25^{O} 48′ S; 29^{O} 2′	19.9 km
Waaikraal	49	25^{O} 59′ S; 28^{O} 40′	26.8 km
Hartebeestspruit	59	25^{O} 46′ S; 29^{O} 7′	29.3 km

6. STORMWATER MANAGEMENT PLAN

6.1 Design Philosophy

Construction of the ADF will progress in 12 Phases, at approximately 5-year intervals.

Disaposal takes place on a prepared area consisting of a perimeter berm, a lining system on the base, a drainage system over the lined base and a layer of coarse, permeable material to protect the liner. Disposal starts at the lower elevation nearest the conveyors, and develop eastwards and then southwards.

At each progressive phase of the ash disposal, temporary upslope access roads and stormwater diversion measures will be installed at the upslope toe of the dump. The measures such as diversion berms, toe drains and attenuation dams would be demolished at the completion of an ash disposal phase, and then similar structures would be installed upslope of the next phase of ash disposal.

The Stormwater Management Plan forms an integral part of the Pre-deposition Works. Whilst not all stormwater management facilities are required for first stages of ash disposal, the planning and design has to be completed up-front. It should be noted that Phase 1 of the ash disposal and related infrastructure is located down-slope of all later phases, and therefore Phase 1 design must allow for the required final-stage capacity of drains and CWDs / PCD’s. The elements making up the stormwater management system are described below.

6.2 Water Management Components

The storm water management system is governed by GN 704 requirements and contains the following key infrastructure:

- Pollution Control Dams (PCDs): 1a, 1b and 2,
- Clean Water Dams (CWDS): 1 and 2,
- 2 temporary Clean Water Attenuation Dams
- 3 permanent Clean Water Attenuation dams,
- Network of contaminated stormwater collection channels,
- Clean stormwater diversion berms,
- Clean stormwater drain,
- Stilling basins,
- River Diversion.

6.3 Upslope cut-off drain

The regional catchment upslope of Kusile ADF is being diverted around the ADF by the clean upslope stormwater cut-off drain on the Power Station property boundaries that is currently under construction. The drain is indicated on Figure 5 above.

6.4 Stream Diversion

The ash dump footprint straddles the Holfonteinspruit and it’s tributary. These streams will each be routed through a large diameter reinforced concrete pipeline under the ash dump. The two streams join into one pipeline, about halfway down the ash dump footprint, which then discharges through an outlet structure back into the original stream at the convergence with the Klipfonteinspruit.

The pipelines have been designed to align with the centre of the drainage lines as far as possible, and close to the existing natural ground level. As the horizontal alignment is determined by the existing topography, the pipelines do not run in a straight line. So, in addition to a cast-in-situ concrete junction box at the joining of the two streams, there would be cast-in-situ junction structures at bends. The number of junctions has been optimised, taking both horizontal and vertical alignment into consideration.

The pipelines run in a straight line and on a single gradient between junction boxes.

6.5 Attenuation Dams

Attenuation Dams will be required, to limit the flow in the streams and so facilitate construction activities downstream. Temporary attenuation dams (Dams 1 and 4) will only serve to divert runoff during construction downstream and will not require a controlled underdrain outlet. Temporary attenuation dams will spill into a temporary diversion channel.

The permanent attenuation dams (Dams 2, 3 and 5) will have spillways as well as controlled-outlet underdrains which will feed into the stream diversion pipelines. The permanent attenuation dams also function as upslope water storage and diversion during pipeline construction. Once a pipeline section has been constructed, the attenuation dams will lower the peak flow in the pipeline (although pipes will be sized to accept all flow) and provide upslope storage when required. The permanent dams upstream of Ashing Phase 3 would be demolished after approximately 15 years. The sequencing of the attenuation dams is described on drawing number 366-511846, *General Arrangement of Attenuation Dams*.

Attenuation Dam 3, will be the largest dam and situated on the Holfonteinpsruit, with a maximum capacity of 100788m3, and a wall height of about 7,8m. This dam will be constructed with Ashing Phase 6 and will remain forever.

At the end of life of the ADF site, the remaining catchment to the east will drain to a final Attenuation dam, and spill over into clean drains that discharge to the environment both west and north-east of the ADF. At the final ashing phases i.e. from Phase 10, the catchment for the tributary (the western pipeline) will reduce to almost nothing. A small intake structure at the pipe inlet would suffice, in lieu of an attenuation dam.

Figure 5: Attenuation Dam Layout (extracted from Drawing 366-511846)

6.6 Diversion Channels

Dams No. 1 and 2 shall have diversion by-pass channels constructed leading from their spillway full supply level draining to the natural river area and clean water channel respectively. Dam No. 4 channel will connect to Dam no. 3. Diversion channels will be earth lined as shown in **Figure 6** below and drawing 366-511924.

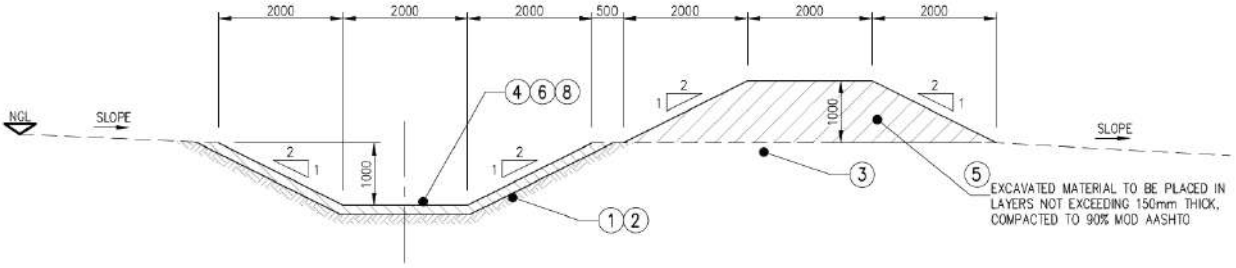


Figure 6: Clean water Diversion Channels

6.7 Perimeter Drainage Channels

A system of three open drains is proposed for the ADF surface water drainage, namely a clean outer drain direct-discharging to the environment, a middle dirty drain for polluted water, and an inner “dirty-to-clean” drain.

The clean drains running east-west to the south (up-slope) of the ash dump will be deemed temporary, and when the footprint extends, these drains will be demolished, and duplicated further upslope, south of the following phase of the ash dump. With each phase the permanent clean drains to the east and west of the ash dump will be extended and joined to the current temporary southern drain. The Phase 1 drains are shown on drawing 366-511895.

The dirty-to-clean drain is the first drain to be utilised, collecting contaminated run-off from the first area receiving ash, i.e. from the ash dump surface and side slopes where that runoff is not contained behind the perimeter berm and draining into the leachate collection system in the ADF base. Water collected in the dirty-to-clean drain ultimately drains to a PCD.

As the next ADF phase becomes the active disposal area, the first phase is to be progressively covered and rehabilitated. The dirty drain is to be constructed to serve the second active ash disposal area, whilst the first leg of the dirty-to-clean drain is by-passed, allowing the first leg to become isolated from upstream runoff, and become progressively cleaner.

Eventually, when serving the final rehabilitated landform, this innermost dirty-to-clean drain becomes the final clean toe drain. The concept is described by way of a schematic figure shown below.

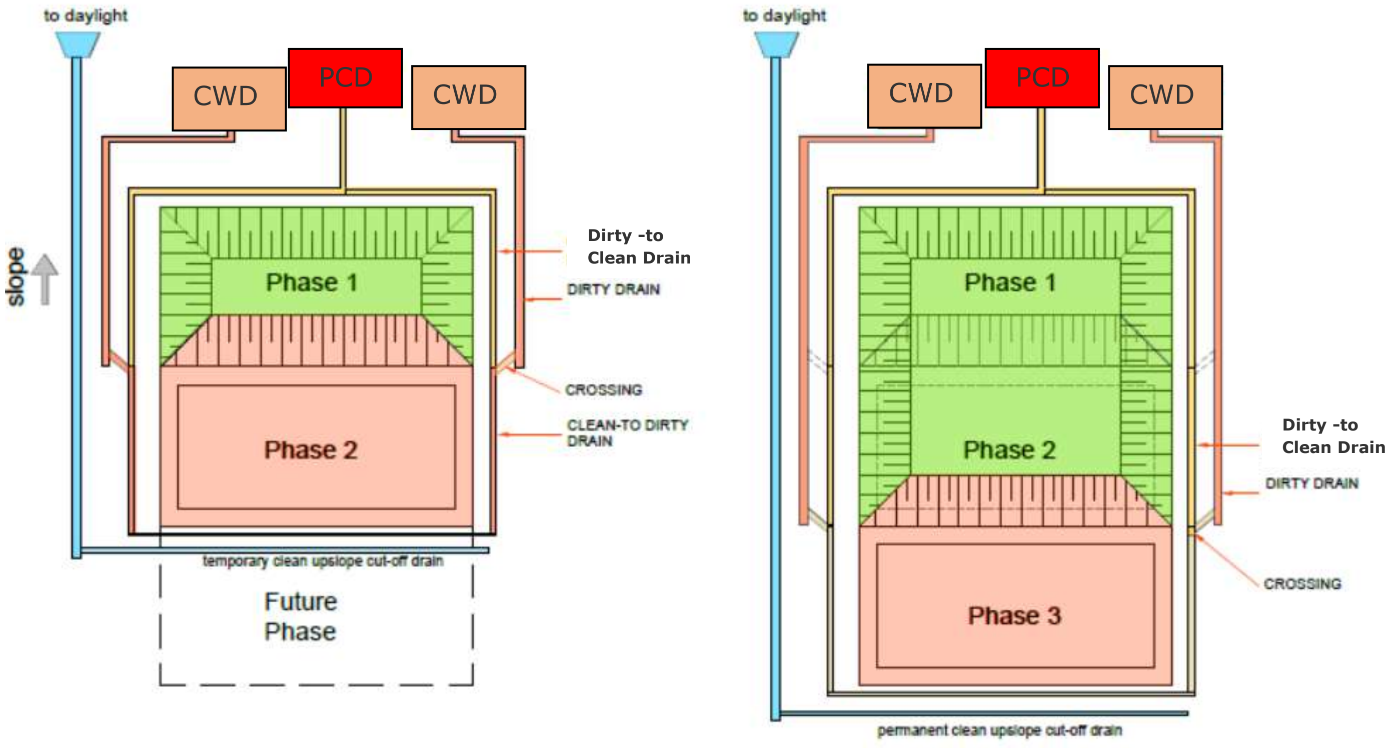


Figure 7: Schematic Open Channels Arrangement

6.8 Polluted Water System

The internal drainage system under the ADF basal barrier collects sub-soil water. The internal drainage system above the basal barrier collects leachate from the base of the ash dump. Both these networks are perforated HDPE pipes, that gravity-drain to main collector pipes, which would be solid wall HDPE pipes. The leachate collector pipes will convey the leachate out through the perimeter berm with a sealed pipe penetration. The pipe will discharge the leachate into the dirty drains, which are open trapezoidal, concrete lined drains on the outside perimeter of the ADF.

In the central low-lying valley areas the leachate collector drainage pipes will run alongside the central stream diversion pipeline, although still above the barrier system. The central collector pipes will ultimately discharge into the northern dirty water drains.

The polluted channels flow northwards to discharge to the Polluted Water Dams (PCDs). Once monitoring of water confirms that water quality has improved sufficiently, water from the “dirty-to-clean” channel would be diverted to the Clean Water Dam (CWD).

Sub-soil drains discharge into the environment via monitoring manholes or into a pipeline junction box.

6.9 Clean Water Dams and Pollution Control Dams Complex

A series of lined dams will be constructed. For Phase 1 of ash disposal, only PCD1a, PCD1b and PCD2 will be constructed, along with CWD1 and CWD2 and Road PCD. The latter will service the conveyor platforms and roads between the conveyors.

There are no process flows entering into the dams from the power station while process flows from Pump Station 1 and 2 (serving the PCD’s) can be pumped to the existing ADDD dam servicing the existing 10 year co-disposal disposal facility. All dams shall be constructed with a spillway which will be channeled into the river diversion. The dams will have a freeboard of 0.8 metres above their normal operating level and have been sized to not spill more than once in 50 years, as per Government Notice (GN) 704. Larger dams have been designed with additional freeboard to allow for wave action at full supply level.

The dam sizes have been based on current reticulation philosophy, after discussion with Eskom, and have similar capacities to previous design where dynamic time-step modelling was undetaken. Once all detailed ADF design is complete, a dynamic model of the entire complex will be re-run to confirm final spillway levels and freeboard.

Dust Suppression water will be extracted from PCD 2, to be sprayed on open ash areas of the ADF. A detailed layout of the dams complex and the interconnections via a pumpstation will be detailed in a separate operational report.

6.10 Preliminary Closure Plan

The preliminary closure plan includes for the ongoing capping and rehabilitation of the ADF as each portion of the ADF is completed to its final height. Capping will include a soil cover, topsoiling and greening with indigenous grasses.

The contaminated water dams would need to remain fenced, and the water quality tested for some time post-closure. Eventually, if the ADF runoff is found to be clean, the dam barrier layers could be removed and disposed on a suitable landfill, and the dams should be filled in. The final attenuation dam and upslope cut-off drains would continue to function, and a single perimeter toe drain would remain for clean run-off routing to a controlled discharge point.

Stilling basins and erosion protection at discharge points would remain post closure. During the life of the site, these structures would be monitored, maintained and re-furbished as necessary.

7. LATEST DESIGN OPTIMISATION

A few design optimisations have been proposed at the time of this report, which have been discussed in recent meetings with EPCM and Eskom representatives in October 2023. Those affecting stormwater management are as follows

- Fewer attenuation dams are proposed. In considering the order of works and constructability, concerns were raised as to how the stream is diverted to allow for the construction of permanent attenuation dams and diversion pipelines. It has been proposed to construct temporary attenuation dams higher up the valleys at the start of the project. This elevation allows for diversion of the streams, from the initial dams, into by-pass drains around the ADF footprint. Although the first construction works would now reach further up the valleys, and a small corner of the ash dump in Phase 1 will be sacrificed, fewer dams are required. This is both a cost saving and improved diversion of upslope runoff.

- All dams in the PCD and CWD complex are to be divided into two compartments, as requested by Eskom.
- The silt traps at the dams have been revised to allow a longer flow path through the traps. It is believed that maintenance would be easier as well.
- A buttress wall is proposed for part of the ADF eastern flank, to be built up as the ash disposal progresses. The wall would be lined on the inside as per the basal barrier layers, with special anchorage and stability measures placed with each progressive lift. Unlike the perimeter berm, the butress wall would rise about 25m at its highest point. The function of the buttress wall would be a contingency against the build-up of stormwater against the ash dump, in the event of stream diversion pipe blockage/ failure. During risk assessments it was previosuly proposed to build a dam on the eastern boundary with enough elevation on the dam wall to force the overspill into drains around the ADF. The required dam wall would have a maximum height of 30m and a length of over 2.5 km. With the proposed optimisation, the ADF would now instead only require an inlet attenuation dam of 7m height, and should the dam or pipeline fail, the ADF would be protected.
- The stream diversion pipelines are proposed to not exceed a diameter of 2,5m.

8. ORDER OF WORKS

8.1 12m Up-slope cut-off drain.

The upslope cut-off drain is currently under construction, and this will divert all clean upslope, natural run-off around the ADF footprint, to discharge north-west of the ADF into the Klipfonteinspruit. The up-slope run-off is thus returned to the pre-development stormwater destination. This drain effectively reduces the stormwater catchment area to be managed at the ADF site. The drain is an unlined trapezoidal drain, 12m wide at the top.

8.2 Relocation Of Services

The relocation of existing services, such as a low-voltage powerline traversing the ADF footprint in an east-west direction will need to planned for timeously. The absence of other services, including buried services needs to be confirmed as well.

8.3 Topsoil stripping, Access

The extent of topsoil stripping for Phase 1 of the ADF and related infrastructure such as drains and dams, and the placement of the resulting topsoil stockpiles needs to be agreed early on. A materials utilisation report is to be consulted to assist in the planning. Related to the excavation and stockpiling of site materials is the planning of site access for the construction, for Ash Disposal Phase 1 and future phases. Double handling of material should preferably be avoided. Construction access should not be hindered.

8.4 First Attenuation Dams

The diversions of the Holfonteinspruit and tributary which traverse the ADF footprint need to be installed before the ADF can be constructed. In turn, upslope attenuation dams need to be in place before the diversion pipeline can be installed. Therefore, with regard to the Stormwater Management Plan, one of the first priorities would be the construction of attenuation dams in the valleys upstream of the proposed pipelines. The order of construction of the attenuation dams is addressed on drawing 366-511846.

8.5 Stream Diversion, ADF basal drainage pipes

The Holfonteinspruit and it’s western tributary traverse the ADF footprint, joining together under Phase 1. Some of the collector pipelines of both the sub-soil and leachate herringbone pipe systems will run in the trench corridor of the stream diversion pipelines. The main collector pipes from these systems will be solid wall HDPE pipes, placed on the outer sides of the stream pipeline. There will be no inter-connection of the ADF internal drainage collector pipes into the stream pipeline. However, the bedding and installation of these pipes should take place simultaneously with the stream pipeline.

8.6 ADF drains, earthworks and the barrier system.

After cut-to-fill earthworks and the formation of the perimeter berm, the sub-soil drainage network should be installed. This would be followed by the installation of the barrier system. A drainage layer would be placed on top of the HDPE geomembrane and protection layer. The leachate drainage system would then be placed within the drainage layer.

So, considering the order of works, once the central stream diversion pipeline is in place, the sub-soil and leachate main collectors would be installed alongside it, on both sides. Then PCD2 and CWD1 are to be installed, and only then could the perimeter external drains be installed. The external perimeter drains discharge to the PCDs.

It is imperative that the dirty drains and the PCD have already been commissioned prior to the leachate collection network being in operation. From the first rainfall event, runoff from the ADF’s active footprint must be diverted to a PCD via the leachate collection system and open drains.

8.7 Clean Water Stormwater Drain

A local clean-water cut-off drain will wrap around Phase 1 of the ash dump footprint. The drain portions running east-west to the south (up-slope) of the ADF will be deemed temporary, and when the footprint extends, these drains will be duplicated further upslope, south of the following Phase of the ADF.

In principle, the clean cut-off drains flow around the ADF into permanent clean water drains both on the east and west of the ADF to discharge into the natural stormwater regime. These clean drains are not to be confused with the existing 12m wide upslope cut-off drain on the site boundary, which is much larger and channels the off-site catchment runoff around the Kusile site.

8.8 Construction of PCDs

For Phase 1 ash disposal only PCD 2 and CWD2 are required first, followed by PCD 1a, 1b and CWD-1. The PCDs and CWDs north of the ADF are all positioned in a relatively narrow footprint and future constructability within this space needs to be taken into account.

8.9 Conveyor Corridors

Contaminated runoff from the conveyor approach area will be channelled to the Road PCD.

Works by others not part of the report scope, but also pertaining to the Stormwater Management Plan, include the conveyor crossings of the Klipfonteinspruit. This comprises two sets of culverts bridges.

9. TECHNICAL DESIGN

9.1 Influence of Pioneering Layer over the barrier System

Initially, a pioneering layer of ash is to be placed over the liner by means of end tipping and spreading to protect the installed liner. No heavy vehicles may drive into the cell before the pioneering layer is placed. A small, light machine is to place the material in front of itself, forming a "road", and spread material outwards from this road.

The design calls for a coarse material as the pioneering layer. Eskom would prefer to utilise ash as soon as possible and therefore proposed using coarse ash for the pioneering layer and drainage layer. However, once covered with even a small amount of ash, the ADF footprint is considered to be polluted. Any water through the leachate drainage layer would then have to be contained in a PCD. Therefore the following is proposed.

The first stages of the ash dump will have a coarse sand drainage layer. Where the layer is to be exposed for some time, a sacrificial layer of sandy soil is to be placed over the sand. Where separation is possible, runoff from unused areas of the ADF could then be deemed clean and runoff diverted to the clean water system. Later stages, probably at Package 2 stage, the drainage layer is to be coarse ash.

9.2 Catchment analyses

The stormwater water management systems normally associated with a waste site address three types of runoffs which are:

- uncontaminated upslope run-off,
- contaminated run-off from the landfill surface and side slopes, and
- leachate generated within the waste body.

The site was divided into sub-catchment areas to delineate clean and dirty systems. Runoff calculations were performed for the sub-catchment areas in order to determine the size of the required drains.

9.2.1 Uncontaminated upslope run-off analysis methodology

All upslope run-off water must be diverted away from the waste, to prevent water contamination and minimise leachate generation. The uncontaminated upslope runoff will be prevented from entering the landfill facility area by means of trapezoidal diversion drains along the higher southern, western and the eastern side of the landfill. These drains will end in a velocity reducing structures from where the water will daylight north-east and north-west of the facility.

Unique to this site are two central valleys of the Holfonteinspruit and its western tributary that flow generally northwards and join together. As described in sections above, the streams will be diverted through a large diameter reinforced concrete pipe. After the installation of the clean cut-off drain on the southern and western boundaries, the remaining upslope run-off would be the on-site catchment and the catchment beyond the ADF boundary to the east, currently the New Largo coal mine property.

The catchments reporting to each pipeline were delineated, and the runoff volumes were calculated. In addition to runoff volumes through each leg of the pipelines, peak flows were calculated. A design storm of 1:200 year recurrence interval was utilised.

Calculations were carried out using the computer programme Storm and Sanitary Analysis (SSA), which is an Autodesk product and compatible with Civil3D design and drafting software. Results are included in Appendix A. The output was compared to the results obtained in the conceptual designs thus far, by others, and found to be in the same order.

From the peak flows calculated, various sizes of pipe and channel could be tested with the Manning’s Equation. Through an iterative process in Excel, a suitable pipe size could be chosen. For concrete trapezoidal drains, a Mannings n value of 0,015 was chosen, and for the pipe, a n value of 0,012.

The required size of the eastern leg pipeline was found to be a 2,0m diameter pipe. The required size of the western leg pipeline was found to be a 1,5m diameter pipe, however a

2m diameter pipe will be installed. The last, combined portion was required to be 2,5m diameter pipe.

9.2.2 Contaminated surface run-off analysis

Surface run-off from the landfill perimeter berm outer slopes, roads and waste handling areas are considered to be potentially contaminated and should not enter natural drainage courses. Contaminated run-off is to be directed towards an open concrete lined trapezoidal drains along the outer toe of the ADF perimeter berms.

In sizing these drains, the worst-case scenario was considered and found to be the sceanrio of the largest lined-and-contaminated area for the duration of diposal. This would be the case when a new phase is lined, and the previous phase is not yet capped.

A combination of Phases 1 and 2 was considered at the worst case, and runoff volumes and peak flows were calculated based on this catchment, and as per methodolgy described in Section 9.2.1.

9.3 Channel Design

The dirty-to-clean drain and the dirty drain would all be concrete lined trapezoidal channels with a minimum base width of 1m and minimum depth of 0,5m. Concrete is to be a minimum 100mm thick and 35 MPa strength. The channels are to have mesh reinforcement at of mesh ref. 193. Channels are to be cast in alternate panels, not exceeding 4,5m in length. Prior to placing concrete, in-situ base soil should be ripped and recompacted and a sand blinding layer placed if necessary.

The clean clean cut-off drain and attenuation water diversion drains are to be lined with selected clayey site soils 150mm thick, compacted. Topsoil-rich soil to be lightly tamped over the surface.

Table 5: Channels Flow Parameters

Channel Shape	Trapezoidal
Manning Factor	0.013
Minimum Flow Velocity	0.7m/s
Maximum Flow Velocity	4.5m/s
Freeboard	At least 10%

9.4 Pipeline Design

Previous designs called for a 3m to 3,2m pipe diameter based on maintenance equipment size, not flow. Envitech design engineers have put forward that sizes over 3m diameter have to be specially designed and probably constructed on site at considerable cost. Considering advances in drone and mobile camera surveillance systems, and in remote control cleaning equipment, it is believed that large size inspection equipment should not be routinely necessary.

It is proposed that the concrete pipe be lined in the lower half with an HDPE cast-in sheet. There are products like this specifically manufactured with anchor knobs on one side, which act as lugs when cast into a concrete surface. The lining would protect the pipe but still allow visual inspections of the pipe soffit, which is where stress cracking and spalling would be expected to show first, should damage occur.

Pipe loading calculations and corresponding bedding and founding design will be discussed in a separate report, once research is completed. A specification on the pipe is that of 100D jacking pipes from Rocla or similar approved by the Engineer. As discussed from the analyses above, pipe section diameters are to be 2m and 2.5m.

Table 6: Pipelines Length and Diameter

Pipe Diameter	Concrete pipes - SABS 677
Tributary 2m diameter	1760.42m
Holfonteinspruit 2m diameter	1682.31m
TOTAL LENGTH 2m DIAMETER	**3442.73m**
Holfonteinspruit lower, 2,5m diameter	1380.16m

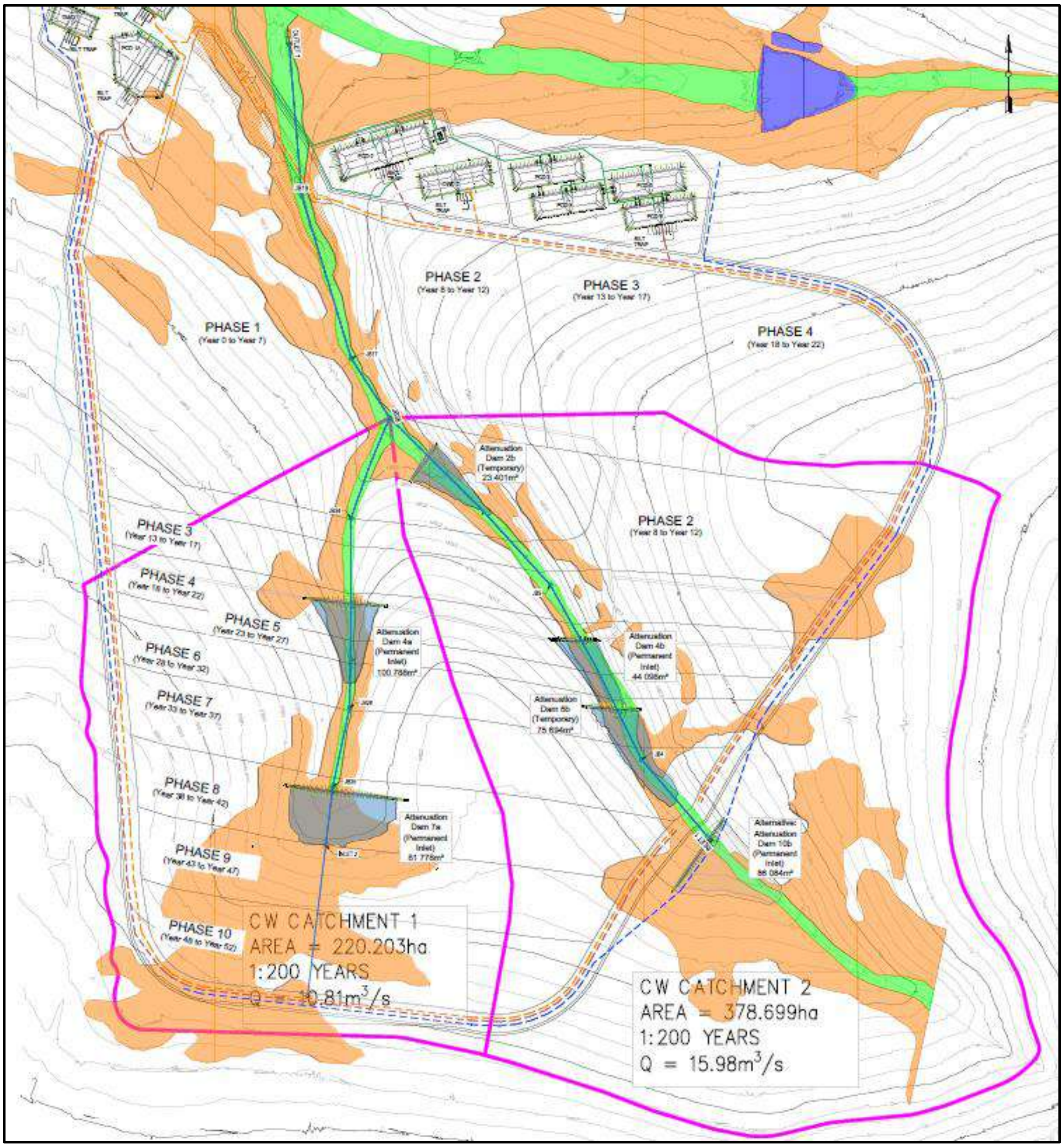


Figure 8: Stormwater management layout around catchment area

The Basis for Design input is tabled below. In addition to runoff volumes through each leg of the pipelines, peak flows were calculated. A design storm of 1:200 year recurrence interval was utilised.

Table 7: Pipelines Catchment Data

Catchment	1	2
Total Catchment Area	220.203 ha	378.699 ha
Mean Annual Precipitation	697mm	697mm
SAWB MAP Station	Wilgerivier (SAR)	Wilgerivier (SAR)
Impervious Area	5%	5%
Average Slope	2.55%	1.57%
Design Flood	1:200 (RMF)	1:200 (RMF)
Overland Manning’s factor		
Pervious fraction (n)	0.40	0.40
Impervious fraction (n)	0.13	0.13
Infiltration Settings		
Initial infiltration rate	60.0 mm/hr	60.0 mm/hr
Final infiltration rate	10.0 mm/hr	10.0 mm/hr

Table 8: Pipelines Peak Flows

Catchment	1	2
Return Periods (yrs.)	Q200	
Peak Flows (m^3/s)	11.31	13.81

Table 9: Pipelines Flow Parameters

Pipe Type	Concrete pipes - SABS 677
Minimum Pipe Class	100D
Pipe Manning Factor	0.013
Minimum Flow Velocity	0.7m/s
Maximum Flow Velocity	4.5m/s
Freeboard	At least 10%

9.5 Attenuation Dams Capacities

Storage capacities of each attenuation dam are tabled below.

Table 9: Attenuation Dam Capacities

Dam Number	Wall Height	Type	Capacity (m^3)
Attenuation Dam 1	4,3	Temporary wall and spillway	23 400
Attenuation Dam 2	5,5	Permanent with underdrain	81 780
Attenuation Dam 3	7,8	Permanent with underdrain	100790
Attenuation Dam 4	6,4	Temporary wall and spillway	75 690
Attenuation Dam 5	5,8	Temporary wall and spillway	44 098

9.6 Pollution Control Dams

The Phase 1 dams are mainly clustered in two areas, as can be seen in Figure below, an extract from Drawing 366-511845 Phase 1 General Layout. PCD-1A, PCD-1B and CWD-1 are situated north-west of the ADF. PCD-2 and CWD-2 are situated north-east of the ADF, receiving the water from channels on the eastern toe of the ADF.

Pump Stations 1 (west) and 2 (east) are indicated as well.

As can be seen, each dam is divided into two compartments. This allows the dam to remain functional when one half is out of service, for repair or maintenance, for example.

Each compartment of the PCDs has a dedicated silt trap, i.e. two silt traps per dam. The CWDs only require one silt trap each that would serve both compartments of the dam.

Future dams will be added east of CWD-2 in following phases.

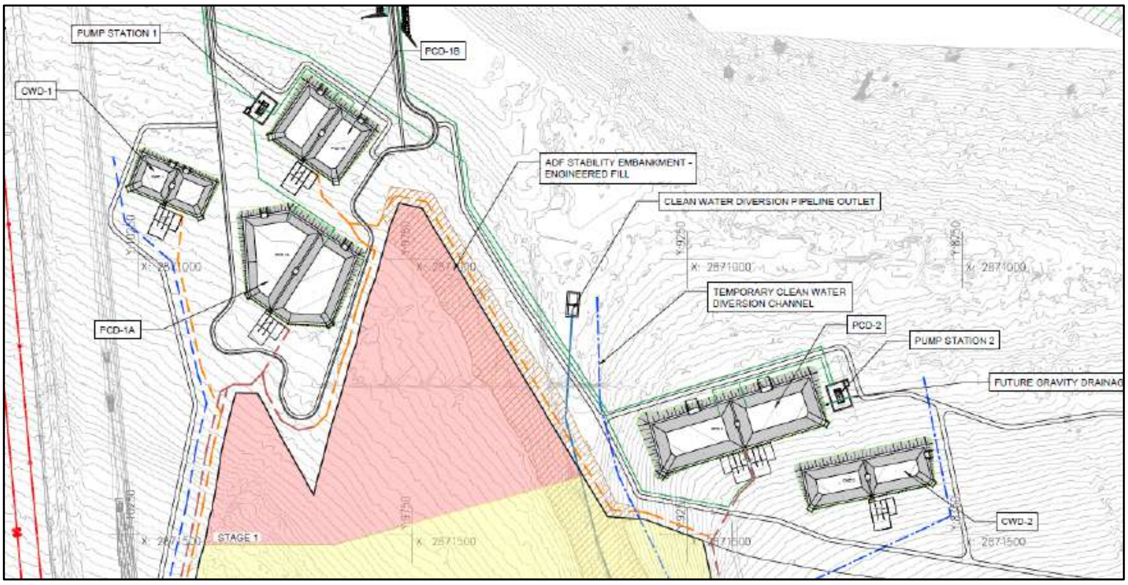


Figure 8: Layout of Dams North of ADF

The Road Dam lies further north, between the conveyors, receiving run-off from the conveyor and road crossing bridges. The Road Dam location is shown in Figure 5 below.

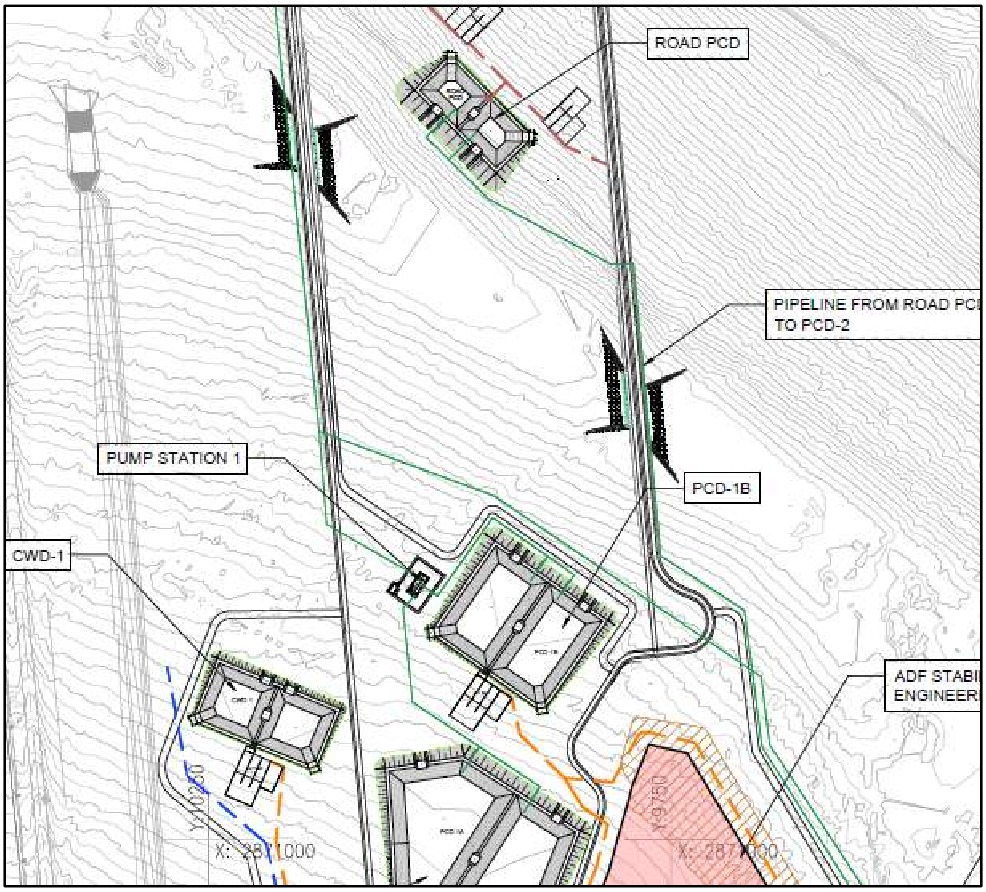


Figure 9: Road Dam Location

PCD 1 is located between the top and bottom extendible conveyors and CWD 1 is located west of the bottom extendible conveyor. Figure 4-16 shows the layout of PCD 1 and CWD 1. Both dams respectively service the contaminated and clean water runoff that is generated on the western side slope of the proposed ADF.

PCD 2, 3, 4, 5 and 6 are located east of the top extendible conveyor. Figure 4-17 below shows the layout of PCD 2 and CWD 2 which form part of Phase 1 of the design. PCDs 3 to 6 are part of Phase 2 of the design.

The catchment for PCD 2 is the northern face of the ADF and the dam also receives overflow from PCDs 3 to 6. The catchment for PCD 6 is the upper eastern face of the ADF. The dam, once full, will overflow into the downstream dam, PCD 5, which will in turn overflow into the downstream dam, PCD 4, which will in turn overflow into the downstream dam, PCD 3. PCD 3 will then overflow into PCD 2. A water transfer provision has been added to PCD 2 to enable transferring of water from PCD 2 to PCD 6. The provision ensures that PCD 2 does not spill whilst there is still capacity in dams PCD 3 to PCD 6. PCD 2 has an abstraction pipeline which is a suction pipeline to Pump Station No. 2. PCDs 3 to 6 have separate abstraction pipelines which feed the inlet to PCD 2. These abstraction pipelines are intended to be used only for maintenance purposes of the PCDs or to supply water to PCD 2 if there is no water in PCD 2 for dust suppression.

CWD 2 similarly services the clean water runoff that is generated on the northern and upper eastern side slope of the proposed ADF.

The general slope of the ADF site is to the north. It should be noted that Phase 1 of the ash disposal and related infrastructure is located down-slope of all later phases, and therefore Phase 1 design, including the stream diversion pipelines, must allow for the required final-stage capacity of channels, pipelines and CWDs / PCD’s. Phase 1 of the ADF operations is expected to last seven years and includes four PCDs and two CWDs.

9.7 Stormwater Design Methods and Software

A combination of Autodesk Civil3D models, Storm and Sanitary Analysis (SSA), and Excel based spreasheets for Rational Formula and Manning’s Equation have been used, with cross-checking between the various manual and automated processes. Hydrocube, a catchment analysis programme has been used as well for pipe and channel size confirmation as well as varifying the performance of chosen pipes.

Most hydrologic models are based on the EPA's Storm Water Management Model (SWMM) model. SWMM is a proven model design and analyzing urban and also rural drainage systems. SSA has a built-in capability to run a SWMM model, as well as TR-55, HEC-1, Rational method, and other hydrologic models. SSA is uniquely suited for land development design due to its integration with Civil 3D.

The Rational Method as described in the document Drainage Manual, Sixth Edition (2013) as published by SANRAL (South African National Roads Agency Limited) was used to determine the magnitude of the 1 in 50 year and 1 in 200 year flows.

The Rational Method is still probably the most commonly used method of estimating the peak runoff value of stormwater runooff generated from urban and rural areas where areas do not exceed 25km^2.

The formula used in this method is

$$\mathbf{Q= f_t \times C \times I \times A/360\ m^3/s}$$

Q= the maximum /peak rate of runoff in m^3/s

F_t= and adjustment factor for the recurrence interval storm considered

C= runoff co-efficient per applicable tables

I= the rainfall Intensity(mm/hr)

A= area of catchment in hectares

The runoff co-efficient is a factor ranging from 0 to 1 which compensates for variations in rainfall over the catchment, infiltration and overland flow velocity during a storm. C can be determined from the Table Method used by the DWAF for consistency of approach.

The initial input was the 0.5 m interval contours that were obtained from the lidar survey. The software Autodesk Civil3D was used to extract cross sections of the watercourse at intervals. The cross sections were imported into the software “SSA” automatically and calculations were processed. The output generally provides alternative methodolgy’s results, for comparison. Longitudonal sections are produced from the Civil3D model for channels and pipelines. ‘Before’ and ‘after’ models can be compared, and earthworks cut and fill volumes calculated.

For sizing of pipelines and channels, manning’s formula below was used.

$$Q = VA = \left(\frac{1.49}{n}\right) AR^{\frac{2}{3}} \sqrt{S} \quad [U.S.]$$

$$Q = VA = \left(\frac{1.00}{n}\right) AR^{\frac{2}{3}} \sqrt{S} \quad [SI]$$

Q = Flow (m^3/s)

V = Velocity (m/s)

A = Cross sectionalm Area (m^2)

R = Hydraulic Radius

S = Channel or Pipe Slope (m/m)

n = Manning's Roughness Coefficient

9.7.1 PCD / CWD Dam sizing

Calculations were carried out using the computer programme *Storm and Sanitary Analysis* (SSA), which is an Autodesk product and compatible with Civil3D design and drafting software. Catchments reporting to each channel were analysed by the Rational method. Results are included in Appendix A. A 1:50 year storm of 24 hour duration was considered in the sizing of the dam. Operational requirements also influenced the dam sizing.

A Water Balance will be reported on separately, including stormwater input and water output from the dams for dust suppression. Water levels will be managed between dams via the pump-stations and piped connections.

10. REFERENCES

10.1 Reports

1. Minimum Requirements for Waste Disposal by Landfill. Second Edition. Department of Water Affairs and Forestry, Pretoria. (Waste Management Series.1998).

2. Environmental Management Programme for the Kusile Power Station 60 Year Ash Disposal Facility, Report No. 12712-46-Rep-001-EMPr-Rev1, by Zitholele Consulting (Pty) Ltd, 20 October 2014.

3. Kusile and Kendal Power Stations Ash Disposal Facilities, Waste Classification Report, Report No. JW030/13/D121 Rev3, by Jones & Wagener January 2014.

4. Design Review - Technical Report, Report No. 22020-T01-CE-RPT-001 Rev B, by EPCM, 12 April 2022.

5. Application for Water Use Licence for the Kusile 60 Year Ash Disposal Facility at Kusile Power Station, Mpumalanga, Report no. 12712-46-Rep-001 Kusile IWULA-Rev0, by Zitholele Consulting (Pty) Ltd, 6 November 2015.

6. Request for Approval of Design Drawings for Eskom SOC Kusile Power Station 60 Year Ash Disposal Facility in Terms of Section 50 of the National Environmental Management Act: Waste Act, 2008 – Mpumalanga Province, Ref No. 12/9/11/L193/6 by DFFE 9 June 2021.

7. Application for Integrated Environmental Authorisation in Terms of the National Environmental Management Act, 1998 : GN R 543 and National Environmental management: waste act, 2008 GN 921: Construction of a 60 Year Ash Disposal Facility at Kusile Power Station, Mpumalanga Province, Ref. No. 12/12/20/2412 by DEA 17 July 2015.

8. Application for Water Use Licence for the Kusile 60 Year Ash Disposal Facility at the Kusile Power Station, Mpumalanga, Report No. 12712-46-Rep-001-Kusile IWULA- Rev0, by Zitholele Consulting (Pty) Ltd, 6 November 2015.

10.2 Drawings

9. 366-511844-B 60yr General Arrangement
10. 366-511846-B Attenuation Dams
11. 366-511849-B Phase 1 PCDs

10.3 Websites

12. None.

Appendix A

Calculations

Project Description

File Name	ADF CHANNELS.SPF
Description	
	About: File generated by iDAS (www.devotechgroup.com) CHANNEL SIZING LTD\Communication site - Documents\Johannesburg\Projects\ES projects\816-2023 About: File generated by iDAS (www.devotechgroup.com) CHANNEL SIZING

Project Options

Flow Units	CMS
Elevation Type	Elevation
Hydrology Method	EPA SWMM
EPA SWMM Infiltration Method	Horton
Link Routing Method	Kinematic Wave
Enable Overflow Ponding at Nodes	NO
Skip Steady State Analysis Time Periods	NO

Analysis Options

Start Analysis On	00:00:00	00:00:00
End Analysis On	00:00:00	00:00:00
Start Reporting On	00:00:00	00:00:00
Antecedent Dry Days	0	days
Runoff (Dry Weather) Time Step	0 00:01:00	days hh:mm:ss
Runoff (Wet Weather) Time Step	0 00:01:00	days hh:mm:ss
Reporting Time Step	0 00:01:00	days hh:mm:ss
Routing Time Step	10	seconds

Number of Elements

	Qty
Rain Gages	1
Subbasins..	18
Nodes..	111
Junctions	99
Outfalls	12
Flow Diversions	0
Inlets	0
Storage Nodes	0
Links..	98
Channels	98
Pipes	0
Pumps	0
Orifices	0
Weirs	0
Outlets	0
Pollutants	0
Land Uses	0

Rainfall Details

SN	Rain Gage ID	Data Source	Data Source ID	Rainfall Type	Rain Units	State	County	Return Period (years)	Rainfall Depth (mm)	Rainfall Distribution
1	Rain Gage-01	Time Series	TS-01_1IN 50YR	Intensity	mm	None	None	50.00	122.00	South Africa 24-hr, Type 2

Subbasin Summary

SN	Subbasin ID	Area (ha)	Impervious Area (%)	Average Slope (%)	Equivalent Width (m)	Impervious Area Manning's Roughness	Pervious Area Manning's Roughness	Total Rainfall (mm)	Total Infiltration (mm)	Total Runoff (mm)	Total Runoff Volume (ha-mm)	Peak Runoff (cms)	Time of Concentration (days hh:mm:ss)
1	{PHASE 1}.Catchment : 1	40.01	25.00	2.1900	1825.00	0.3000	0.1300	121.89	66.3280	54.60	2184.78	6.54	0 01:12:09
2	{PHASE 1}.Catchment : 10	12.10	25.00	9.7100	801.60	0.3000	0.1300	121.89	63.9820	57.26	692.72	3.37	0 00:36:53
3	{PHASE 1}.Catchment : 11	1.70	25.00	5.3300	163.60	0.3000	0.1300	121.89	63.8920	57.36	97.74	0.49	0 00:35:21
4	{PHASE 1}.Catchment : 12	3.17	25.00	1.6100	289.60	0.3000	0.1300	121.89	64.9500	56.16	178.06	0.66	0 00:52:10
5	{PHASE 1}.Catchment : 13	30.53	25.00	1.8500	1968.80	0.3000	0.1300	121.89	65.5960	55.43	1692.11	5.67	0 01:01:39
6	{PHASE 1}.Catchment : 14	9.10	30.00	1.7600	810.40	0.3000	0.1300	121.89	60.4540	60.44	549.93	1.96	0 00:49:28
7	{PHASE 1}.Catchment : 15	30.49	30.00	2.1600	1055.70	0.3000	0.1300	121.89	62.5530	57.94	1766.24	4.41	0 01:22:01
8	{PHASE 1}.Catchment : 18	10.49	25.00	0.8600	1093.00	0.3000	0.1300	121.89	65.3500	55.70	584.39	2.03	0 00:58:11
9	{PHASE 1}.Catchment : 19	5.54	25.00	3.3800	909.00	0.3000	0.1300	121.89	63.5540	57.75	319.84	1.85	0 00:29:22
10	{PHASE 1}.Catchment : 2	4.94	25.00	4.9800	710.85	0.3000	0.1300	121.89	63.5060	57.81	285.82	1.69	0 00:28:19
11	{PHASE 1}.Catchment : 3	2.34	25.00	17.0400	254.10	0.3000	0.1300	121.89	63.2520	58.11	135.70	0.90	0 00:23:08
12	{PHASE 1}.Catchment : 4	3.55	25.00	13.2800	325.30	0.3000	0.1300	121.89	63.4700	57.85	205.15	1.23	0 00:27:37
13	{PHASE 1}.Catchment : 5	2.75	25.00	8.8800	407.10	0.3000	0.1300	121.89	63.2620	58.10	159.49	1.06	0 00:23:22
14	{PHASE 1}.Catchment : 6	26.53	25.00	4.0100	1535.70	0.3000	0.1300	121.89	64.9500	56.16	1489.90	5.52	0 00:52:10
15	{PHASE 1}.Catchment : 7	27.52	25.00	12.6200	757.40	0.3000	0.1300	121.89	65.3290	55.73	1533.38	5.36	0 00:57:46
16	{PHASE 1}.Catchment : 8	8.47	25.00	4.5700	911.00	0.3000	0.1300	121.89	63.8480	57.41	486.12	2.49	0 00:34:34
17	{PHASE 1}.Catchment : 9	4.23	25.00	11.3400	378.10	0.3000	0.1300	121.89	63.5640	57.74	244.48	1.41	0 00:29:26
18	Catchment : 16	111.95	25.00	2.0000	1741.00	0.3000	0.1300	121.89	71.0020	49.29	5517.84	9.22	0 02:21:25

Node Summary

SN	Element ID	Element Type	Invert Elevation (m)	Ground/Rim (Max) Elevation (m)	Initial Water Elevation (m)	Surcharge Elevation (m)	Ponded Area (m^2)	Peak Inflow (cms)	Max HGL Elevation Attained (m)	Max Surcharge Depth Attained (m)	Min Freeboard Attained (m)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ha-mm)	Total Time Flooded (min)
1	INLET1	Junction	1489.97	1491.77	1489.97	1491.77	0.00	18.21	1490.50	0.00	1.27	0 00:00	0.00	0.00
2	INLET10	Junction	1493.15	1495.07	1493.15	1495.07	0.00	1.06	1493.31	0.00	1.76	0 00:00	0.00	0.00
3	INLET11	Junction	1494.47	1495.87	1494.47	1495.87	0.00	7.81	1494.95	0.00	0.92	0 00:00	0.00	0.00
4	INLET12	Junction	1494.97	1496.37	1494.97	1496.37	0.00	5.67	1495.44	0.00	0.92	0 00:00	0.00	0.00
5	INLET13	Junction	1461.94	1463.84	1461.94	1463.84	0.00	5.36	1462.26	0.00	1.58	0 00:00	0.00	0.00
6	INLET14	Junction	1460.22	1461.12	1460.22	1461.12	0.00	1.41	1460.41	0.00	0.71	0 00:00	0.00	0.00
7	INLET15	Junction	1460.85	1461.85	1460.85	1461.85	0.00	3.37	1461.34	0.00	0.51	0 00:00	0.00	0.00
8	INLET16	Junction	1450.97	1452.50	1450.97	1452.50	0.00	8.72	1451.73	0.00	0.77	0 00:00	0.00	0.00
9	INLET17	Junction	1457.45	1458.64	1457.45	1458.64	0.00	1.88	1457.84	0.00	0.80	0 00:00	0.00	0.00
10	INLET18	Junction	1459.92	1460.92	1459.92	1460.92	0.00	3.98	1460.30	0.00	0.62	0 00:00	0.00	0.00
11	INLET19	Junction	1461.98	1463.48	1461.98	1463.48	0.00	18.35	1462.59	0.00	0.89	0 00:00	0.00	0.00
12	INLET2	Junction	1490.10	1492.00	1490.10	1492.00	0.00	6.54	1490.58	0.00	1.42	0 00:00	0.00	0.00
13	INLET3	Junction	1486.48	1487.48	1486.48	1487.48	0.00	1.69	1486.69	0.00	0.78	0 00:00	0.00	0.00
14	INLET4	Junction	1485.79	1487.20	1485.79	1487.20	0.00	4.41	1486.15	0.00	1.05	0 00:00	0.00	0.00
15	INLET5	Junction	1471.23	1472.13	1471.23	1472.13	0.00	0.90	1471.43	0.00	0.70	0 00:00	0.00	0.00
16	INLET6	Junction	1471.14	1473.62	1471.14	1473.62	0.00	0.00	1471.14	0.00	2.48	0 00:00	0.00	0.00
17	INLET7	Junction	1471.14	1472.54	1471.14	1472.54	0.00	0.00	1471.14	0.00	1.40	0 00:00	0.00	0.00
18	INLET8	Junction	1471.23	1472.13	1471.23	1472.13	0.00	1.23	1471.49	0.00	0.64	0 00:00	0.00	0.00
19	INLET81	Junction	1459.67	1462.27	1459.67	1462.27	0.00	24.66	1460.96	0.00	1.31	0 00:00	0.00	0.00
20	INLET9	Junction	1494.91	1495.81	1494.91	1495.81	0.00	1.96	1495.13	0.00	0.68	0 00:00	0.00	0.00
21	MH103	Junction	1489.37	1490.37	1489.37	1490.37	0.00	1.96	1489.59	0.00	0.78	0 00:00	0.00	0.00
22	MH104	Junction	1485.66	1486.66	1485.66	1486.66	0.00	1.96	1485.84	0.00	0.82	0 00:00	0.00	0.00
23	MH107	Junction	1488.49	1489.99	1488.49	1489.99	0.00	1.03	1488.65	0.00	1.34	0 00:00	0.00	0.00
24	MH108	Junction	1484.97	1486.47	1484.97	1486.47	0.00	1.03	1485.10	0.00	1.38	0 00:00	0.00	0.00
25	MH110	Junction	1488.00	1489.90	1488.00	1489.90	0.00	18.19	1488.54	0.00	1.36	0 00:00	0.00	0.00
26	MH111	Junction	1486.15	1488.05	1486.15	1488.05	0.00	18.17	1486.69	0.00	1.36	0 00:00	0.00	0.00
27	MH112	Junction	1484.06	1485.96	1484.06	1485.96	0.00	18.17	1484.64	0.00	1.32	0 00:00	0.00	0.00
28	MH113	Junction	1482.60	1484.50	1482.60	1484.50	0.00	18.16	1483.17	0.00	1.32	0 00:00	0.00	0.00
29	MH114	Junction	1480.92	1482.82	1480.92	1482.82	0.00	18.15	1481.47	0.00	1.35	0 00:00	0.00	0.00
30	MH115	Junction	1478.95	1480.85	1478.95	1480.85	0.00	18.15	1479.48	0.00	1.37	0 00:00	0.00	0.00
31	MH116	Junction	1476.94	1478.84	1476.94	1478.84	0.00	18.15	1477.47	0.00	1.37	0 00:00	0.00	0.00
32	MH117	Junction	1488.09	1489.99	1488.09	1489.99	0.00	6.54	1488.57	0.00	1.42	0 00:00	0.00	0.00
33	MH118	Junction	1486.29	1488.17	1486.29	1488.17	0.00	6.54	1486.77	0.00	1.41	0 00:00	0.00	0.00
34	MH119	Junction	1484.30	1486.26	1484.30	1486.26	0.00	6.54	1484.79	0.00	1.46	0 00:00	0.00	0.00
35	MH12	Junction	1457.86	1459.76	1457.86	1459.76	0.00	27.83	1458.51	0.00	1.24	0 00:00	0.00	0.00
36	MH120	Junction	1482.70	1484.60	1482.70	1484.60	0.00	6.54	1483.19	0.00	1.40	0 00:00	0.00	0.00
37	MH121	Junction	1480.96	1482.86	1480.96	1482.86	0.00	6.54	1481.45	0.00	1.41	0 00:00	0.00	0.00
38	MH122	Junction	1478.99	1480.89	1478.99	1480.89	0.00	6.54	1479.46	0.00	1.43	0 00:00	0.00	0.00
39	MH123	Junction	1477.09	1478.98	1477.09	1478.98	0.00	6.54	1477.56	0.00	1.42	0 00:00	0.00	0.00
40	MH125	Junction	1477.60	1478.60	1477.60	1478.60	0.00	1.96	1477.76	0.00	0.84	0 00:00	0.00	0.00
41	MH126	Junction	1477.05	1479.60	1477.05	1479.60	0.00	1.03	1477.15	0.00	2.45	0 00:00	0.00	0.00
42	MH127	Junction	1465.03	1466.53	1465.03	1466.53	0.00	7.80	1465.57	0.00	0.96	0 00:00	0.00	0.00
43	MH128	Junction	1466.46	1467.96	1466.46	1467.96	0.00	7.43	1466.89	0.00	1.07	0 00:00	0.00	0.00
44	MH130	Junction	1478.49	1480.34	1478.49	1480.34	0.00	4.39	1478.83	0.00	1.51	0 00:00	0.00	0.00
45	MH132	Junction	1479.45	1480.45	1479.45	1480.45	0.00	1.65	1479.65	0.00	0.80	0 00:00	0.00	0.00
46	MH133	Junction	1473.99	1474.99	1473.99	1474.99	0.00	1.65	1474.17	0.00	0.82	0 00:00	0.00	0.00
47	MH134	Junction	1483.50	1484.50	1483.50	1484.50	0.00	1.66	1483.71	0.00	0.78	0 00:00	0.00	0.00
48	MH135	Junction	1473.29	1475.19	1473.29	1475.19	0.00	4.38	1473.61	0.00	1.58	0 00:00	0.00	0.00
49	MH136	Junction	1482.49	1484.39	1482.49	1484.39	0.00	4.39	1482.85	0.00	1.54	0 00:00	0.00	0.00
50	MH138	Junction	1469.99	1471.89	1469.99	1471.89	0.00	0.00	1469.99	0.00	1.90	0 00:00	0.00	0.00
51	MH139	Junction	1470.29	1471.32	1470.29	1471.32	0.00	0.89	1470.49	0.00	0.84	0 00:00	0.00	0.00
52	MH140	Junction	1470.24	1472.14	1470.24	1472.14	0.00	0.00	1470.24	0.00	1.90	0 00:00	0.00	0.00
53	MH141	Junction	1470.40	1471.50	1470.40	1471.50	0.00	1.20	1470.66	0.00	0.84	0 00:00	0.00	0.00
54	MH144	Junction	1458.30	1460.20	1458.30	1460.20	0.00	27.83	1459.18	0.00	1.02	0 00:00	0.00	0.00
55	MH145	Junction	1461.98	1463.48	1461.98	1463.48	0.00	18.12	1462.56	0.00	0.92	0 00:00	0.00	0.00
56	MH15	Junction	1476.23	1478.12	1476.23	1478.12	0.00	6.54	1476.73	0.00	1.39	0 00:00	0.00	0.00
57	MH16	Junction	1475.51	1477.41	1475.51	1477.41	0.00	6.54	1476.01	0.00	1.40	0 00:00	0.00	0.00
58	MH17	Junction	1469.19	1471.09	1469.19	1471.09	0.00	6.54	1469.61	0.00	1.48	0 00:00	0.00	0.00
59	MH18	Junction	1464.35	1466.25	1464.35	1466.25	0.00	6.54	1464.78	0.00	1.47	0 00:00	0.00	0.00
60	MH19	Junction	1461.01	1462.89	1461.01	1462.89	0.00	6.54	1461.47	0.00	1.42	0 00:00	0.00	0.00
61	MH2	Junction	1476.00	1477.90	1476.00	1477.90	0.00	18.14	1476.56	0.00	1.33	0 00:00	0.00	0.00
62	MH3	Junction	1475.29	1477.13	1475.29	1477.13	0.00	18.14	1475.86	0.00	1.27	0 00:00	0.00	0.00
63	MH34	Junction	1493.95	1496.01	1493.95	1496.01	0.00	7.81	1494.43	0.00	1.58	0 00:00	0.00	0.00
64	MH35	Junction	1493.59	1495.74	1493.59	1495.74	0.00	7.81	1494.06	0.00	1.67	0 00:00	0.00	0.00
65	MH36	Junction	1493.18	1495.10	1493.18	1495.10	0.00	7.81	1493.66	0.00	1.45	0 00:00	0.00	0.00
66	MH37	Junction	1492.86	1494.36	1492.86	1494.36	0.00	7.81	1493.33	0.00	1.02	0 00:00	0.00	0.00
67	MH38	Junction	1484.35	1485.85	1484.35	1485.85	0.00	7.81	1484.76	0.00	1.09	0 00:00	0.00	0.00
68	MH39	Junction	1472.82	1474.32	1472.82	1474.32	0.00	7.81	1473.28	0.00	1.04	0 00:00	0.00	0.00
69	MH4	Junction	1469.39	1471.29	1469.39	1471.29	0.00	18.13	1469.87	0.00	1.42	0 00:00	0.00	0.00
70	MH40	Junction	1469.73	1471.23	1469.73	1471.23	0.00	7.80	1470.31	0.00	0.92	0 00:00	0.00	0.00
71	MH41	Junction	1467.78	1469.28	1467.78	1469.28	0.00	7.80	1468.36	0.00	0.92	0 00:00	0.00	0.00
72	MH42	Junction	1462.08	1463.58	1462.08	1463.58	0.00	7.79	1462.77	0.00	0.81	0 00:00	0.00	0.00
73	MH43	Junction	1461.58	1463.59	1461.58	1463.59	0.00	7.79	1462.27	0.00	1.32	0 00:00	0.00	0.00

Node Summary

SN	Element ID	Element Type	Invert Elevation (m)	Ground/Rim (Max) Elevation (m)	Initial Water Elevation (m)	Surcharge Elevation (m)	Ponded Area (m²)	Peak Inflow (cms)	Max HGL Elevation Attained (m)	Max Surcharge Depth Attained (m)	Min Freeboard Attained (m)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ha-mm)	Total Time Flooded (min)
74	MH44	Junction	1461.39	1463.74	1461.39	1463.74	0.00	7.79	1462.09	0.00	1.66	0 00:00	0.00	0.00
75	MH45	Junction	1459.86	1461.36	1459.86	1461.36	0.00	7.79	1461.36	0.00	0.00	0 00:00	0.00	0.00
76	MH5	Junction	1463.08	1466.28	1463.08	1466.28	0.00	18.13	1464.80	0.00	1.48	0 00:00	0.00	0.00
77	MH51	Junction	1494.24	1495.74	1494.24	1495.74	0.00	5.66	1494.72	0.00	1.02	0 00:00	0.00	0.00
78	MH52	Junction	1492.96	1494.46	1492.96	1494.46	0.00	5.66	1493.35	0.00	1.10	0 00:00	0.00	0.00
79	MH53	Junction	1484.65	1486.15	1484.65	1486.15	0.00	5.65	1485.04	0.00	1.12	0 00:00	0.00	0.00
80	MH54	Junction	1479.67	1481.17	1479.67	1481.17	0.00	5.65	1480.08	0.00	1.09	0 00:00	0.00	0.00
81	MH55	Junction	1474.35	1475.85	1474.35	1475.85	0.00	5.64	1474.81	0.00	1.04	0 00:00	0.00	0.00
82	MH56	Junction	1471.24	1472.74	1471.24	1472.74	0.00	5.63	1471.83	0.00	0.91	0 00:00	0.00	0.00
83	MH57	Junction	1469.06	1470.56	1469.06	1470.56	0.00	5.61	1469.65	0.00	0.91	0 00:00	0.00	0.00
84	MH58	Junction	1461.92	1470.30	1461.92	1470.30	0.00	7.43	1462.47	0.00	7.83	0 00:00	0.00	0.00
85	MH61	Junction	1459.15	1461.00	1459.15	1461.00	0.00	5.35	1459.47	0.00	1.53	0 00:00	0.00	0.00
86	MH62	Junction	1455.15	1457.06	1455.15	1457.06	0.00	5.35	1455.77	0.00	1.29	0 00:00	0.00	0.00
87	MH63	Junction	1455.10	1456.90	1455.10	1456.90	0.00	5.35	1455.72	0.00	1.18	0 00:00	0.00	0.00
88	MH64	Junction	1454.77	1456.35	1454.77	1456.35	0.00	7.43	1455.48	0.00	0.87	0 00:00	0.00	0.00
89	MH66	Junction	1450.68	1452.23	1450.68	1452.23	0.00	8.71	1451.45	0.00	0.78	0 00:00	0.00	0.00
90	MH68	Junction	1450.29	1452.15	1450.29	1452.15	0.00	10.27	1451.41	0.00	0.73	0 00:00	0.00	0.00
91	MH72	Junction	1459.50	1460.50	1459.50	1460.50	0.00	1.41	1459.70	0.00	0.80	0 00:00	0.00	0.00
92	MH75	Junction	1453.47	1454.47	1453.47	1454.47	0.00	1.84	1453.71	0.00	0.76	0 00:00	0.00	0.00
93	MH76	Junction	1452.53	1453.53	1452.53	1453.53	0.00	1.84	1452.77	0.00	0.76	0 00:00	0.00	0.00
94	MH79	Junction	1460.68	1461.85	1460.68	1461.85	0.00	3.36	1461.16	0.00	0.68	0 00:00	0.00	0.00
95	MH8	Junction	1459.54	1461.44	1459.54	1461.44	0.00	24.66	1460.15	0.00	1.29	0 00:00	0.00	0.00
96	MH80	Junction	1460.34	1461.34	1460.34	1461.34	0.00	3.34	1460.80	0.00	0.53	0 00:00	0.00	0.00
97	MH84	Junction	1460.78	1462.67	1460.78	1462.67	0.00	18.35	1461.39	0.00	1.28	0 00:00	0.00	0.00
98	MH85	Junction	1456.52	1458.42	1456.52	1458.42	0.00	18.35	1457.22	0.00	1.21	0 00:00	0.00	0.00
99	MH86	Junction	1455.79	1457.35	1455.79	1457.35	0.00	18.35	1456.48	0.00	0.87	0 00:00	0.00	0.00
100	MH101	Outfall	1466.65					0.00	1466.65					
101	MH105	Outfall	1465.60					1.95	1465.75					
102	MH109	Outfall	1469.12					1.03	1469.23					
103	MH13	Outfall	1457.41					27.83	1458.06					
104	MH22	Outfall	1466.17					1.64	1466.33					
105	MH24	Outfall	1465.93					4.38	1466.21					
106	MH26	Outfall	1466.91					0.88	1467.04					
107	MH28	Outfall	1466.40					0.00	1466.40					
108	MH59	Outfall	1456.97					7.43	1457.52					
109	MH70	Outfall	1450.29					10.27	1451.13					
110	MH87	Outfall	1455.41					18.35	1456.02					
111	MH99	Outfall	1466.49					1.19	1466.65					

Link Summary

SN	Element ID	Element Type	From (Inlet) Node	To (Outlet) Node	Length (m)	Inlet Invert Elevation (m)	Outlet Invert Elevation (m)	Average Slope (%)	Diameter or Height (mm)	Manning's Roughness	Peak Flow (cms)	Design Flow Capacity (cms)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (m/sec)	Peak Flow Depth (m)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Reported Condition
1	C1	Channel	INLET1	MH110	90.02	1489.97	1488.00	2.1900	1000.000	0.0150	18.19	56.45	0.32	5.67	0.53	0.53	0.00	
2	C10	Channel	MH12	MH13	18.45	1457.86	1457.41	2.4200	1000.000	0.0150	27.83	59.39	0.47	6.73	0.66	0.66	0.00	
3	C11	Channel	INLET2	MH117	104.03	1490.10	1488.09	1.9300	650.000	0.0150	6.54	11.44	0.57	4.55	0.48	0.74	0.00	
4	C12	Channel	MH117	MH118	90.08	1488.09	1486.29	2.0000	650.000	0.0150	6.54	11.65	0.56	4.61	0.48	0.74	0.00	
5	C13	Channel	MH118	MH119	89.96	1486.29	1484.30	2.2100	650.000	0.0150	6.54	12.24	0.53	4.78	0.47	0.72	0.00	
6	C14	Channel	MH119	MH120	90.01	1484.30	1482.70	1.7800	650.000	0.0150	6.54	10.99	0.60	4.42	0.49	0.76	0.00	
7	C15	Channel	MH120	MH121	90.02	1482.70	1480.96	1.9300	650.000	0.0150	6.54	11.44	0.57	4.55	0.48	0.74	0.00	
8	C16	Channel	MH121	MH122	90.03	1480.96	1478.99	2.1900	650.000	0.0150	6.54	12.20	0.54	4.77	0.47	0.72	0.00	
9	C17	Channel	MH122	MH123	90.02	1478.99	1477.09	2.1100	650.000	0.0150	6.54	11.97	0.55	4.70	0.47	0.73	0.00	
10	C18	Channel	MH123	MH15	42.59	1477.09	1476.23	2.0200	650.000	0.0150	6.54	11.71	0.56	4.63	0.48	0.74	0.00	
11	C19	Channel	MH15	MH16	42.73	1476.23	1475.51	1.6800	650.000	0.0150	6.54	10.66	0.61	4.33	0.50	0.77	0.00	
12	C2	Channel	MH110	MH111	90.03	1488.00	1486.15	2.0500	1000.000	0.0150	18.17	54.59	0.33	5.54	0.54	0.54	0.00	
13	C20	Channel	MH16	MH17	199.09	1475.51	1469.19	3.1800	650.000	0.0150	6.54	14.68	0.45	5.43	0.42	0.65	0.00	
14	C21	Channel	MH17	MH18	103.23	1469.19	1464.35	4.6800	650.000	0.0150	6.54	17.83	0.37	6.23	0.38	0.59	0.00	
15	C22	Channel	MH18	MH19	109.86	1464.35	1461.01	3.0400	650.000	0.0150	6.54	14.36	0.46	5.35	0.43	0.66	0.00	
16	C23	Channel	MH19	INLET81	25.56	1461.01	1460.40	2.4200	650.000	0.0150	6.54	12.82	0.51	4.94	0.46	0.70	0.00	
17	C24	Channel	INLET3	MH134	133.18	1486.48	1483.50	2.2400	500.000	0.0150	1.66	7.49	0.22	3.17	0.22	0.43	0.00	
18	C25	Channel	MH132	MH133	133.26	1479.45	1473.99	4.1000	500.000	0.0150	1.65	10.14	0.16	3.86	0.18	0.36	0.00	
19	C26	Channel	MH133	MH22	133.37	1473.99	1466.17	5.8600	500.000	0.0150	1.64	12.12	0.14	4.34	0.16	0.33	0.00	
20	C27	Channel	MH134	MH132	133.21	1483.50	1479.45	3.0400	500.000	0.0150	1.65	8.73	0.19	3.51	0.20	0.40	0.00	
21	C28	Channel	MH2	MH3	40.97	1476.00	1475.29	1.7200	1000.000	0.0150	18.14	50.05	0.36	5.22	0.57	0.57	0.00	
22	C29	Channel	INLET4	MH136	126.28	1485.79	1482.49	2.6100	500.000	0.0150	4.39	8.09	0.54	4.51	0.36	0.72	0.00	
23	C3	Channel	MH111	MH112	90.01	1486.15	1484.06	2.3300	1000.000	0.0150	18.17	58.18	0.31	5.78	0.52	0.52	0.00	
24	C30	Channel	MH130	MH135	126.34	1478.49	1473.29	4.1100	500.000	0.0150	4.38	10.15	0.43	5.28	0.32	0.63	0.00	
25	C31	Channel	MH135	MH24	126.45	1473.29	1465.93	5.8300	500.000	0.0150	4.38	12.08	0.36	5.95	0.29	0.57	0.00	
26	C32	Channel	MH136	MH130	126.30	1482.49	1478.49	3.1700	500.000	0.0150	4.39	8.91	0.49	4.82	0.34	0.68	0.00	
27	C33	Channel	INLET5	MH139	101.57	1471.23	1470.29	0.9200	500.000	0.0150	0.89	4.81	0.18	1.93	0.19	0.39	0.00	
28	C34	Channel	MH139	MH26	103.23	1470.29	1466.91	3.2800	500.000	0.0150	0.88	9.07	0.10	2.91	0.13	0.27	0.00	
29	C35	Channel	INLET6	MH138	101.33	1471.14	1469.99	1.1300	500.000	0.0150	0.00	5.33	0.00	0.00	0.00	0.00	0.00	
30	C36	Channel	MH138	MH28	103.50	1469.99	1466.40	3.4700	500.000	0.0150	0.00	9.33	0.00	0.00	0.00	0.00	0.00	
31	C37	Channel	INLET11	MH34	25.84	1494.47	1493.95	2.0300	1000.000	0.0150	7.81	31.91	0.24	4.76	0.48	0.48	0.00	
32	C38	Channel	MH34	MH35	17.79	1493.95	1493.59	2.0300	1000.000	0.0150	7.81	31.96	0.24	4.76	0.48	0.48	0.00	
33	C39	Channel	MH35	MH36	19.99	1493.59	1493.18	2.0300	1000.000	0.0150	7.81	31.90	0.24	4.76	0.48	0.48	0.00	
34	C4	Channel	MH112	MH113	90.01	1484.06	1482.60	1.6300	1000.000	0.0150	18.16	48.69	0.37	5.13	0.58	0.58	0.00	
35	C40	Channel	MH3	MH4	196.74	1475.29	1469.39	3.0000	1000.000	0.0150	18.13	66.10	0.27	6.30	0.48	0.48	0.00	
36	C41	Channel	MH36	MH37	16.11	1493.18	1492.86	2.0300	1000.000	0.0150	7.81	31.93	0.24	4.76	0.48	0.48	0.00	
37	C42	Channel	MH37	MH38	245.93	1492.86	1484.35	3.4600	1000.000	0.0150	7.81	41.68	0.19	5.75	0.41	0.41	0.00	
38	C43C	Channel	MH38	MH39	201.41	1484.35	1472.82	5.7200	1000.000	0.0150	7.81	53.62	0.15	6.83	0.36	0.36	0.00	
39	C44	Channel	MH39	MH40	132.85	1472.82	1469.73	2.3200	1000.000	0.0150	7.80	34.16	0.23	5.00	0.46	0.46	0.00	
40	C45	Channel	MH40	MH41	194.47	1469.73	1467.78	1.0000	1000.000	0.0150	7.80	22.45	0.35	3.72	0.58	0.58	0.00	
41	C46	Channel	MH41	MH127	210.89	1467.78	1465.03	1.3000	1000.000	0.0150	7.80	25.59	0.30	4.08	0.54	0.54	0.00	
42	C47	Channel	MH127	MH42	226.51	1465.03	1462.08	1.3000	1000.000	0.0150	7.79	25.59	0.30	4.08	0.54	0.54	0.00	
43	C48	Channel	MH42	MH43	100.20	1462.08	1461.58	0.5000	1000.000	0.0150	7.79	15.85	0.49	2.90	0.69	0.69	0.00	
44	C49	Channel	MH43	MH44	36.85	1461.58	1461.39	0.5000	1000.000	0.0150	7.79	15.83	0.49	2.89	0.69	0.69	0.00	
45	C5	Channel	MH113	MH114	90.02	1482.60	1480.92	1.8600	1000.000	0.0150	18.15	52.04	0.35	5.36	0.55	0.55	0.00	
46	C50	Channel	MH44	MH45	57.25	1461.39	1459.86	2.6800	1000.000	0.0150	7.79	36.71	0.21	5.24	0.44	0.44	0.00	
47	C51	Channel	MH4	MH5	103.19	1469.39	1464.38	4.8500	1000.000	0.0150	18.13	84.06	0.22	7.39	0.42	0.42	0.00	

Link Summary

SN	Element ID	Element Type	From (Inlet) Node	To (Outlet) Node	Length (m)	Inlet Invert Elevation (m)	Outlet Invert Elevation (m)	Average Slope (%)	Diameter or Height (mm)	Manning's Roughness	Peak Flow (cms)	Design Flow Capacity (cms)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (m/sec)	Peak Flow Depth (m)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Reported Condition
48	C52	Channel	INLET12	MH51	48.68	1494.97	1494.24	1.4900	1000.000	0.0150	5.66	23.65	0.24	3.99	0.48	0.48	0.00	
49	C53	Channel	MH51	MH52	41.90	1494.24	1492.96	3.0500	1000.000	0.0150	5.66	33.81	0.17	5.14	0.39	0.40	0.00	
50	C54	Channel	MH52	MH53	246.49	1492.96	1484.65	3.3700	1000.000	0.0150	5.65	35.53	0.16	5.34	0.38	0.38	0.00	
51	C55	Channel	MH53	MH54	98.30	1484.65	1479.67	5.0700	1000.000	0.0150	5.65	43.58	0.13	6.13	0.34	0.34	0.00	
52	C56	Channel	MH54	MH55	200.67	1479.67	1474.35	2.6500	1000.000	0.0150	5.64	31.50	0.18	4.89	0.41	0.41	0.00	
53	C57	Channel	MH55	MH56	178.16	1474.35	1471.24	1.7500	1000.000	0.0150	5.63	25.57	0.22	4.22	0.46	0.46	0.00	
54	C58	Channel	MH56	MH57	316.16	1471.24	1469.06	0.6900	1000.000	0.0150	5.61	16.05	0.35	3.04	0.59	0.59	0.00	
55	C59	Channel	MH57	MH128	106.98	1469.06	1466.46	2.4400	1000.000	0.0150	5.61	30.20	0.19	4.73	0.42	0.42	0.00	
56	C6	Channel	MH114	MH115	90.02	1480.92	1478.95	2.1900	1000.000	0.0150	18.15	56.48	0.32	5.67	0.53	0.53	0.00	
57	C60	Channel	MH128	MH58	120.62	1466.46	1461.92	3.7600	1000.000	0.0150	7.43	37.53	0.20	5.99	0.43	0.43	0.00	
58	C61	Channel	MH58	MH59	308.52	1461.92	1456.97	1.6000	1000.000	0.0150	7.43	24.51	0.30	4.44	0.54	0.54	0.00	
59	C62	Channel	INLET13	MH61	47.07	1461.94	1459.15	5.9400	1000.000	0.0150	5.35	47.17	0.11	6.37	0.32	0.32	0.00	
60	C63	Channel	MH61	MH62	71.54	1459.15	1455.15	5.5900	1000.000	0.0150	5.35	45.73	0.12	6.24	0.32	0.32	0.00	
61	C64	Channel	MH62	MH63	10.03	1455.15	1455.10	0.5100	1000.000	0.0150	5.35	13.80	0.39	2.67	0.62	0.62	0.00	
62	C65	Channel	MH63	MH64	57.06	1455.10	1454.77	0.5800	1000.000	0.0150	5.34	14.76	0.36	2.81	0.60	0.60	0.00	
63	C66	Channel	MH64	INLET16	653.71	1454.77	1450.97	0.5800	1000.000	0.0150	7.45	14.76	0.50	3.16	0.71	0.71	0.00	
64	C67	Channel	INLET16	MH66	49.15	1450.97	1450.68	0.5800	1000.000	0.0150	8.72	14.76	0.59	3.21	0.77	0.77	0.00	
65	C68	Channel	MH66	MH68	17.26	1450.68	1450.58	0.5900	1000.000	0.0150	8.71	14.80	0.59	3.21	0.77	0.77	0.00	
66	C69	Channel	MH68	MH70	49.10	1450.58	1450.29	0.5800	1000.000	0.0150	10.27	14.74	0.70	3.35	0.84	0.84	0.00	
67	C7	Channel	MH115	MH116	90.02	1478.95	1476.94	2.2300	1000.000	0.0150	18.15	56.91	0.32	5.70	0.53	0.53	0.00	
68	C70	Channel	INLET14	MH72	30.01	1460.22	1459.50	2.4000	500.000	0.0150	1.41	7.75	0.18	3.06	0.19	0.39	0.00	
69	C71	Channel	MH72	INLET17	93.68	1459.50	1457.64	1.9800	500.000	0.0150	1.39	7.05	0.20	2.87	0.20	0.41	0.00	
70	C72	Channel	INLET17	MH75	217.80	1457.45	1453.47	1.8300	500.000	0.0150	1.85	6.77	0.27	3.07	0.24	0.49	0.00	
71	C73	Channel	MH75	MH76	50.95	1453.47	1452.53	1.8400	500.000	0.0150	1.84	6.79	0.27	3.05	0.24	0.49	0.00	
72	C74	Channel	INLET15	MH79	35.51	1460.85	1460.68	0.5000	500.000	0.0150	3.36	3.53	0.95	2.32	0.48	0.97	0.00	
73	C75	Channel	MH79	MH80	58.46	1460.68	1460.34	0.5800	500.000	0.0150	3.34	3.81	0.88	2.45	0.46	0.93	0.00	
74	C76	Channel	INLET81	MH8	29.51	1460.38	1459.54	2.8200	1000.000	0.0150	24.66	64.06	0.39	6.81	0.59	0.59	0.00	
75	C77	Channel	MH80	INLET18	22.65	1460.34	1459.92	1.8600	500.000	0.0150	3.34	6.83	0.49	3.69	0.34	0.68	0.00	
76	C78	Channel	INLET18	MH144	63.64	1459.92	1458.80	1.7500	500.000	0.0150	3.97	6.62	0.60	3.80	0.38	0.76	0.00	
77	C79	Channel	INLET19	MH84	88.21	1461.98	1460.78	1.3600	1000.000	0.0150	18.35	44.53	0.41	4.84	0.61	0.61	0.00	
78	C8	Channel	MH116	MH2	43.57	1476.94	1476.00	2.1700	1000.000	0.0150	18.14	56.22	0.32	5.65	0.53	0.53	0.00	
79	C80	Channel	MH84	MH85	202.25	1460.78	1456.52	2.1000	1000.000	0.0150	18.35	55.34	0.33	5.61	0.54	0.54	0.00	
80	C81	Channel	MH85	MH86	85.53	1456.52	1455.79	0.8600	1000.000	0.0150	18.35	35.44	0.52	4.14	0.69	0.69	0.00	
81	C82	Channel	MH86	MH87	28.19	1455.79	1455.41	1.3300	1000.000	0.0150	18.35	44.00	0.42	4.80	0.61	0.61	0.00	
82	C83	Channel	MH8	MH144	50.78	1459.54	1458.30	2.4400	1000.000	0.0150	24.66	59.64	0.41	6.49	0.61	0.61	0.00	
83	C84	Channel	INLET8	MH141	135.76	1471.23	1470.40	0.6100	500.000	0.0150	1.20	3.91	0.31	1.85	0.26	0.52	0.00	
84	C85	Channel	MH141	MH99	135.81	1470.40	1466.49	2.8800	500.000	0.0150	1.19	8.50	0.14	3.08	0.17	0.33	0.00	
85	C86	Channel	INLET7	MH140	135.88	1471.14	1470.24	0.6600	500.000	0.0150	0.00	4.08	0.00	0.00	0.00	0.00	0.00	
86	C87	Channel	MH140	MH101	135.92	1470.24	1466.65	2.6400	500.000	0.0150	0.00	8.14	0.00	0.00	0.00	0.00	0.00	
87	C88	Channel	INLET9	MH103	191.42	1494.91	1489.37	2.8900	500.000	0.0150	1.96	8.51	0.23	3.65	0.22	0.44	0.00	
88	C89	Channel	MH103	MH104	62.21	1489.37	1485.66	5.9700	500.000	0.0150	1.96	12.23	0.16	4.62	0.18	0.36	0.00	
89	C9	Channel	MH144	MH12	18.45	1458.30	1457.86	2.4200	1000.000	0.0150	27.83	59.39	0.47	6.73	0.66	0.66	0.00	
90	C90	Channel	MH104	MH125	90.73	1485.66	1477.60	8.8800	500.000	0.0150	1.96	14.91	0.13	5.27	0.16	0.32	0.00	
91	C91	Channel	MH125	MH105	107.52	1477.60	1465.60	11.1600	500.000	0.0150	1.95	16.73	0.12	5.69	0.15	0.30	0.00	
92	C92	Channel	INLET10	MH107	186.50	1493.15	1488.49	2.5000	1000.000	0.0150	1.03	30.59	0.03	2.83	0.16	0.16	0.00	
93	C93	Channel	MH107	MH108	57.65	1488.49	1484.97	6.1000	1000.000	0.0150	1.03	47.78	0.02	3.74	0.12	0.12	0.00	
94	C94	Channel	MH108	MH126	81.07	1484.97	1477.05	9.7800	1000.000	0.0150	1.03	60.50	0.02	4.36	0.11	0.11	0.00	

Link Summary

SN	Element ID	Element Type	From (Inlet) Node	To (Outlet) Node	Length (m)	Inlet Invert Elevation (m)	Outlet Invert Elevation (m)	Average Slope (%)	Diameter or Height (mm)	Manning's Roughness	Peak Flow (cms)	Design Flow Capacity (cms)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (m/sec)	Peak Flow Depth (m)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Reported Condition
95	C95	Channel	MH126	MH109	81.07	1477.05	1469.12	9.7700	1000.000	0.0150	1.03	60.50	0.02	4.36	0.11	0.11	0.00	
96	C96	Channel	MH76	MH68	57.09	1452.53	1450.29	3.9200	500.000	0.0150	1.84	9.91	0.19	3.94	0.19	0.39	0.00	
97	C97	Channel	MH5	MH145	70.57	1463.08	1461.98	1.5600	1000.000	0.0150	18.12	47.68	0.38	5.05	0.58	0.58	0.00	
98	C98	Channel	MH145	INLET81	62.23	1461.98	1459.67	3.7100	1000.000	0.0150	18.12	73.47	0.25	6.75	0.45	0.45	0.00	

Subbasin Hydrology

Subbasin : {PHASE 1}.Catchment : 1

Input Data

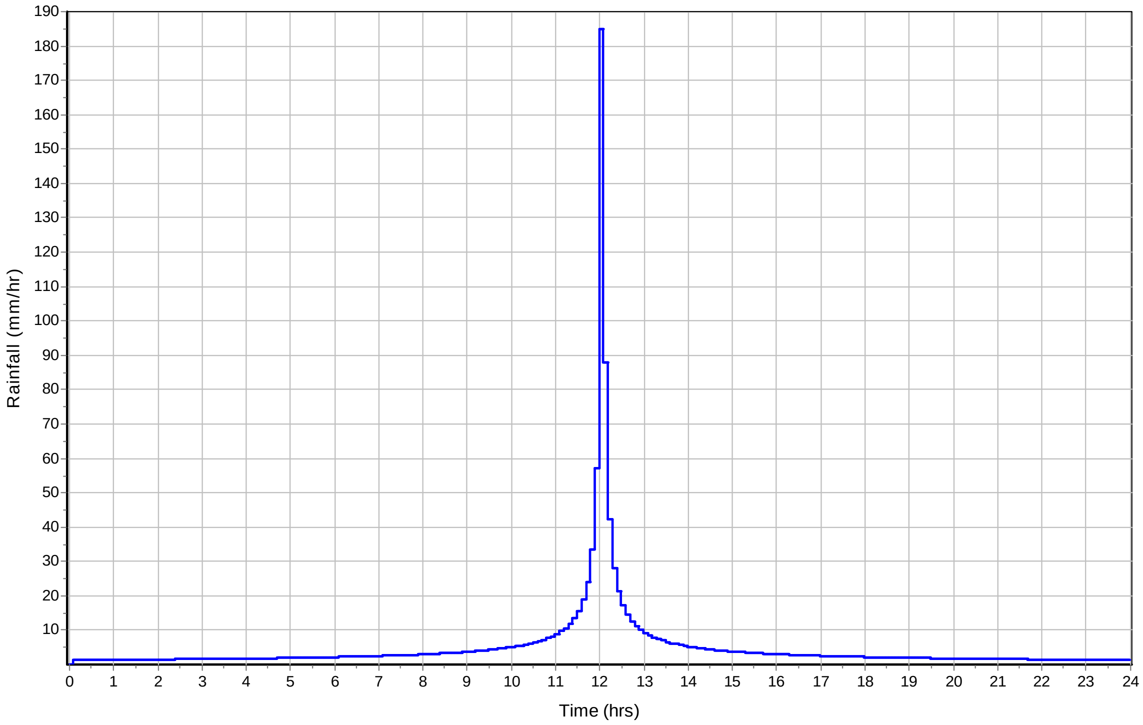
Area (ha)	40.01
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	2.19
Equivalent Width (m)	1825
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

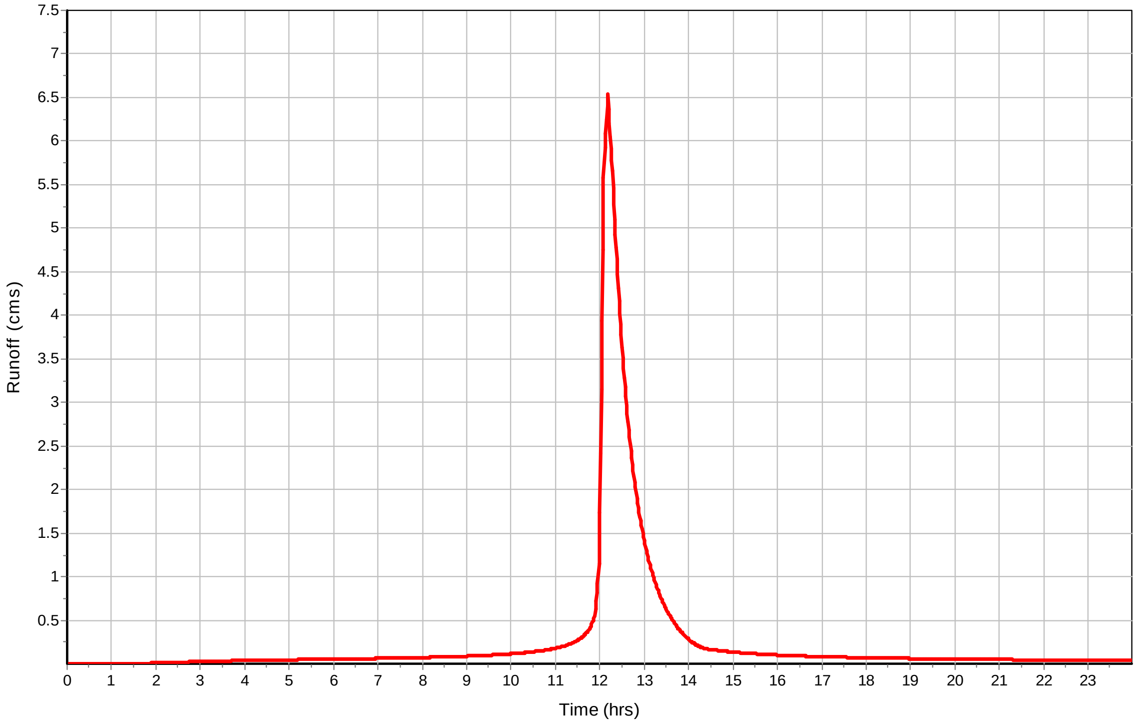
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	66.328
Total Runoff (mm)	54.6
Peak Runoff (cms)	6.54
Time of Concentration (days hh:mm:ss)	0 01:12:09

Subbasin : {PHASE 1}.Catchment : 1

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 10

Input Data

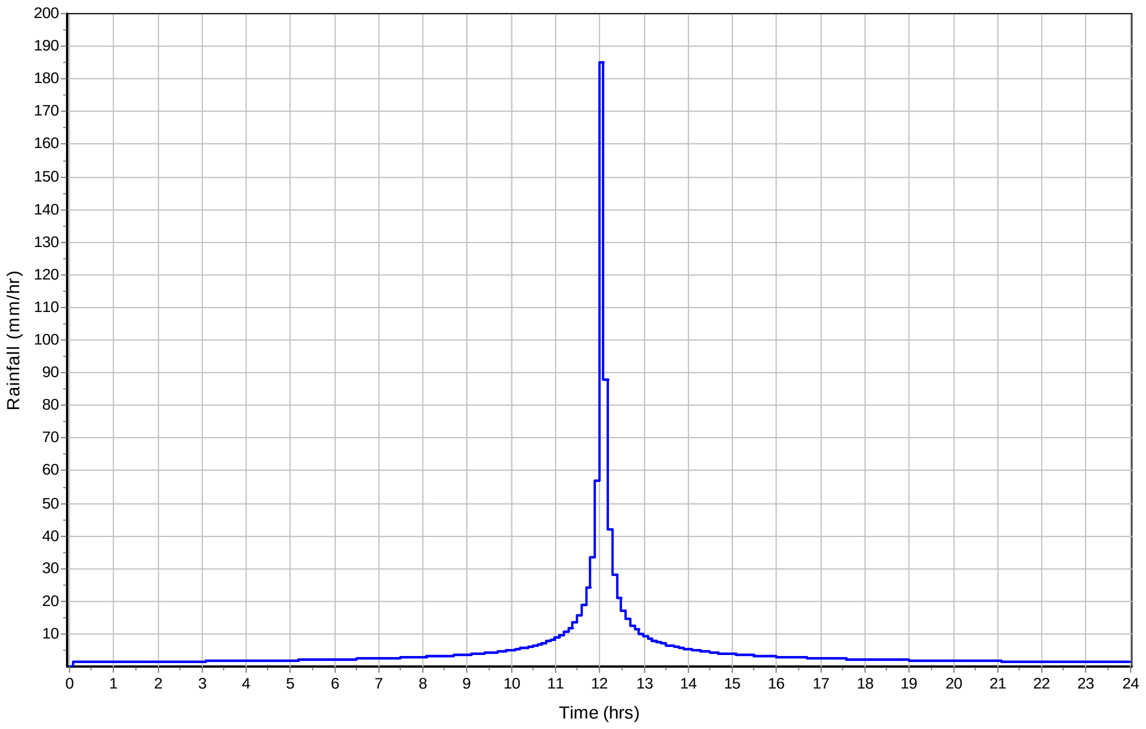
Area (ha)	12.1
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	9.71
Equivalent Width (m)	801.6
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

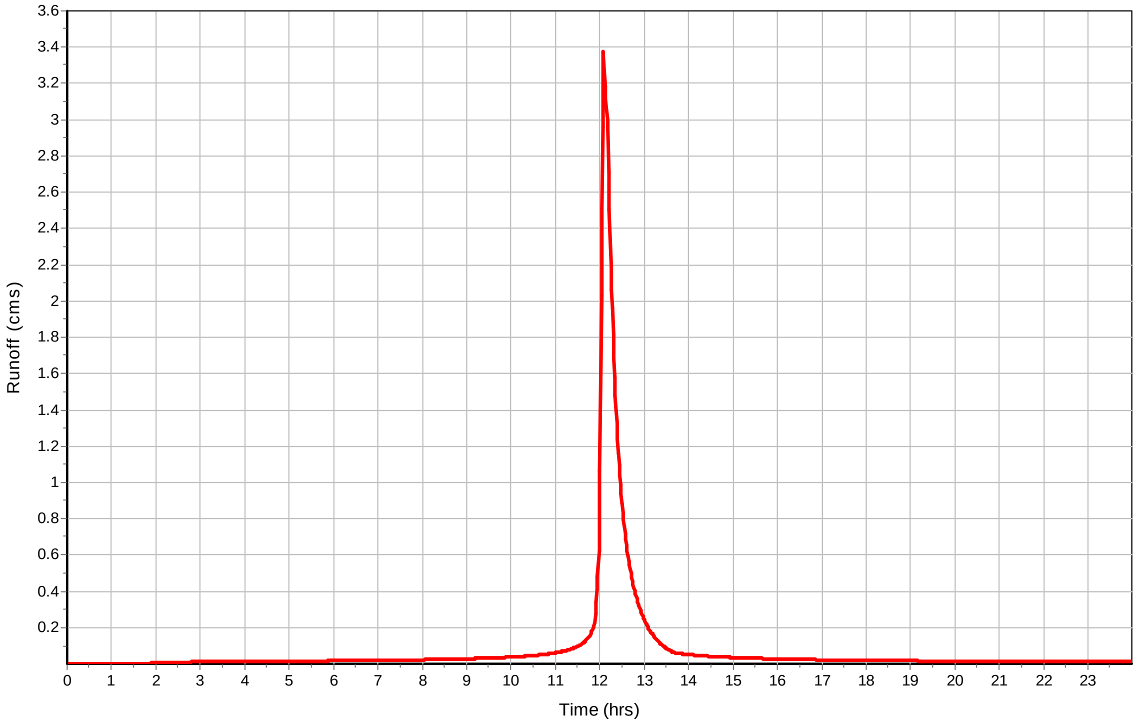
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	63.982
Total Runoff (mm)	57.26
Peak Runoff (cms)	3.37
Time of Concentration (days hh:mm:ss)	0 00:36:53

Subbasin : {PHASE 1}.Catchment : 10

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 11

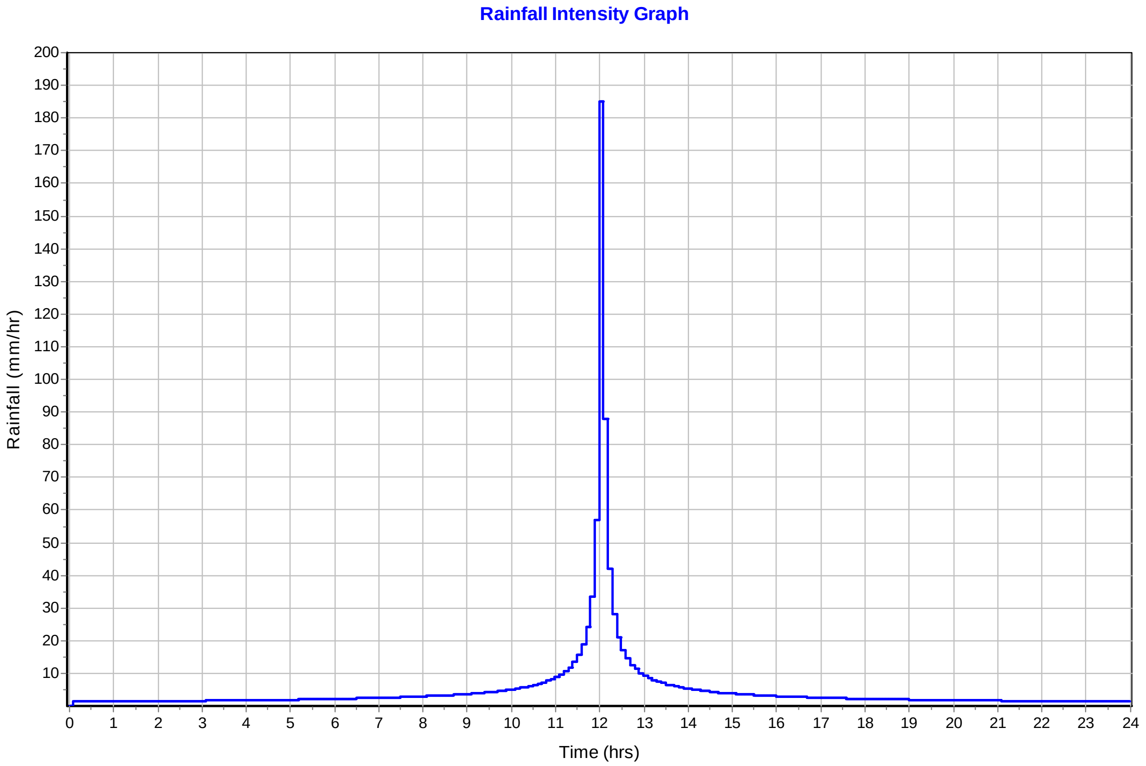
Input Data

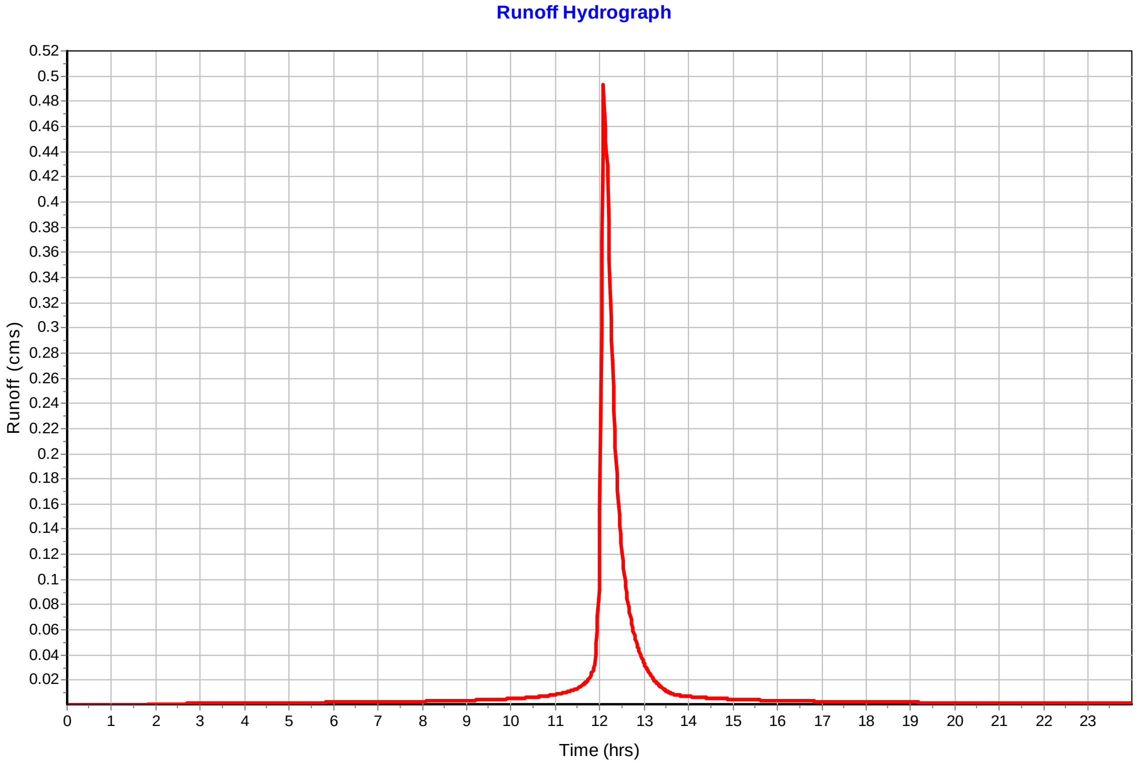
Area (ha)	1.7
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	5.33
Equivalent Width (m)	163.6
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	63.892
Total Runoff (mm)	57.36
Peak Runoff (cms)	0.49
Time of Concentration (days hh:mm:ss)	0 00:35:21

Subbasin : {PHASE 1}.Catchment : 11





Subbasin : {PHASE 1}.Catchment : 12

Input Data

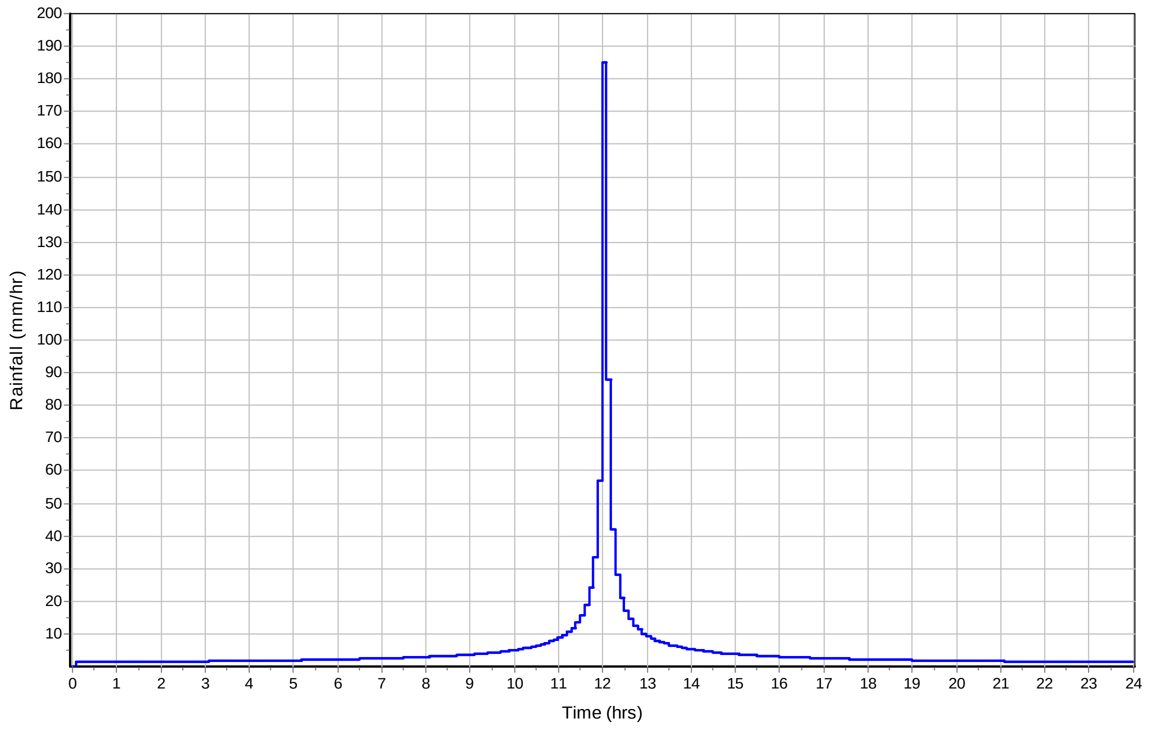
Area (ha)	3.17
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	1.61
Equivalent Width (m)	289.6
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

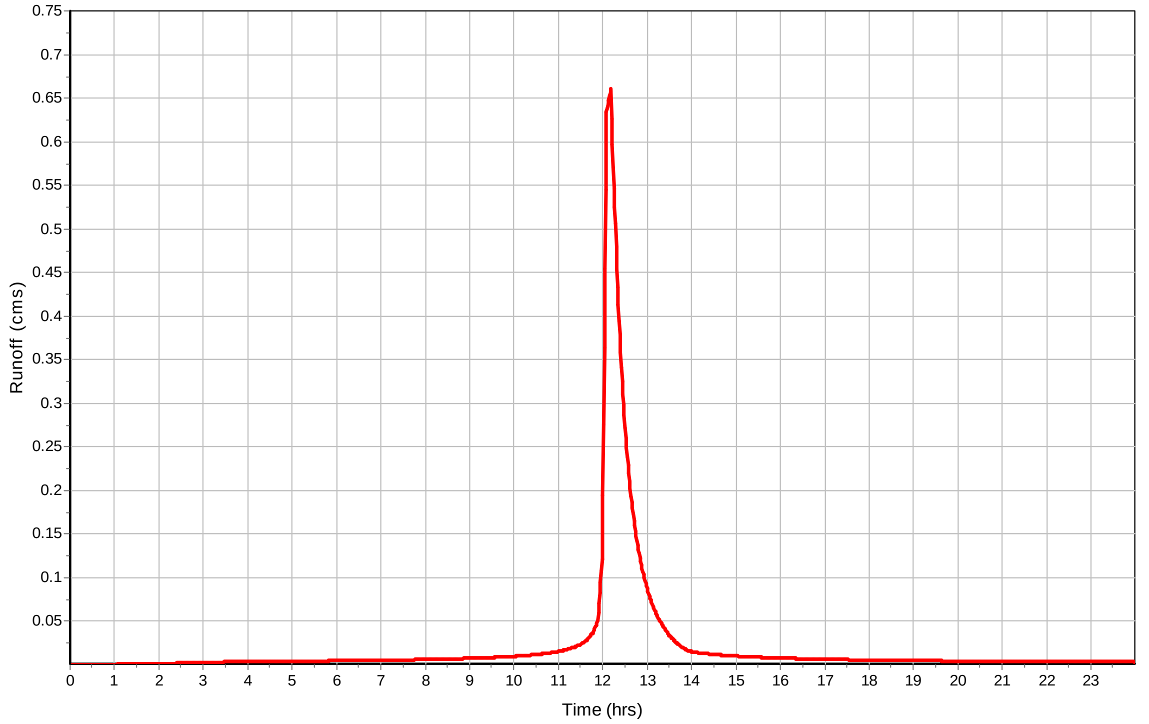
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	64.95
Total Runoff (mm)	56.16
Peak Runoff (cms)	0.66
Time of Concentration (days hh:mm:ss)	0 00:52:10

Subbasin : {PHASE 1}.Catchment : 12

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 13

Input Data

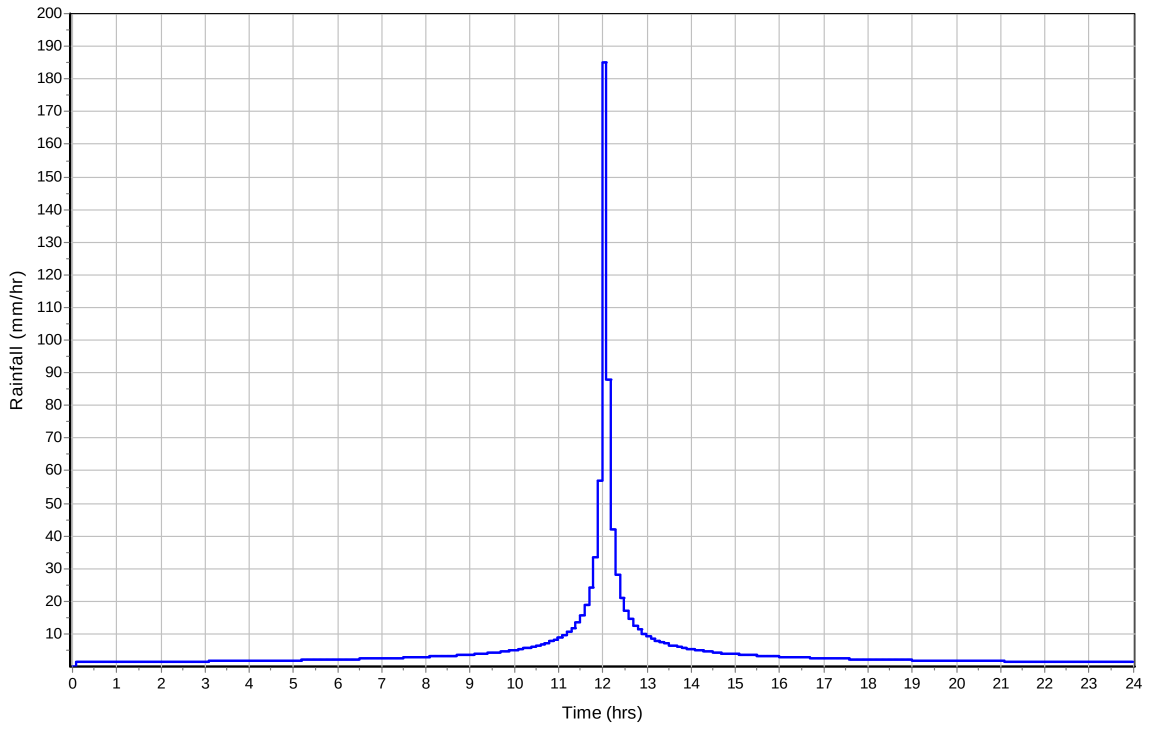
Area (ha)	30.53
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	1.85
Equivalent Width (m)	1968.8
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

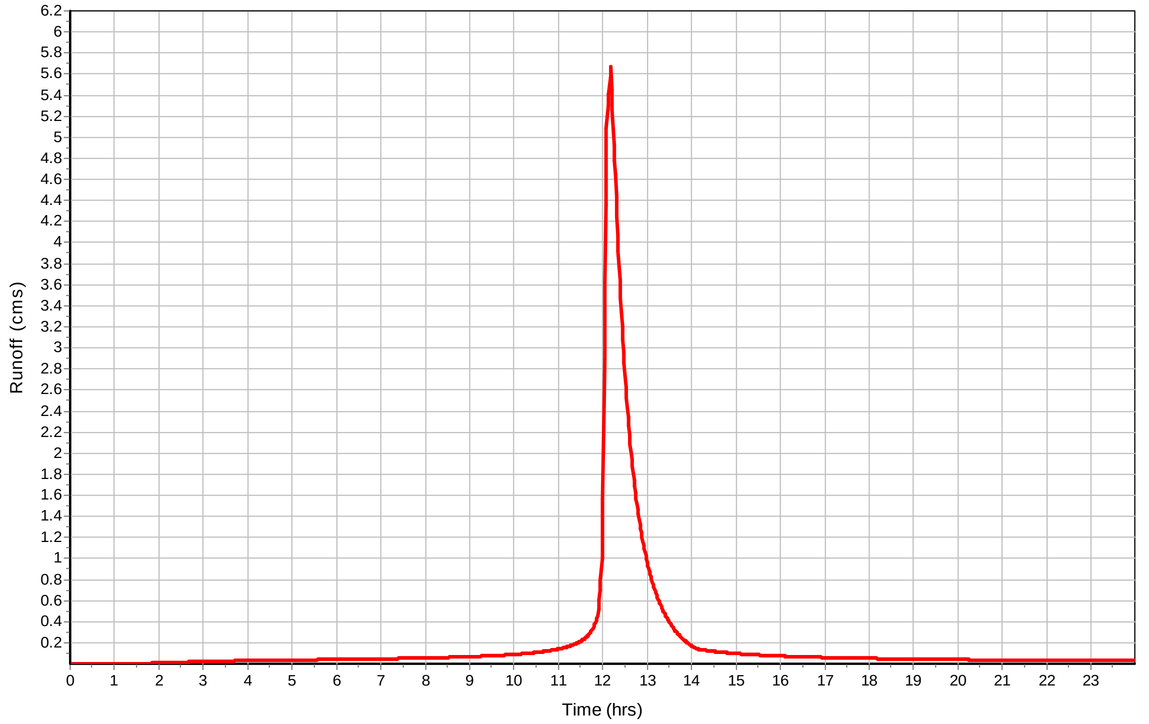
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	65.596
Total Runoff (mm)	55.43
Peak Runoff (cms)	5.67
Time of Concentration (days hh:mm:ss)	0 01:01:39

Subbasin : {PHASE 1}.Catchment : 13

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 14

Input Data

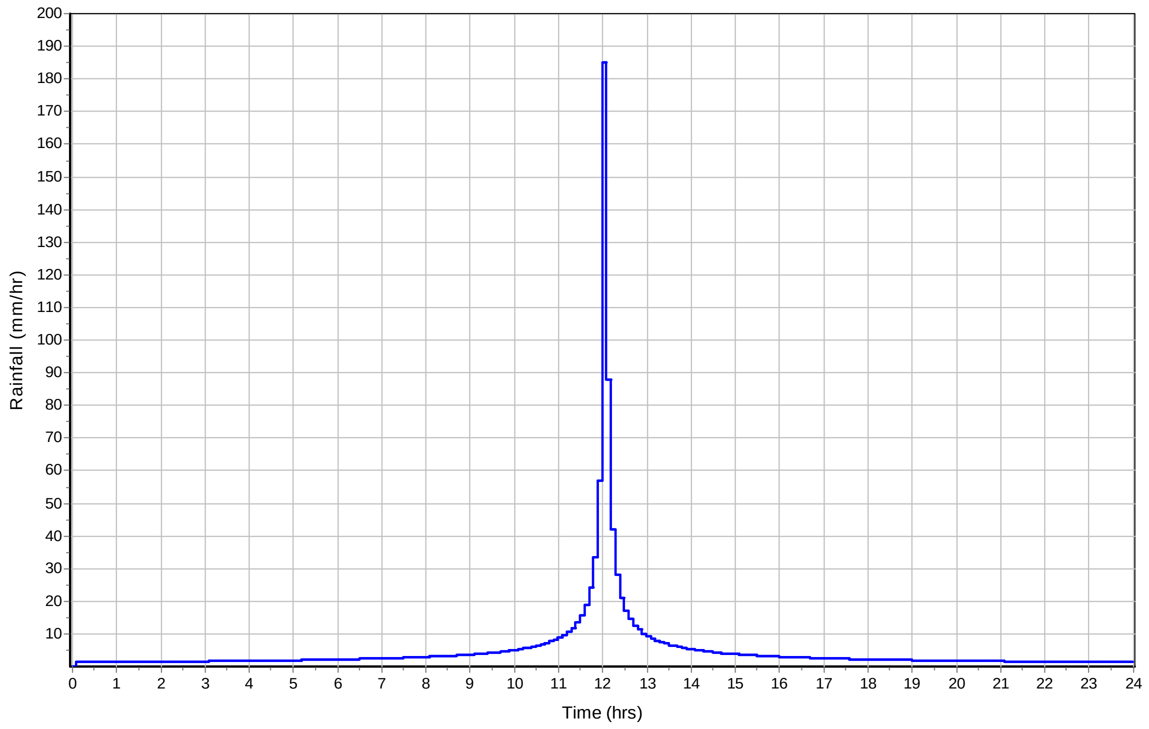
Area (ha)	9.1
Impervious Area (%)	30
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	1.76
Equivalent Width (m)	810.4
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

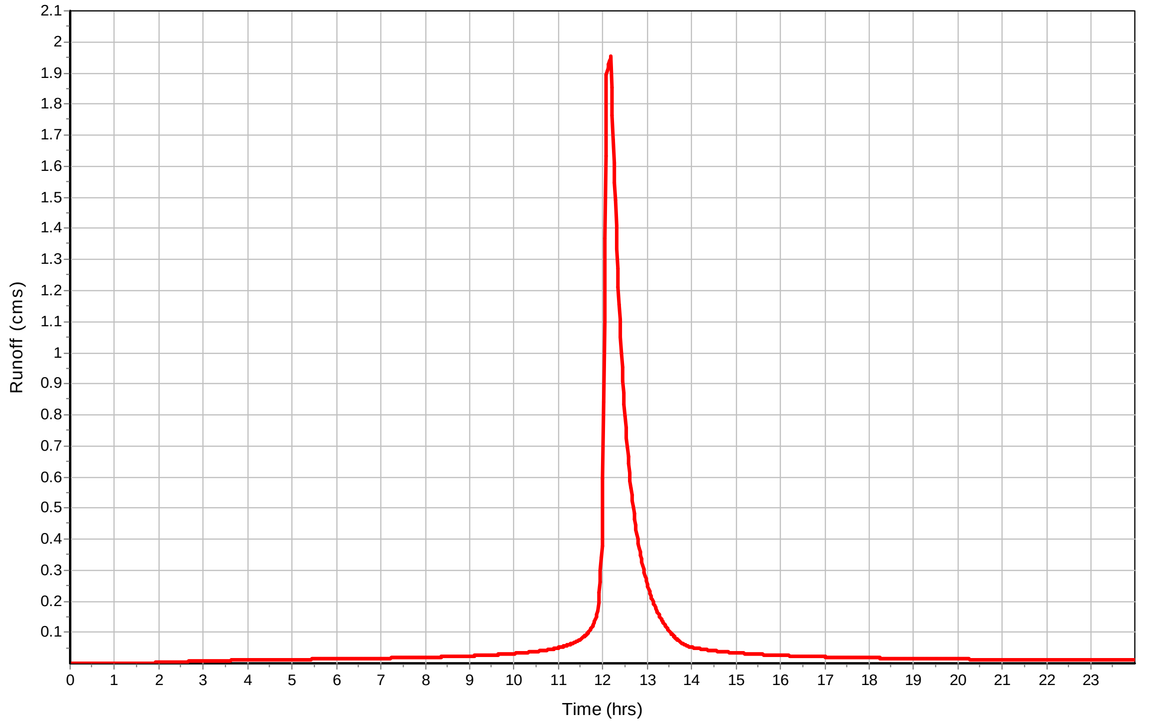
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	60.454
Total Runoff (mm)	60.44
Peak Runoff (cms)	1.96
Time of Concentration (days hh:mm:ss)	0 00:49:28

Subbasin : {PHASE 1}.Catchment : 14

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 15

Input Data

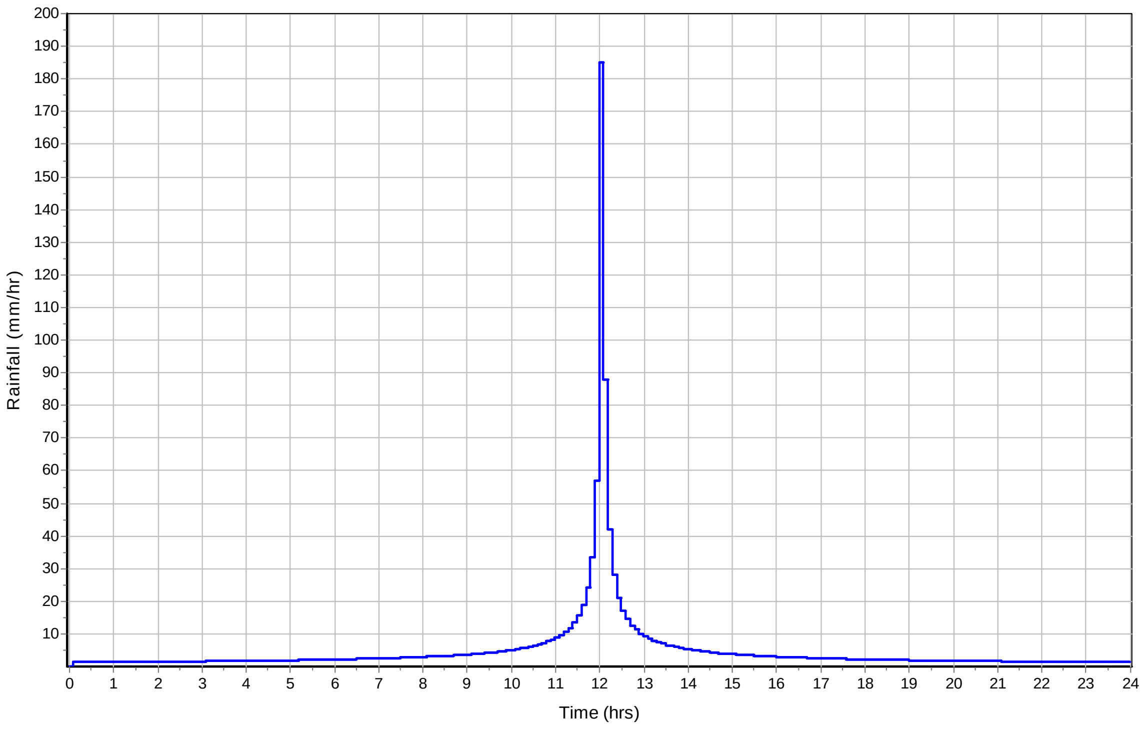
Area (ha)	30.49
Impervious Area (%)	30
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	2.16
Equivalent Width (m)	1055.7
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

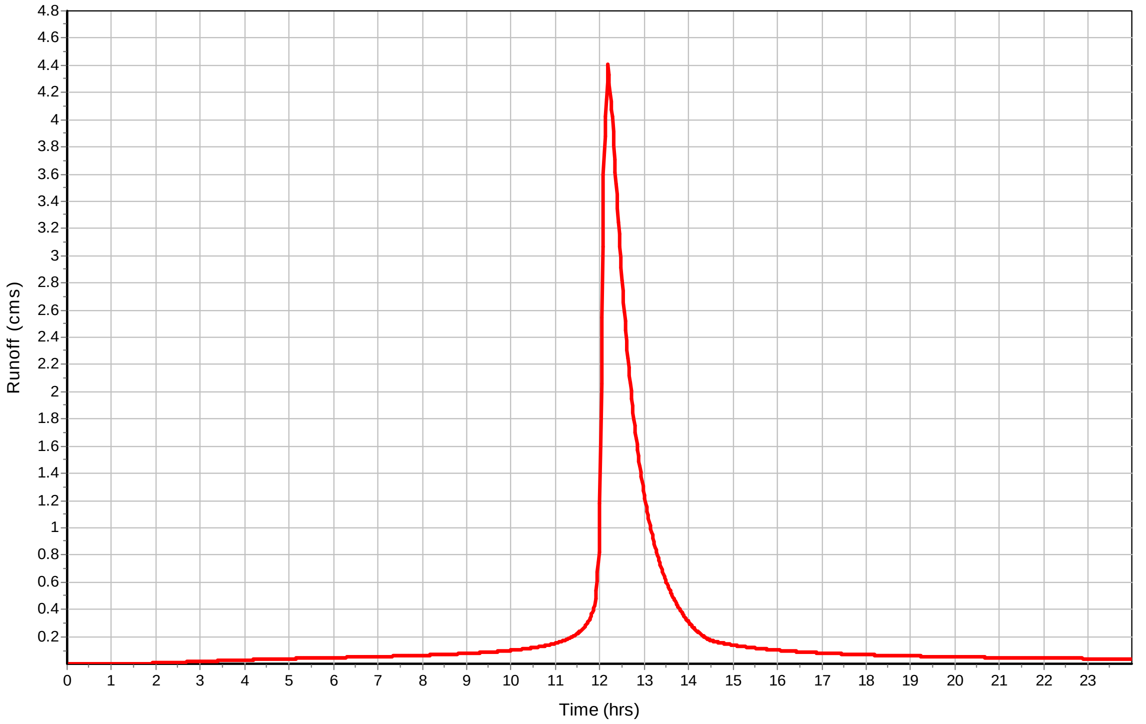
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	62.553
Total Runoff (mm)	57.94
Peak Runoff (cms)	4.41
Time of Concentration (days hh:mm:ss)	0 01:22:01

Subbasin : {PHASE 1}.Catchment : 15

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 18

Input Data

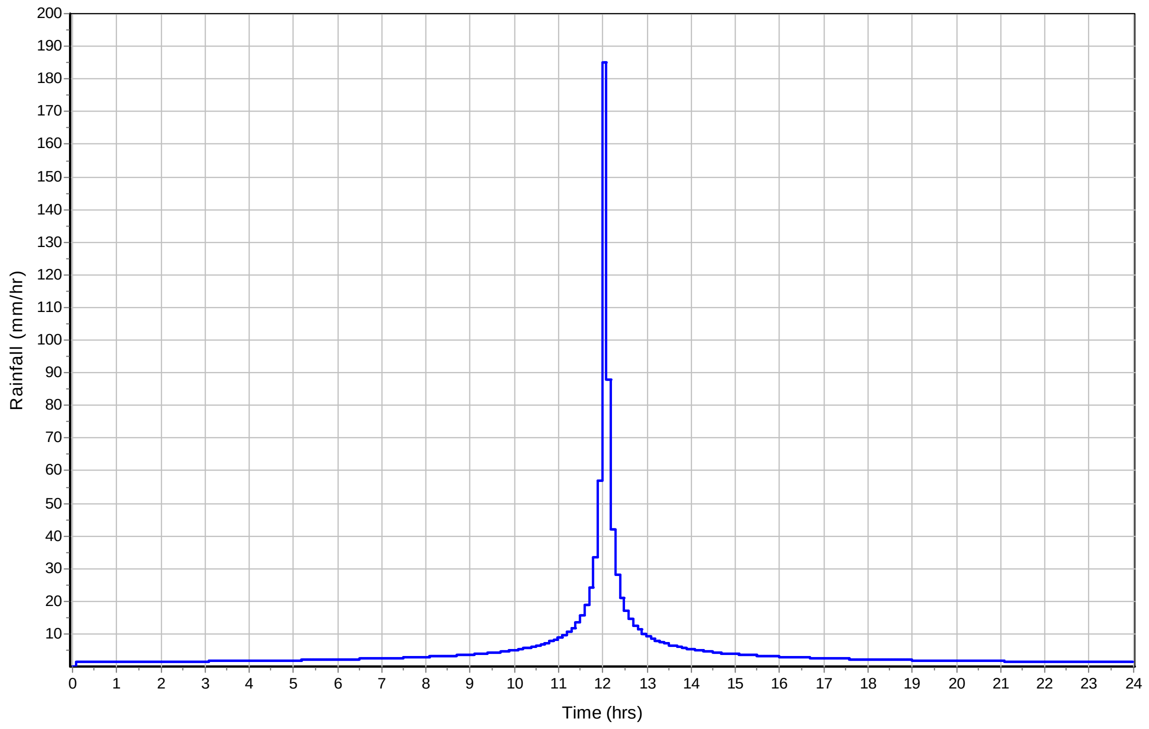
Area (ha)	10.49
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	7
Max Volume (mm)	0
Average Slope (%)	0.86
Equivalent Width (m)	1093
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

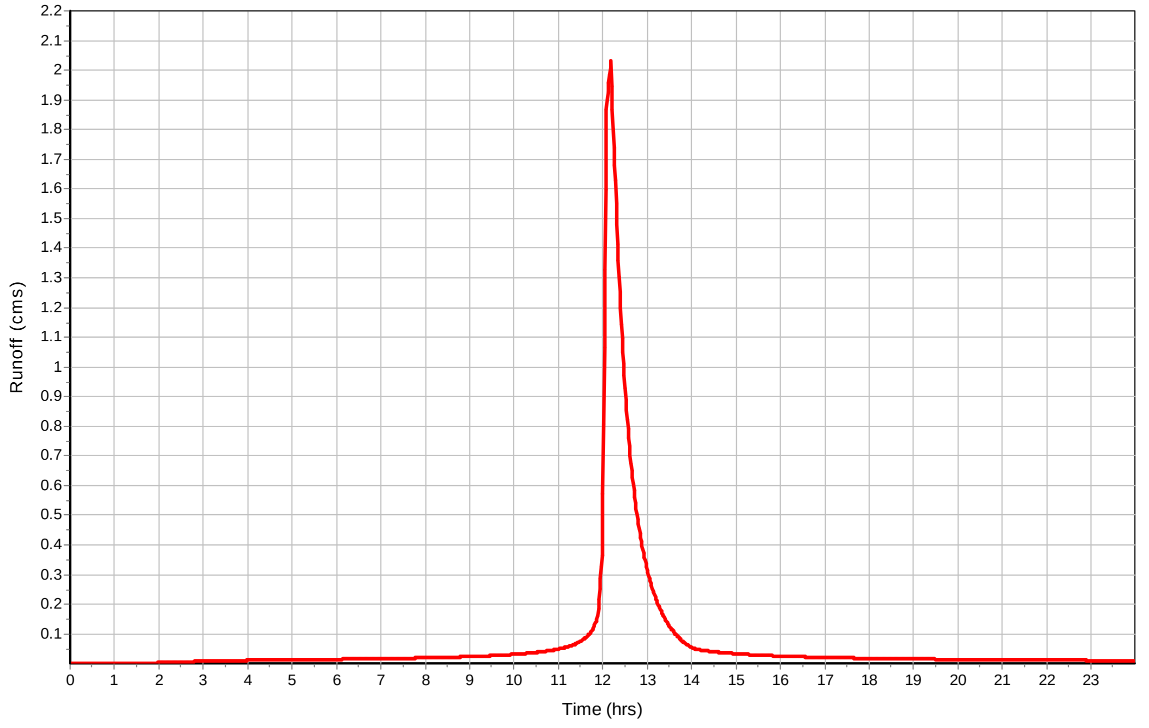
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	65.35
Total Runoff (mm)	55.7
Peak Runoff (cms)	2.03
Time of Concentration (days hh:mm:ss)	0 00:58:11

Subbasin : {PHASE 1}.Catchment : 18

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 19

Input Data

```
Area (ha) .......................................................... 5.54
Impervious Area (%) ......................................... 25
Max Infiltration Rate (mm/hr) .......................... 60
Min Infiltration Rate (mm/hr) .......................... 10
Drying Time (days) ........................................... 7
Decay Constant (1/hrs) ..................................... 7
Max Volume (mm) ........................................... 0
Average Slope (%) ............................................ 3.38
Equivalent Width (m) ....................................... 909
Impervious Area
    Manning's Roughness ................................. 0.3
Pervious Area
    Manning's Roughness ................................. 0.13
Curb & Gutter Length (m) ................................ 0
Rain Gage ID .................................................... Rain Gage-01
```

Subbasin Runoff Results

```
Total Rainfall (mm) .......................................... 121.89
Total Runon (mm) ............................................ 0
Total Evaporation (mm) ................................... 0
Total Infiltration (mm) ..................................... 63.554
Total Runoff (mm) ............................................ 57.75
Peak Runoff (cms) ............................................ 1.85
Time of Concentration (days hh:mm:ss) ........... 0  00:29:22
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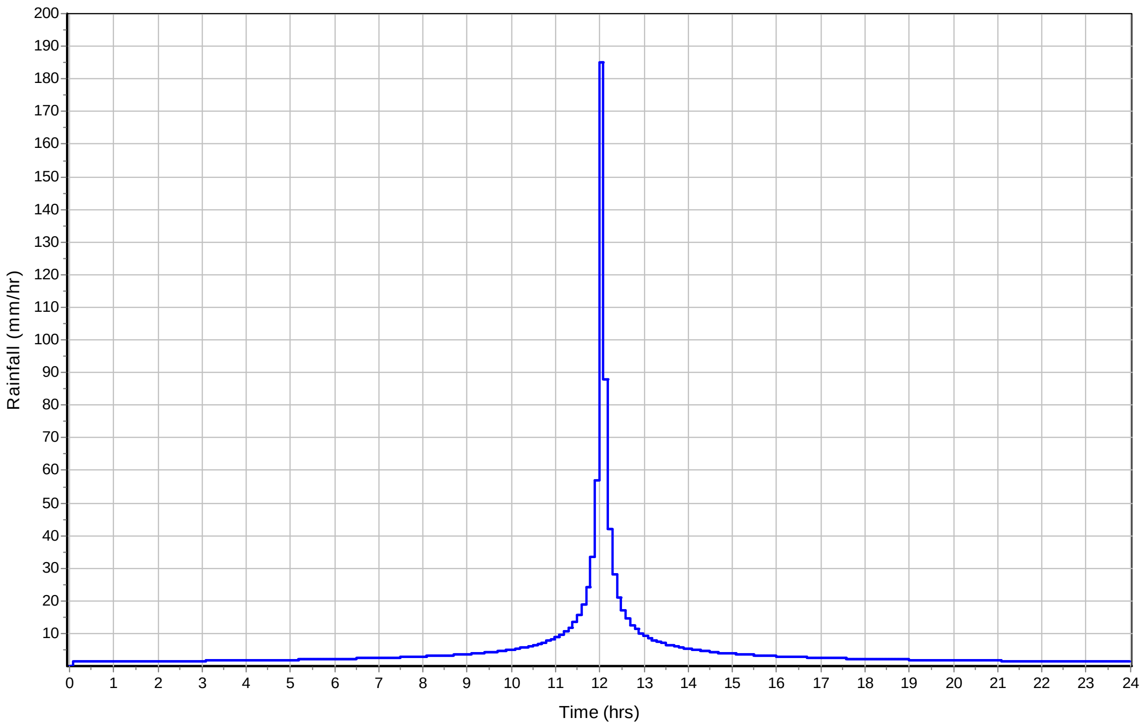
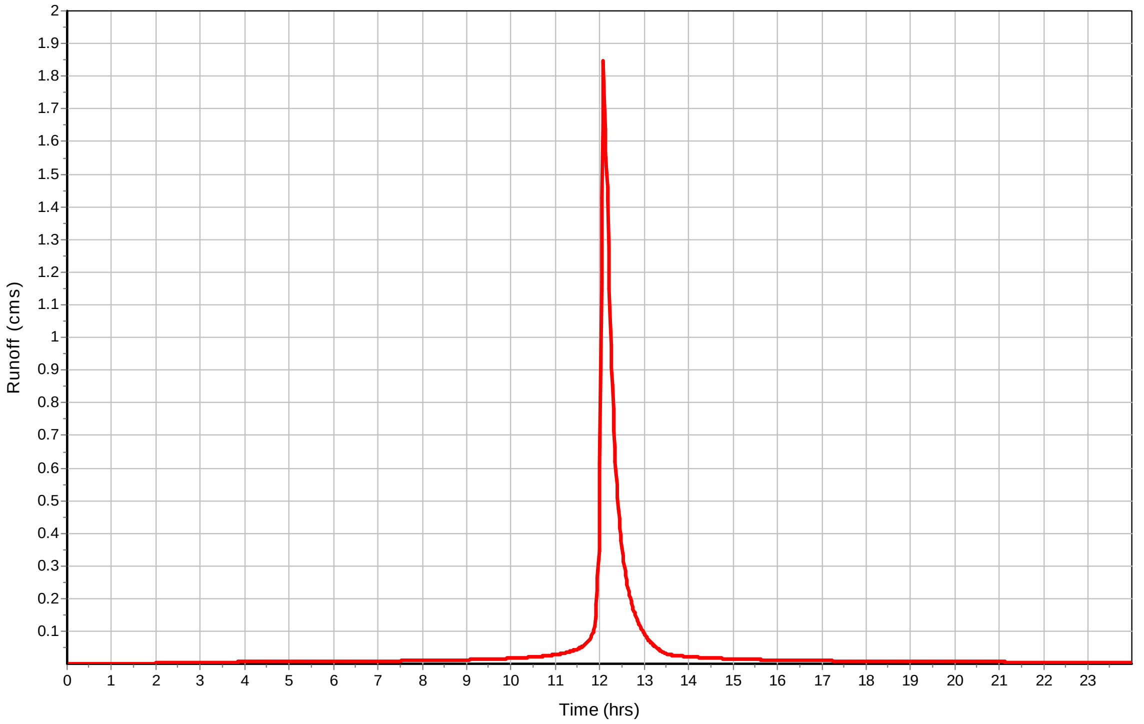
Subbasin : {PHASE 1}.Catchment : 19

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 2

Input Data

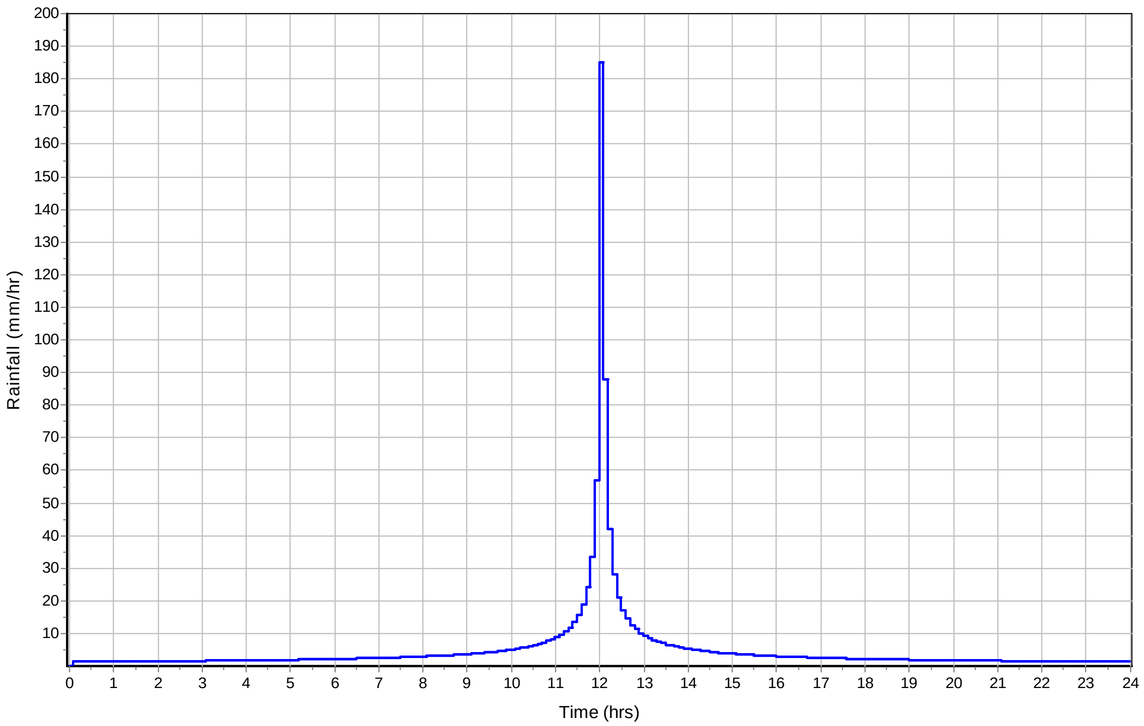
Area (ha)	4.94
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	4.98
Equivalent Width (m)	710.85
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

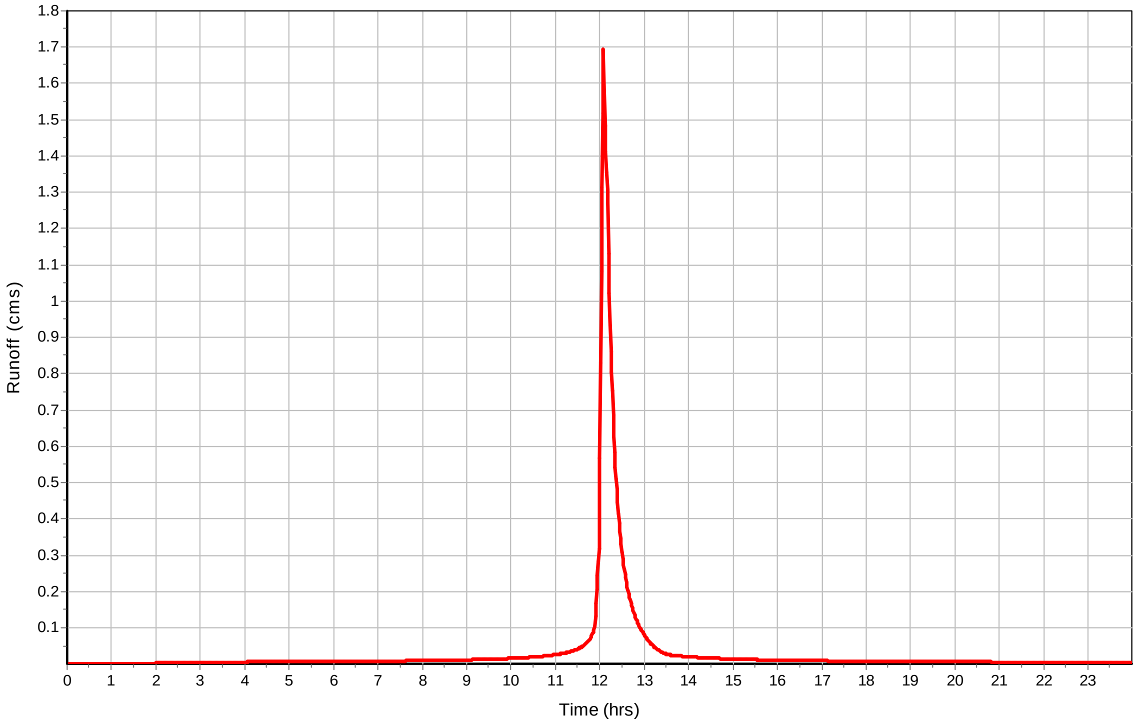
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	63.506
Total Runoff (mm)	57.81
Peak Runoff (cms)	1.69
Time of Concentration (days hh:mm:ss)	0 00:28:19

Subbasin : {PHASE 1}.Catchment : 2

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 3

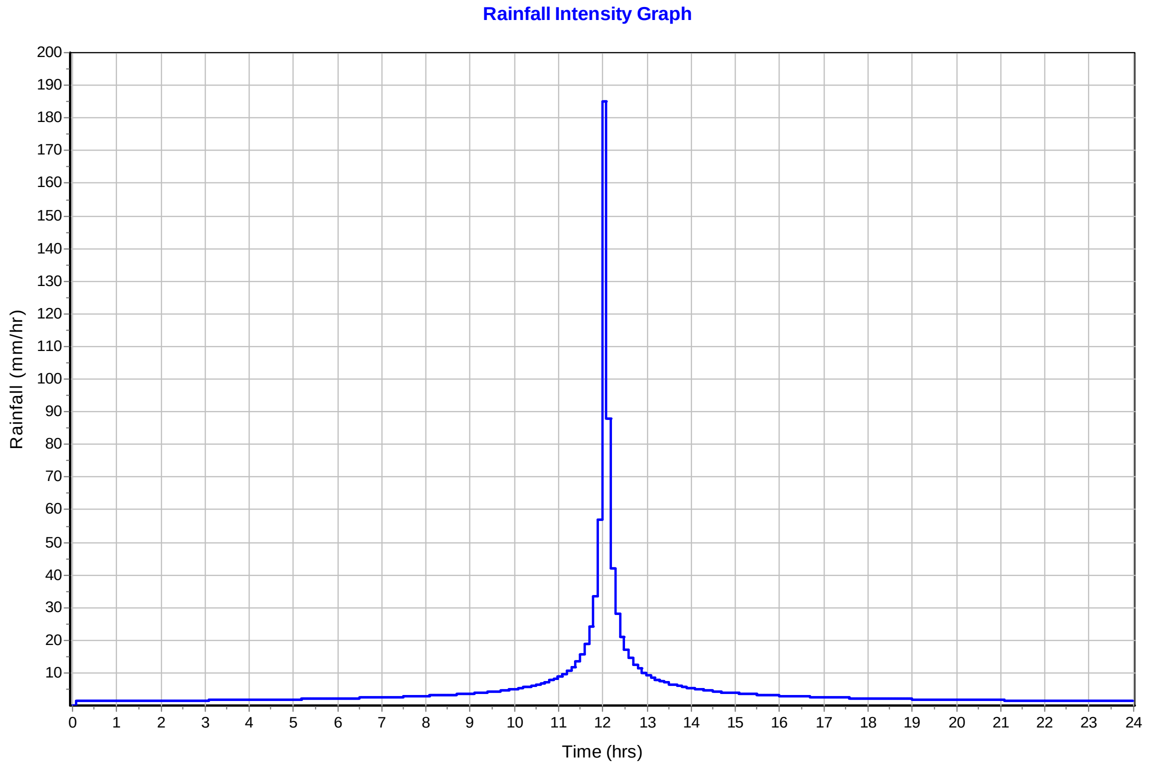
Input Data

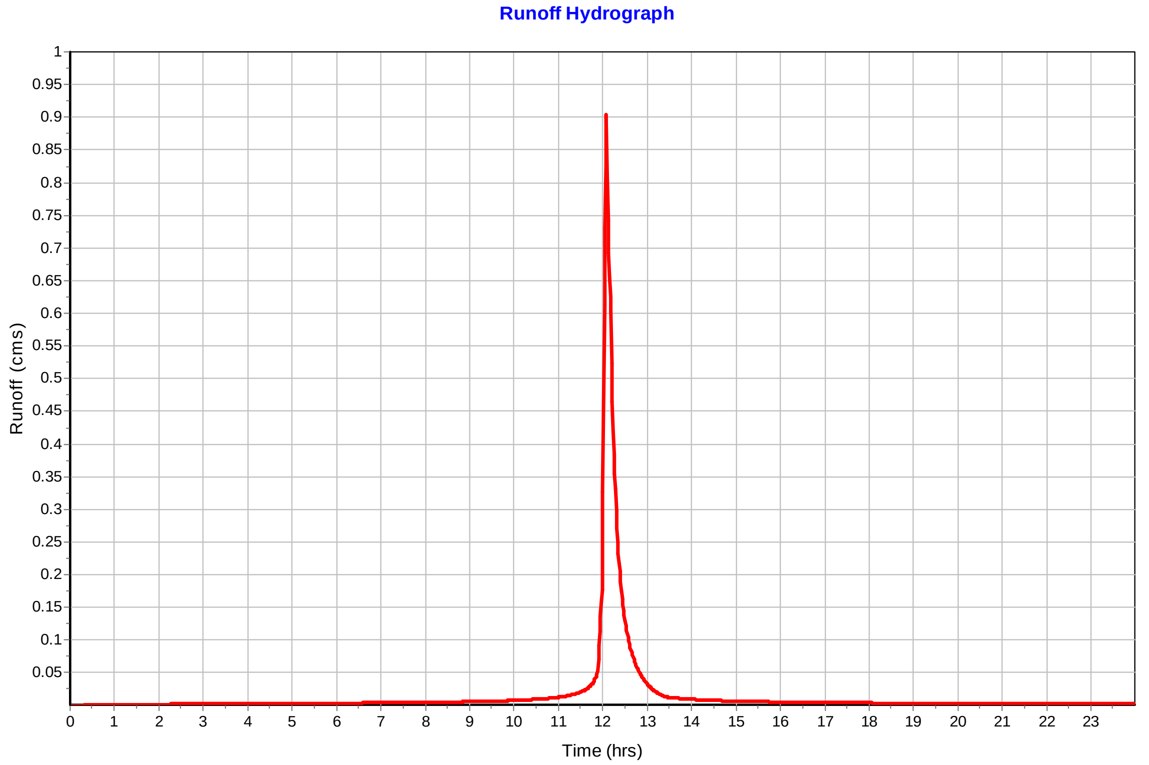
Area (ha)	2.34
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	17.04
Equivalent Width (m)	254.1
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	63.252
Total Runoff (mm)	58.11
Peak Runoff (cms)	0.9
Time of Concentration (days hh:mm:ss)	0 00:23:08

Subbasin : {PHASE 1}.Catchment : 3





Subbasin : {PHASE 1}.Catchment : 4

Input Data

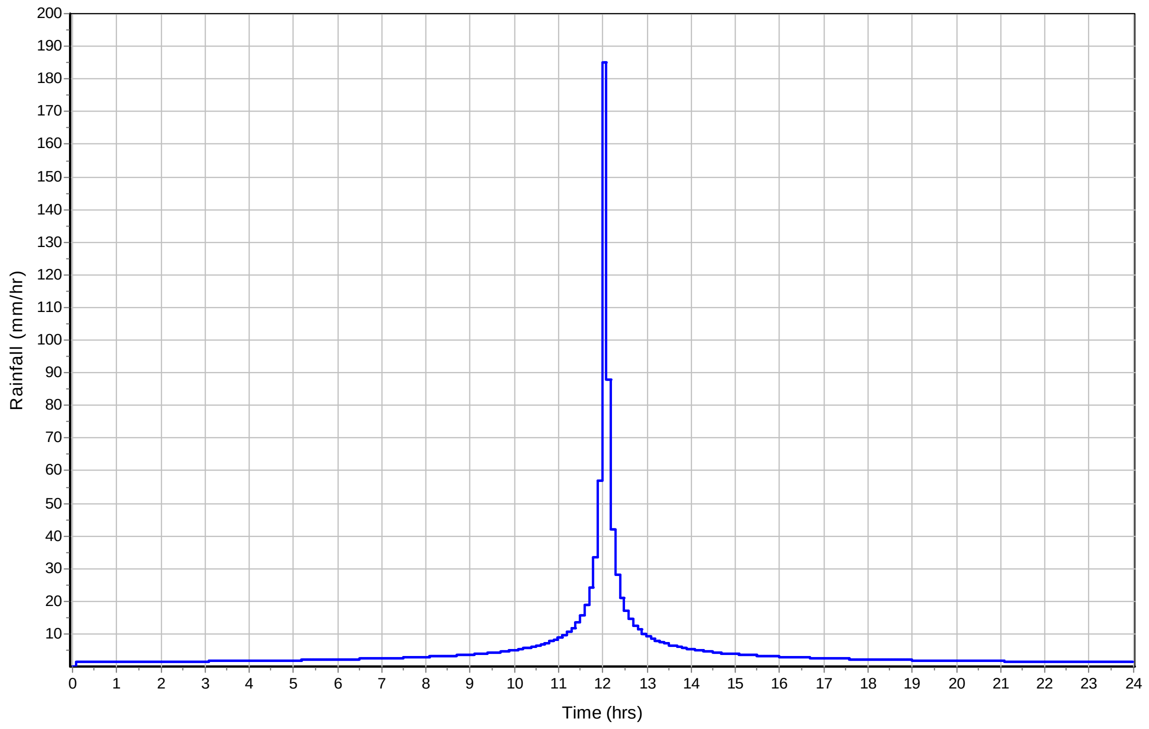
Area (ha)	3.55
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	13.28
Equivalent Width (m)	325.3
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

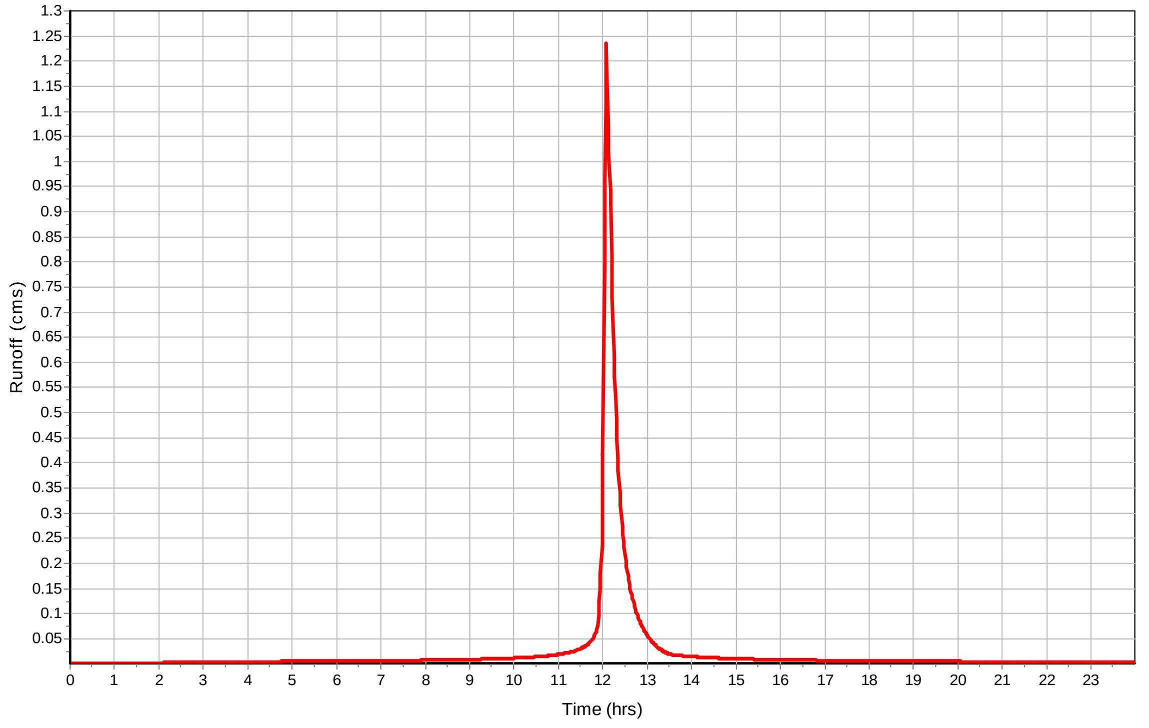
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	63.47
Total Runoff (mm)	57.85
Peak Runoff (cms)	1.23
Time of Concentration (days hh:mm:ss)	0 00:27:37

Subbasin : {PHASE 1}.Catchment : 4

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 5

Input Data

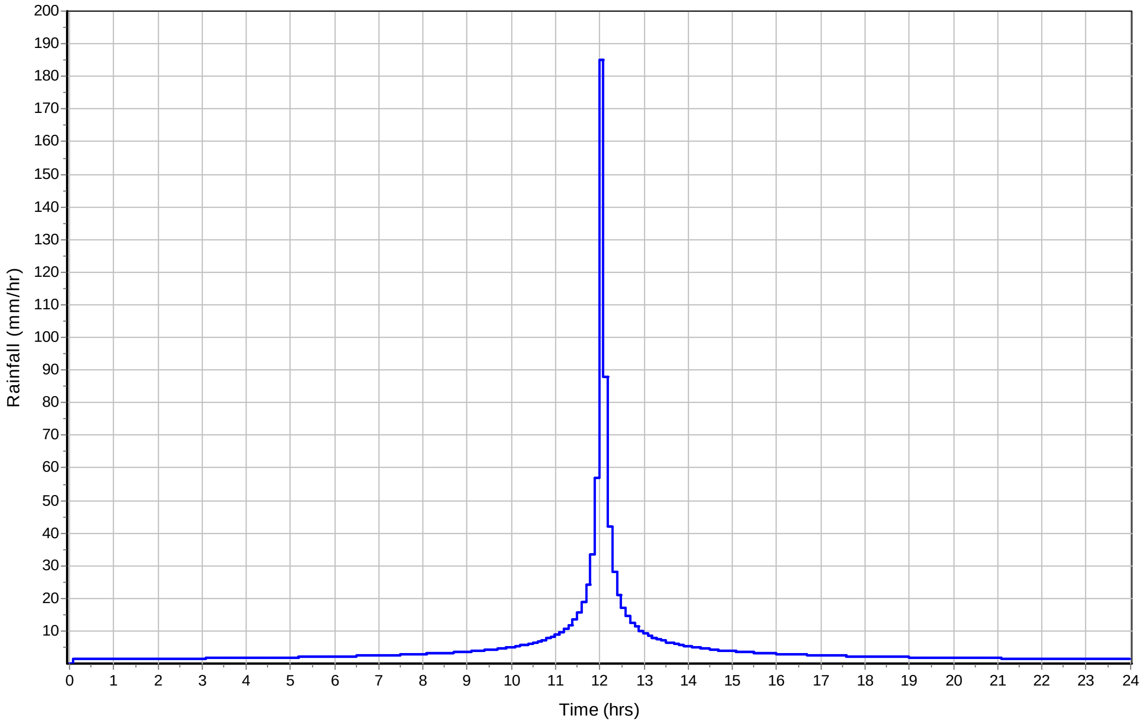
Area (ha)	2.75
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	8.88
Equivalent Width (m)	407.1
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

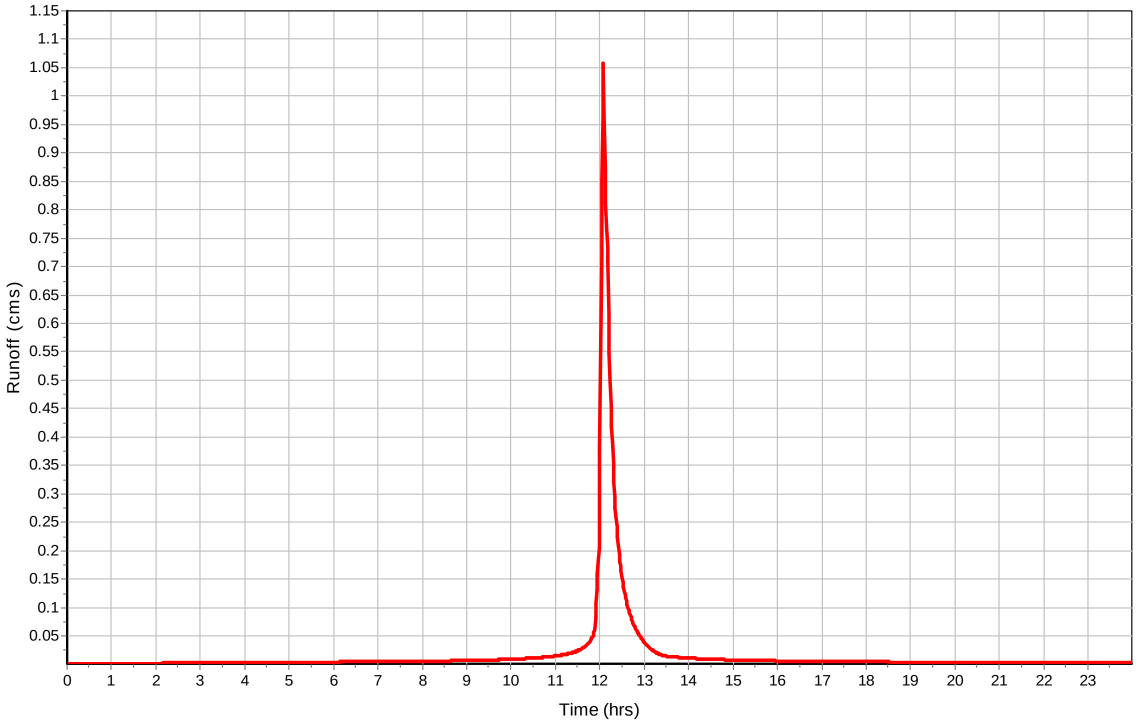
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	63.262
Total Runoff (mm)	58.1
Peak Runoff (cms)	1.06
Time of Concentration (days hh:mm:ss)	0 00:23:22

Subbasin : {PHASE 1}.Catchment : 5

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 6

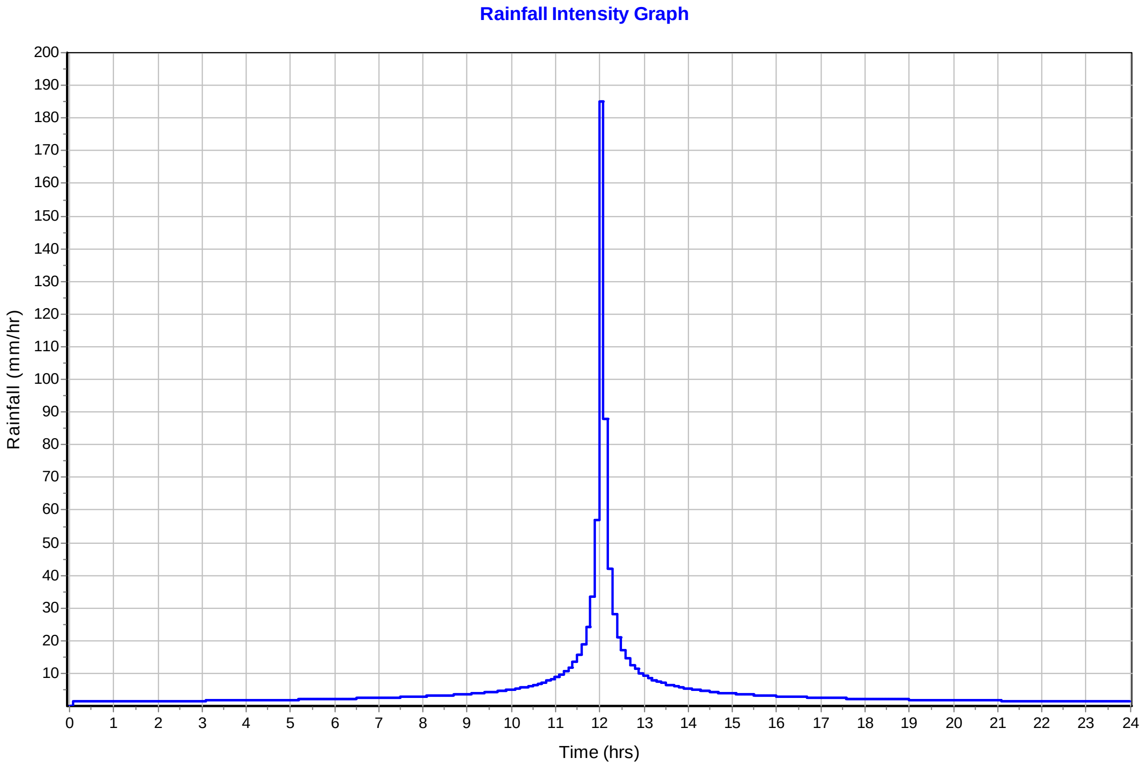
Input Data

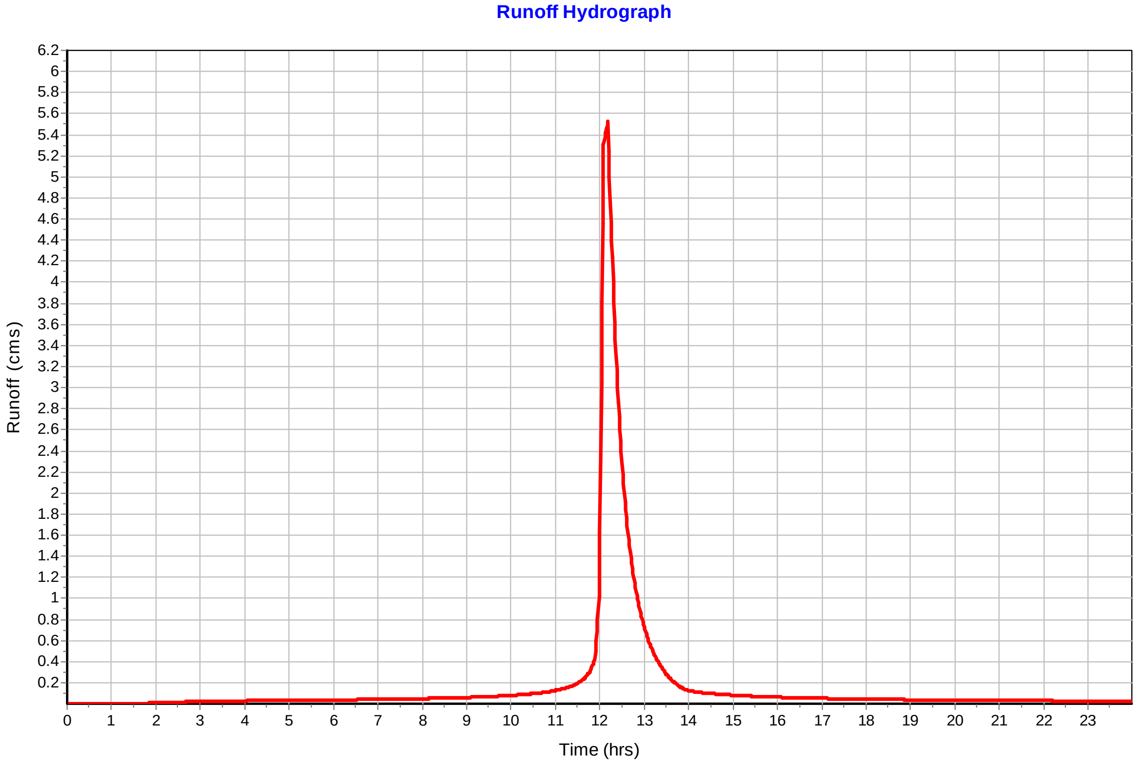
Area (ha)	26.53
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	4.01
Equivalent Width (m)	1535.7
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	64.95
Total Runoff (mm)	56.16
Peak Runoff (cms)	5.52
Time of Concentration (days hh:mm:ss)	0 00:52:10

Subbasin : {PHASE 1}.Catchment : 6





Subbasin : {PHASE 1}.Catchment : 7

Input Data

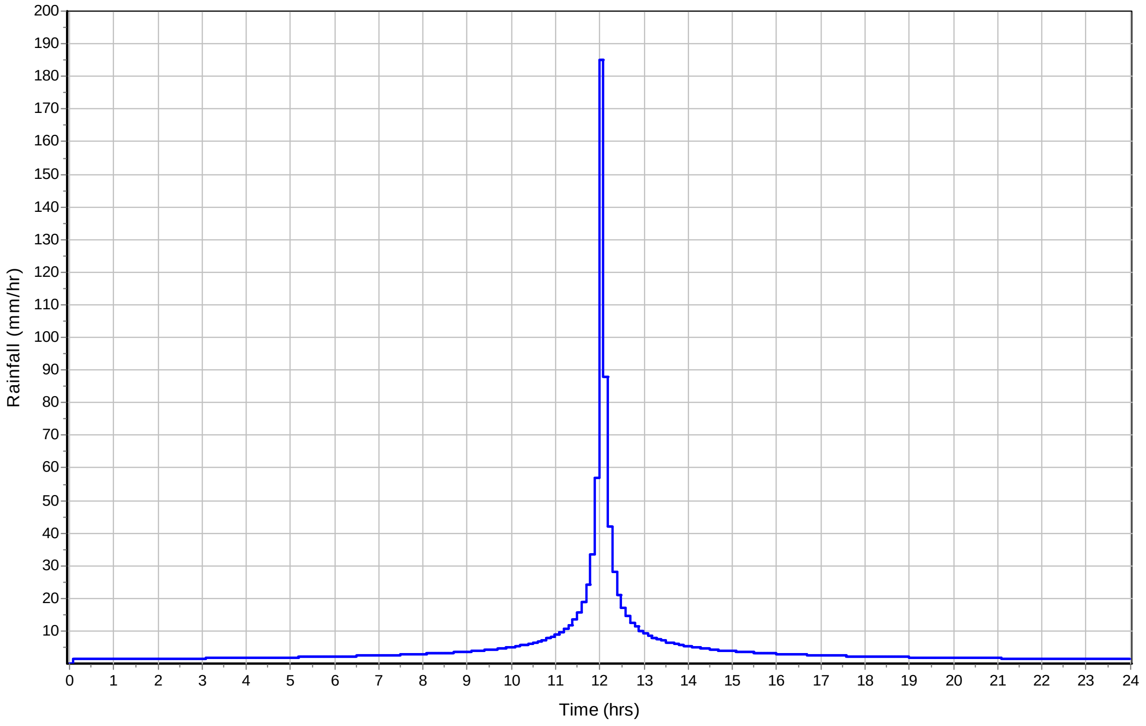
Area (ha)	27.52
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	12.62
Equivalent Width (m)	757.4
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

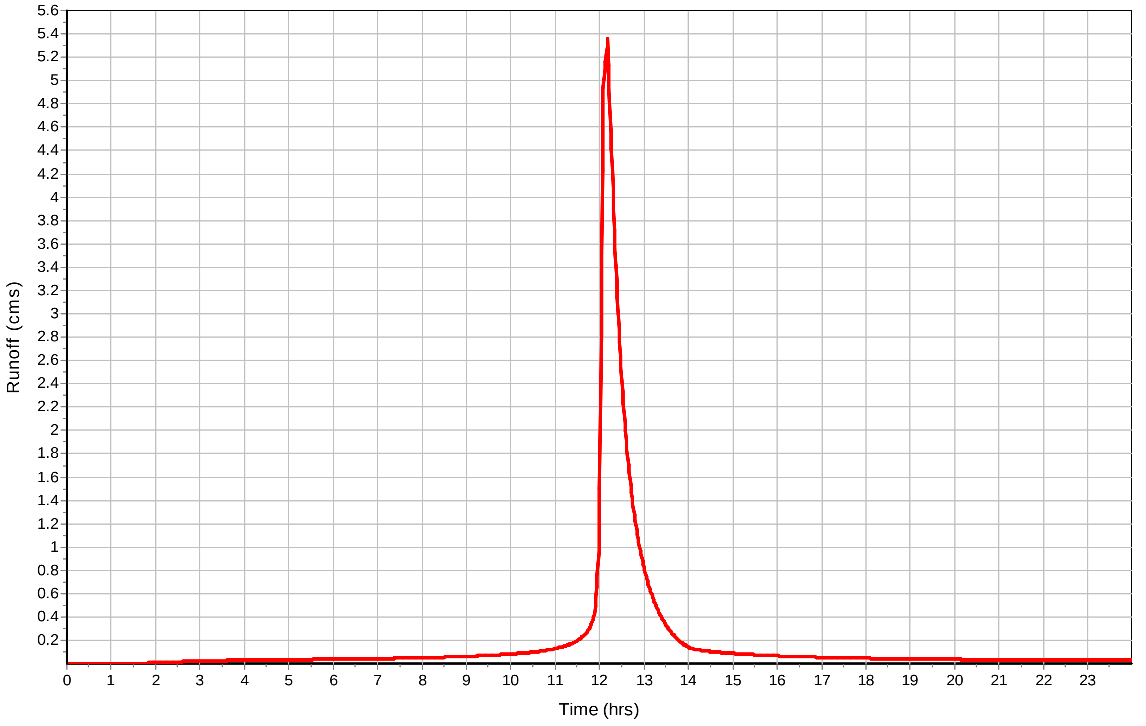
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	65.329
Total Runoff (mm)	55.73
Peak Runoff (cms)	5.36
Time of Concentration (days hh:mm:ss)	0 00:57:46

Subbasin : {PHASE 1}.Catchment : 7

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 8

Input Data

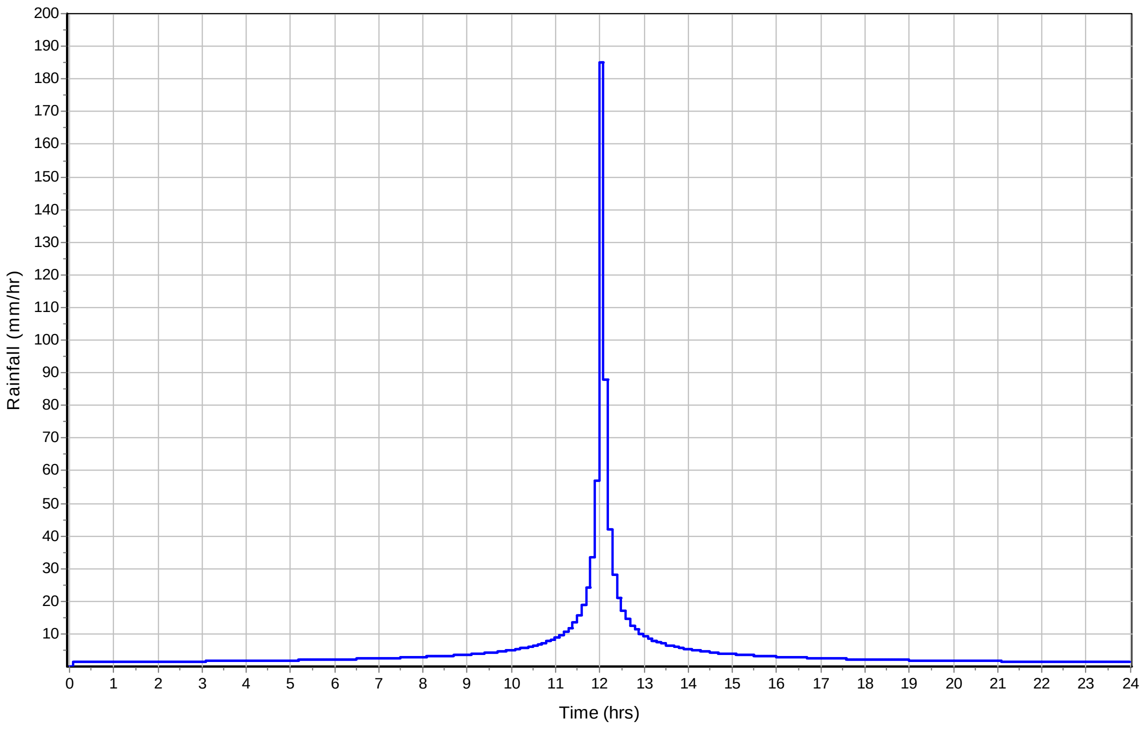
Area (ha)	8.47
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	4.57
Equivalent Width (m)	911
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

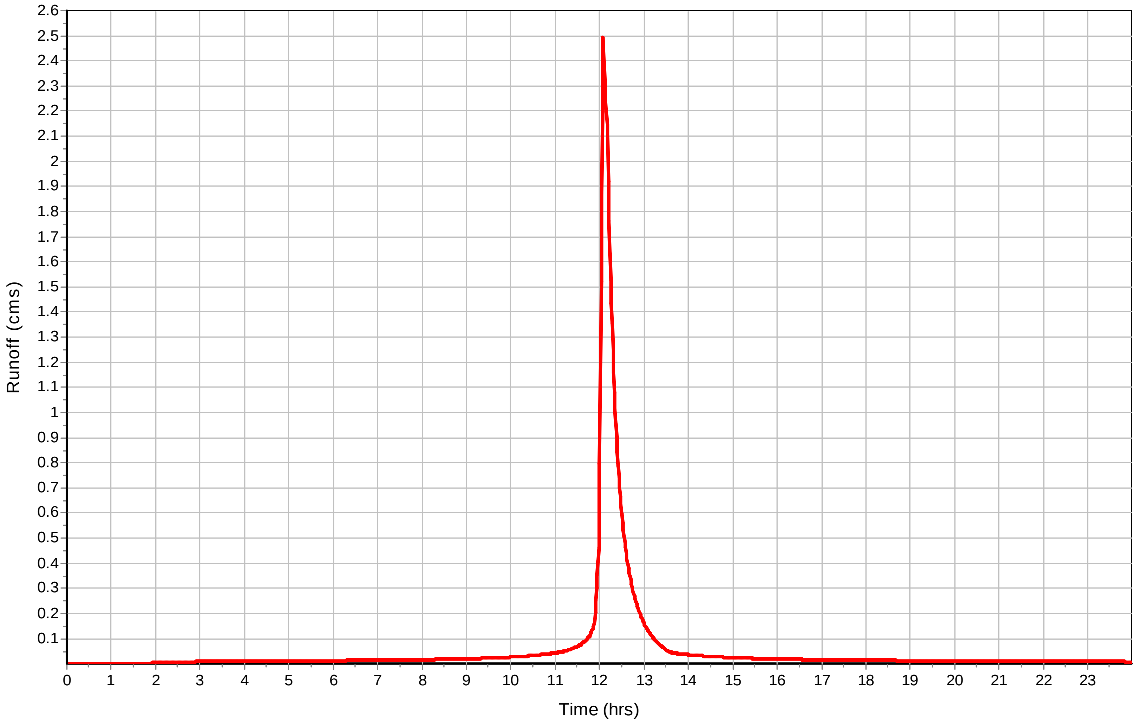
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	63.848
Total Runoff (mm)	57.41
Peak Runoff (cms)	2.49
Time of Concentration (days hh:mm:ss)	0 00:34:34

Subbasin : {PHASE 1}.Catchment : 8

Rainfall Intensity Graph



Runoff Hydrograph



Subbasin : {PHASE 1}.Catchment : 9

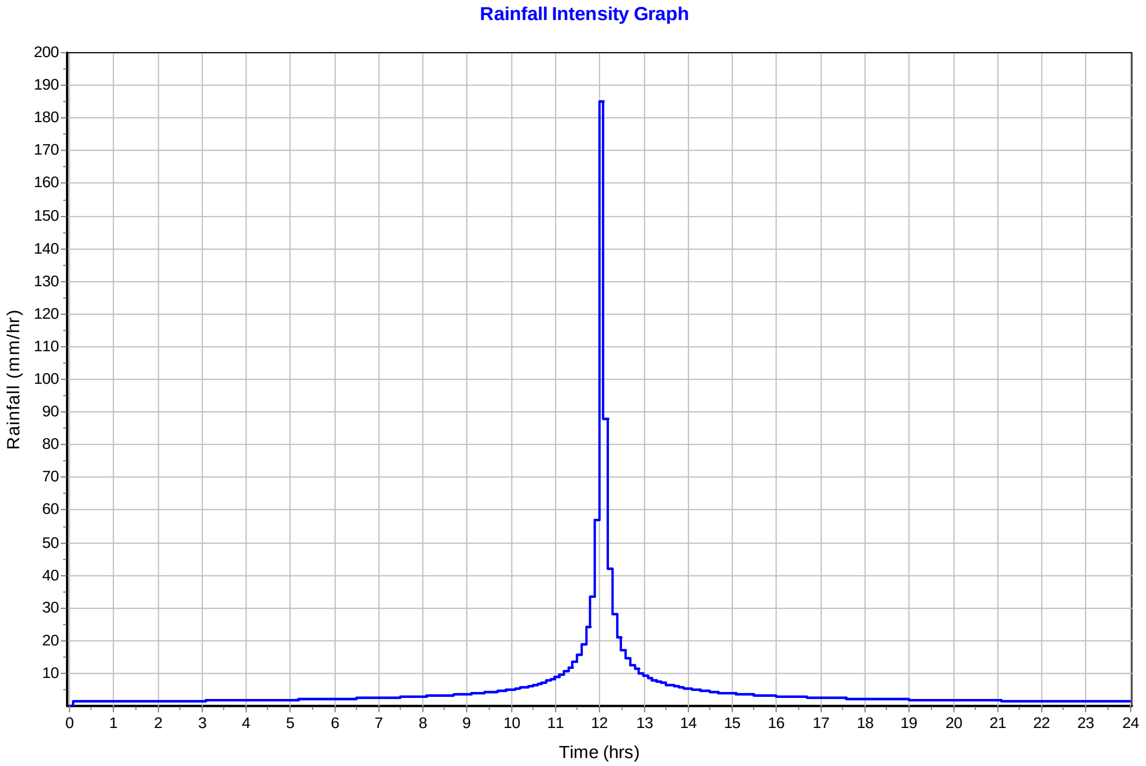
Input Data

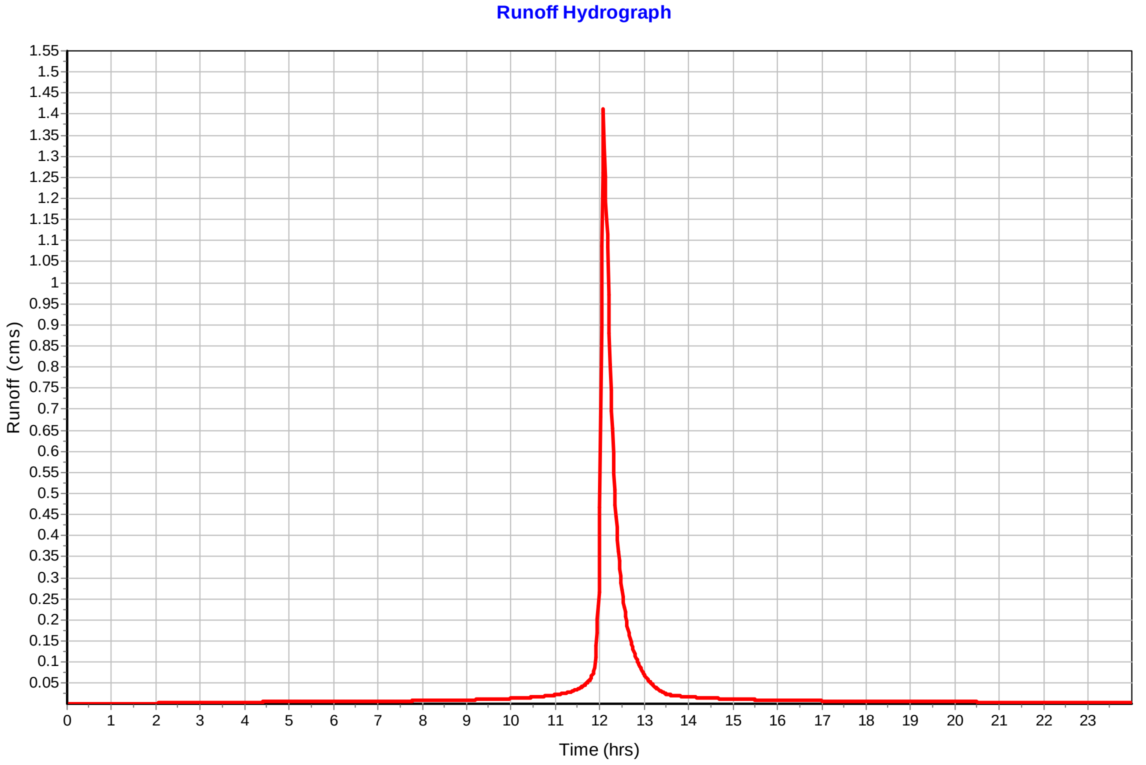
Area (ha)	4.23
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	11.34
Equivalent Width (m)	378.1
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	63.564
Total Runoff (mm)	57.74
Peak Runoff (cms)	1.41
Time of Concentration (days hh:mm:ss)	0 00:29:26

Subbasin : {PHASE 1}.Catchment : 9





Subbasin : Catchment : 16

Input Data

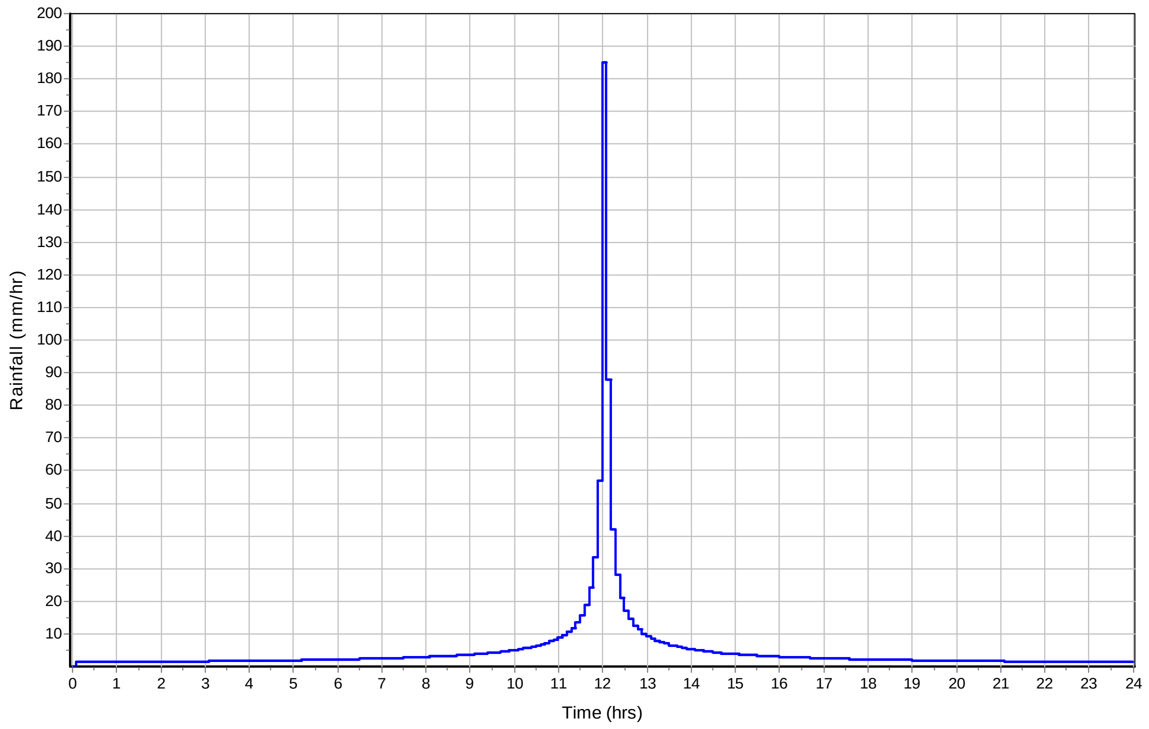
Area (ha)	111.95
Impervious Area (%)	25
Max Infiltration Rate (mm/hr)	60
Min Infiltration Rate (mm/hr)	10
Drying Time (days)	7
Decay Constant (1/hrs)	4
Max Volume (mm)	0
Average Slope (%)	2
Equivalent Width (m)	1741
Impervious Area	
Manning's Roughness	0.3
Pervious Area	
Manning's Roughness	0.13
Curb & Gutter Length (m)	0
Rain Gage ID	Rain Gage-01

Subbasin Runoff Results

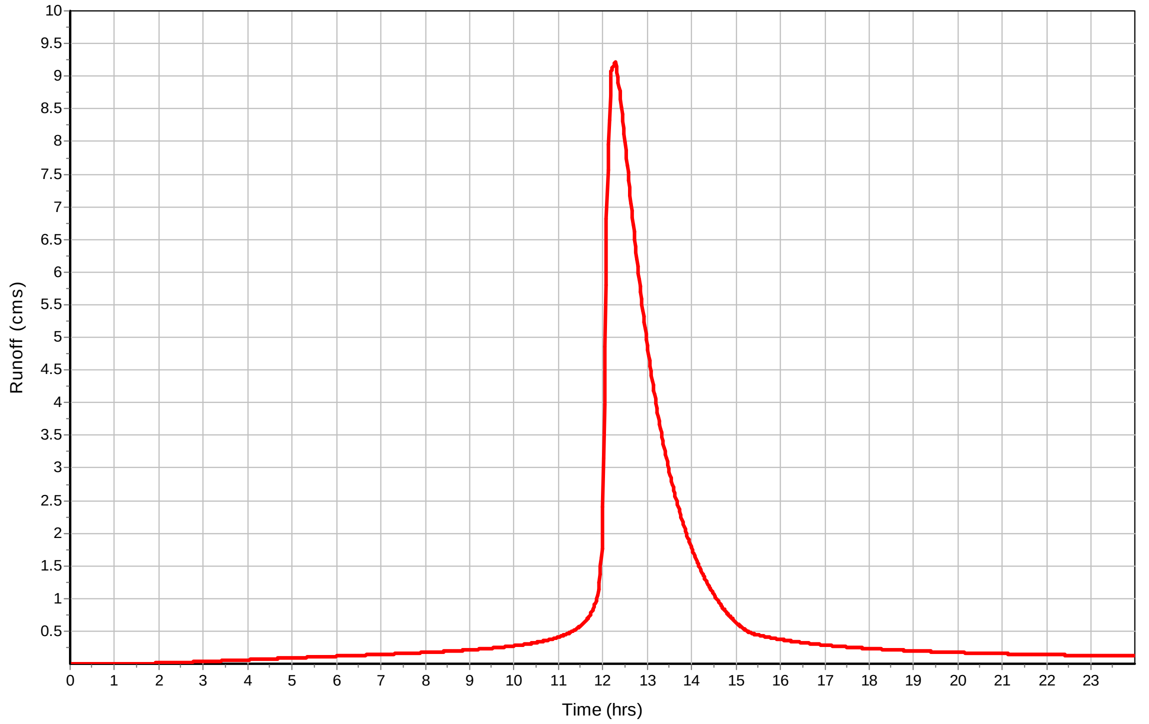
Total Rainfall (mm)	121.89
Total Runon (mm)	0
Total Evaporation (mm)	0
Total Infiltration (mm)	71.002
Total Runoff (mm)	49.29
Peak Runoff (cms)	9.22
Time of Concentration (days hh:mm:ss)	0 02:21:25

Subbasin : Catchment : 16

Rainfall Intensity Graph



Runoff Hydrograph



Junction Input

SN	Element ID	Invert Elevation (m)	Ground/Rim (Max) Elevation (m)	Ground/Rim (Max) Offset (m)	Initial Water Elevation (m)	Initial Water Depth (m)	Surcharge Elevation (m)	Surcharge Depth (m)	Ponded Area (m²)	Minimum Pipe Cover (mm)
1	INLET1	1489.97	1491.77	1.80	1489.97	0.00	1491.77	0.00	0.00	0.00
2	INLET10	1493.15	1495.07	1.92	1493.15	0.00	1495.07	0.00	0.00	0.00
3	INLET11	1494.47	1495.87	1.40	1494.47	0.00	1495.87	0.00	0.00	0.00
4	INLET12	1494.97	1496.37	1.40	1494.97	0.00	1496.37	0.00	0.00	0.00
5	INLET13	1461.94	1463.84	1.90	1461.94	0.00	1463.84	0.00	0.00	0.00
6	INLET14	1460.22	1461.12	0.90	1460.22	0.00	1461.12	0.00	0.00	0.00
7	INLET15	1460.85	1461.85	1.00	1460.85	0.00	1461.85	0.00	0.00	0.00
8	INLET16	1450.97	1452.50	1.54	1450.97	0.00	1452.50	0.00	0.00	0.00
9	INLET17	1457.45	1458.64	1.20	1457.45	0.00	1458.64	0.00	0.00	0.00
10	INLET18	1459.92	1460.92	1.00	1459.92	0.00	1460.92	0.00	0.00	0.00
11	INLET19	1461.98	1463.48	1.50	1461.98	0.00	1463.48	0.00	0.00	0.00
12	INLET2	1490.10	1492.00	1.90	1490.10	0.00	1492.00	0.00	0.00	0.00
13	INLET3	1486.48	1487.48	1.00	1486.48	0.00	1487.48	0.00	0.00	0.00
14	INLET4	1485.79	1487.20	1.41	1485.79	0.00	1487.20	0.00	0.00	0.00
15	INLET5	1471.23	1472.13	0.90	1471.23	0.00	1472.13	0.00	0.00	0.00
16	INLET6	1471.14	1473.62	2.48	1471.14	0.00	1473.62	0.00	0.00	0.00
17	INLET7	1471.14	1472.54	1.40	1471.14	0.00	1472.54	0.00	0.00	0.00
18	INLET8	1471.23	1472.13	0.90	1471.23	0.00	1472.13	0.00	0.00	0.00
19	INLET81	1459.67	1462.27	2.60	1459.67	0.00	1462.27	0.00	0.00	0.00
20	INLET9	1494.91	1495.81	0.90	1494.91	0.00	1495.81	0.00	0.00	0.00
21	MH103	1489.37	1490.37	1.00	1489.37	0.00	1490.37	0.00	0.00	0.00
22	MH104	1485.66	1486.66	1.00	1485.66	0.00	1486.66	0.00	0.00	0.00
23	MH107	1488.49	1489.99	1.50	1488.49	0.00	1489.99	0.00	0.00	0.00
24	MH108	1484.97	1486.47	1.50	1484.97	0.00	1486.47	0.00	0.00	0.00
25	MH110	1488.00	1489.90	1.90	1488.00	0.00	1489.90	0.00	0.00	0.00
26	MH111	1486.15	1488.05	1.90	1486.15	0.00	1488.05	0.00	0.00	0.00
27	MH112	1484.06	1485.96	1.90	1484.06	0.00	1485.96	0.00	0.00	0.00
28	MH113	1482.60	1484.50	1.90	1482.60	0.00	1484.50	0.00	0.00	0.00
29	MH114	1480.92	1482.82	1.90	1480.92	0.00	1482.82	0.00	0.00	0.00
30	MH115	1478.95	1480.85	1.90	1478.95	0.00	1480.85	0.00	0.00	0.00
31	MH116	1476.94	1478.84	1.90	1476.94	0.00	1478.84	0.00	0.00	0.00
32	MH117	1488.09	1489.99	1.90	1488.09	0.00	1489.99	0.00	0.00	0.00
33	MH118	1486.29	1488.17	1.89	1486.29	0.00	1488.17	0.00	0.00	0.00
34	MH119	1484.30	1486.26	1.96	1484.30	0.00	1486.26	0.00	0.00	0.00
35	MH12	1457.86	1459.76	1.90	1457.86	0.00	1459.76	0.00	0.00	0.00
36	MH120	1482.70	1484.60	1.90	1482.70	0.00	1484.60	0.00	0.00	0.00
37	MH121	1480.96	1482.86	1.90	1480.96	0.00	1482.86	0.00	0.00	0.00
38	MH122	1478.99	1480.89	1.90	1478.99	0.00	1480.89	0.00	0.00	0.00
39	MH123	1477.09	1478.98	1.90	1477.09	0.00	1478.98	0.00	0.00	0.00
40	MH125	1477.60	1478.60	1.00	1477.60	0.00	1478.60	0.00	0.00	0.00
41	MH126	1477.05	1479.60	2.56	1477.05	0.00	1479.60	0.00	0.00	0.00
42	MH127	1465.03	1466.53	1.50	1465.03	0.00	1466.53	0.00	0.00	0.00
43	MH128	1466.46	1467.96	1.50	1466.46	0.00	1467.96	0.00	0.00	0.00
44	MH130	1478.49	1480.34	1.85	1478.49	0.00	1480.34	0.00	0.00	0.00
45	MH132	1479.45	1480.45	1.00	1479.45	0.00	1480.45	0.00	0.00	0.00
46	MH133	1473.99	1474.99	1.00	1473.99	0.00	1474.99	0.00	0.00	0.00
47	MH134	1483.50	1484.50	1.00	1483.50	0.00	1484.50	0.00	0.00	0.00
48	MH135	1473.29	1475.19	1.90	1473.29	0.00	1475.19	0.00	0.00	0.00
49	MH136	1482.49	1484.39	1.90	1482.49	0.00	1484.39	0.00	0.00	0.00
50	MH138	1469.99	1471.89	1.90	1469.99	0.00	1471.89	0.00	0.00	0.00
51	MH139	1470.29	1471.32	1.03	1470.29	0.00	1471.32	0.00	0.00	0.00
52	MH140	1470.24	1472.14	1.90	1470.24	0.00	1472.14	0.00	0.00	0.00
53	MH141	1470.40	1471.50	1.10	1470.40	0.00	1471.50	0.00	0.00	0.00
54	MH144	1458.30	1460.20	1.90	1458.30	0.00	1460.20	0.00	0.00	0.00
55	MH145	1461.98	1463.48	1.50	1461.98	0.00	1463.48	0.00	0.00	0.00
56	MH15	1476.23	1478.12	1.90	1476.23	0.00	1478.12	0.00	0.00	0.00
57	MH16	1475.51	1477.41	1.90	1475.51	0.00	1477.41	0.00	0.00	0.00
58	MH17	1469.19	1471.09	1.90	1469.19	0.00	1471.09	0.00	0.00	0.00
59	MH18	1464.35	1466.25	1.90	1464.35	0.00	1466.25	0.00	0.00	0.00
60	MH19	1461.01	1462.89	1.88	1461.01	0.00	1462.89	0.00	0.00	0.00
61	MH2	1476.00	1477.90	1.90	1476.00	0.00	1477.90	0.00	0.00	0.00
62	MH3	1475.29	1477.13	1.84	1475.29	0.00	1477.13	0.00	0.00	0.00
63	MH34	1493.95	1496.01	2.06	1493.95	0.00	1496.01	0.00	0.00	0.00
64	MH35	1493.59	1495.74	2.15	1493.59	0.00	1495.74	0.00	0.00	0.00
65	MH36	1493.18	1495.10	1.92	1493.18	0.00	1495.10	0.00	0.00	0.00
66	MH37	1492.86	1494.36	1.50	1492.86	0.00	1494.36	0.00	0.00	0.00
67	MH38	1484.35	1485.85	1.50	1484.35	0.00	1485.85	0.00	0.00	0.00
68	MH39	1472.82	1474.32	1.50	1472.82	0.00	1474.32	0.00	0.00	0.00
69	MH4	1469.39	1471.29	1.90	1469.39	0.00	1471.29	0.00	0.00	0.00
70	MH40	1469.73	1471.23	1.50	1469.73	0.00	1471.23	0.00	0.00	0.00
71	MH41	1467.78	1469.28	1.50	1467.78	0.00	1469.28	0.00	0.00	0.00
72	MH42	1462.08	1463.58	1.50	1462.08	0.00	1463.58	0.00	0.00	0.00
73	MH43	1461.58	1463.59	2.01	1461.58	0.00	1463.59	0.00	0.00	0.00
74	MH44	1461.39	1463.74	2.35	1461.39	0.00	1463.74	0.00	0.00	0.00

Junction Input

SN	Element ID	Invert Elevation (m)	Ground/Rim (Max) Elevation (m)	Ground/Rim (Max) Offset (m)	Initial Water Elevation (m)	Initial Water Depth (m)	Surcharge Elevation (m)	Surcharge Depth (m)	Ponded Area (m^2)	Minimum Pipe Cover (mm)
75	MH45	1459.86	1461.36	1.50	1459.86	0.00	1461.36	0.00	0.00	0.00
76	MH5	1463.08	1466.28	3.20	1463.08	0.00	1466.28	0.00	0.00	0.00
77	MH51	1494.24	1495.74	1.50	1494.24	0.00	1495.74	0.00	0.00	0.00
78	MH52	1492.96	1494.46	1.50	1492.96	0.00	1494.46	0.00	0.00	0.00
79	MH53	1484.65	1486.15	1.50	1484.65	0.00	1486.15	0.00	0.00	0.00
80	MH54	1479.67	1481.17	1.50	1479.67	0.00	1481.17	0.00	0.00	0.00
81	MH55	1474.35	1475.85	1.50	1474.35	0.00	1475.85	0.00	0.00	0.00
82	MH56	1471.24	1472.74	1.50	1471.24	0.00	1472.74	0.00	0.00	0.00
83	MH57	1469.06	1470.56	1.50	1469.06	0.00	1470.56	0.00	0.00	0.00
84	MH58	1461.92	1470.30	8.38	1461.92	0.00	1470.30	0.00	0.00	0.00
85	MH61	1459.15	1461.00	1.85	1459.15	0.00	1461.00	0.00	0.00	0.00
86	MH62	1455.15	1457.06	1.91	1455.15	0.00	1457.06	0.00	0.00	0.00
87	MH63	1455.10	1456.90	1.80	1455.10	0.00	1456.90	0.00	0.00	0.00
88	MH64	1454.77	1456.35	1.58	1454.77	0.00	1456.35	0.00	0.00	0.00
89	MH66	1450.68	1452.23	1.55	1450.68	0.00	1452.23	0.00	0.00	0.00
90	MH68	1450.29	1452.15	1.86	1450.29	0.00	1452.15	0.00	0.00	0.00
91	MH72	1459.50	1460.50	1.00	1459.50	0.00	1460.50	0.00	0.00	0.00
92	MH75	1453.47	1454.47	1.00	1453.47	0.00	1454.47	0.00	0.00	0.00
93	MH76	1452.53	1453.53	1.00	1452.53	0.00	1453.53	0.00	0.00	0.00
94	MH79	1460.68	1461.85	1.17	1460.68	0.00	1461.85	0.00	0.00	0.00
95	MH8	1459.54	1461.44	1.90	1459.54	0.00	1461.44	0.00	0.00	0.00
96	MH80	1460.34	1461.34	1.00	1460.34	0.00	1461.34	0.00	0.00	0.00
97	MH84	1460.78	1462.67	1.89	1460.78	0.00	1462.67	0.00	0.00	0.00
98	MH85	1456.52	1458.42	1.90	1456.52	0.00	1458.42	0.00	0.00	0.00
99	MH86	1455.79	1457.35	1.57	1455.79	0.00	1457.35	0.00	0.00	0.00

Junction Results

SN	Element ID	Peak Inflow (cms)	Peak Lateral Inflow (cms)	Max HGL Elevation Attained (m)	Max HGL Depth Attained (m)	Max Surcharge Depth Attained (m)	Min Freeboard Attained (m)	Average HGL Elevation Attained (m)	Average HGL Depth Attained (m)	Time of Max HGL Occurrence (days hh:mm)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ha-mm)	Total Time Flooded (min)
1	INLET1	18.21	18.21	1490.50	0.53	0.00	1.27	1490.16	0.19	0 12:12	0 00:00	0.00	0.00
2	INLET10	1.06	1.06	1493.31	0.16	0.00	1.76	1493.16	0.01	0 12:06	0 00:00	0.00	0.00
3	INLET11	7.81	7.81	1494.95	0.48	0.00	0.92	1494.72	0.25	0 12:12	0 00:00	0.00	0.00
4	INLET12	5.67	5.67	1495.44	0.48	0.00	0.92	1495.01	0.05	0 12:12	0 00:00	0.00	0.00
5	INLET13	5.36	5.36	1462.26	0.32	0.00	1.58	1461.97	0.03	0 12:12	0 00:00	0.00	0.00
6	INLET14	1.41	1.41	1460.41	0.19	0.00	0.71	1460.23	0.01	0 12:06	0 00:00	0.00	0.00
7	INLET15	3.37	3.37	1461.34	0.49	0.00	0.51	1460.89	0.04	0 12:06	0 00:00	0.00	0.00
8	INLET16	8.72	1.85	1451.73	0.77	0.00	0.77	1451.04	0.08	0 12:12	0 00:00	0.00	0.00
9	INLET17	1.88	0.49	1457.84	0.39	0.00	0.80	1457.66	0.21	0 12:06	0 00:00	0.00	0.00
10	INLET18	3.98	0.66	1460.30	0.38	0.00	0.62	1459.95	0.03	0 12:06	0 00:00	0.00	0.00
11	INLET19	18.35	18.35	1462.59	0.61	0.00	0.89	1462.59	0.61	0 00:00	0 00:00	0.00	0.00
12	INLET2	6.54	6.54	1490.58	0.48	0.00	1.42	1490.58	0.48	0 00:00	0 00:00	0.00	0.00
13	INLET3	1.69	1.69	1486.69	0.22	0.00	0.78	1486.49	0.02	0 12:06	0 00:00	0.00	0.00
14	INLET4	4.41	4.41	1486.15	0.36	0.00	1.05	1485.83	0.04	0 12:12	0 00:00	0.00	0.00
15	INLET5	0.90	0.90	1471.43	0.20	0.00	0.70	1471.24	0.01	0 12:06	0 00:00	0.00	0.00
16	INLET6	0.00	0.00	1471.14	0.00	0.00	2.48	1471.14	0.00	0 00:00	0 00:00	0.00	0.00
17	INLET7	0.00	0.00	1471.14	0.00	0.00	1.40	1471.14	0.00	0 00:00	0 00:00	0.00	0.00
18	INLET8	1.23	1.23	1471.49	0.26	0.00	0.64	1471.24	0.01	0 12:06	0 00:00	0.00	0.00
19	INLET81	24.66	0.00	1460.96	1.29	0.00	1.31	1460.85	1.18	0 12:14	0 00:00	0.00	0.00
20	INLET9	1.96	1.96	1495.13	0.22	0.00	0.68	1494.93	0.02	0 12:12	0 00:00	0.00	0.00
21	MH103	1.96	0.00	1489.59	0.22	0.00	0.78	1489.39	0.02	0 12:12	0 00:00	0.00	0.00
22	MH104	1.96	0.00	1485.84	0.18	0.00	0.82	1485.67	0.01	0 12:12	0 00:00	0.00	0.00
23	MH107	1.03	0.00	1488.65	0.16	0.00	1.34	1488.50	0.01	0 12:06	0 00:00	0.00	0.00
24	MH108	1.03	0.00	1485.10	0.13	0.00	1.38	1484.98	0.01	0 12:06	0 00:00	0.00	0.00
25	MH110	18.19	0.00	1488.54	0.54	0.00	1.36	1488.20	0.20	0 12:12	0 00:00	0.00	0.00
26	MH111	18.17	0.00	1486.69	0.54	0.00	1.36	1486.35	0.20	0 12:12	0 00:00	0.00	0.00
27	MH112	18.17	0.00	1484.64	0.58	0.00	1.32	1484.27	0.21	0 12:12	0 00:00	0.00	0.00
28	MH113	18.16	0.00	1483.17	0.57	0.00	1.32	1482.81	0.21	0 12:12	0 00:00	0.00	0.00
29	MH114	18.15	0.00	1481.47	0.55	0.00	1.35	1481.13	0.21	0 12:13	0 00:00	0.00	0.00
30	MH115	18.15	0.00	1479.48	0.53	0.00	1.37	1479.14	0.19	0 12:13	0 00:00	0.00	0.00
31	MH116	18.15	0.00	1477.47	0.53	0.00	1.37	1477.14	0.20	0 12:13	0 00:00	0.00	0.00
32	MH117	6.54	0.00	1488.57	0.48	0.00	1.42	1488.57	0.48	0 00:06	0 00:00	0.00	0.00
33	MH118	6.54	0.00	1486.77	0.48	0.00	1.41	1486.77	0.48	0 00:06	0 00:00	0.00	0.00
34	MH119	6.54	0.00	1484.79	0.49	0.00	1.46	1484.79	0.49	0 00:06	0 00:00	0.00	0.00
35	MH12	27.83	0.00	1458.51	0.65	0.00	1.24	1458.22	0.36	0 12:14	0 00:00	0.00	0.00
36	MH120	6.54	0.00	1483.19	0.49	0.00	1.40	1483.19	0.49	0 00:07	0 00:00	0.00	0.00
37	MH121	6.54	0.00	1481.45	0.49	0.00	1.41	1481.45	0.49	0 00:07	0 00:00	0.00	0.00
38	MH122	6.54	0.00	1479.46	0.47	0.00	1.43	1479.46	0.47	0 00:08	0 00:00	0.00	0.00
39	MH123	6.54	0.00	1477.56	0.47	0.00	1.42	1477.56	0.47	0 00:08	0 00:00	0.00	0.00
40	MH125	1.96	0.00	1477.76	0.16	0.00	0.84	1477.62	0.02	0 12:12	0 00:00	0.00	0.00
41	MH126	1.03	0.00	1477.15	0.10	0.00	2.45	1477.05	0.00	0 12:06	0 00:00	0.00	0.00
42	MH127	7.80	0.00	1465.57	0.54	0.00	0.96	1465.31	0.28	0 12:13	0 00:00	0.00	0.00
43	MH128	7.43	2.03	1466.89	0.43	0.00	1.07	1466.50	0.04	0 12:14	0 00:00	0.00	0.00
44	MH130	4.39	0.00	1478.83	0.34	0.00	1.51	1478.53	0.04	0 12:12	0 00:00	0.00	0.00
45	MH132	1.65	0.00	1479.65	0.20	0.00	0.80	1479.46	0.01	0 12:06	0 00:00	0.00	0.00
46	MH133	1.65	0.00	1474.17	0.18	0.00	0.82	1474.00	0.01	0 12:07	0 00:00	0.00	0.00
47	MH134	1.66	0.00	1483.71	0.21	0.00	0.78	1483.51	0.01	0 12:06	0 00:00	0.00	0.00
48	MH135	4.38	0.00	1473.61	0.32	0.00	1.58	1473.33	0.04	0 12:13	0 00:00	0.00	0.00
49	MH136	4.39	0.00	1482.85	0.36	0.00	1.54	1482.54	0.05	0 12:12	0 00:00	0.00	0.00
50	MH138	0.00	0.00	1469.99	0.00	0.00	1.90	1469.99	0.00	0 00:00	0 00:00	0.00	0.00
51	MH139	0.89	0.00	1470.49	0.20	0.00	0.84	1470.30	0.01	0 12:06	0 00:00	0.00	0.00
52	MH140	0.00	0.00	1470.24	0.00	0.00	1.90	1470.24	0.00	0 00:00	0 00:00	0.00	0.00
53	MH141	1.20	0.00	1470.66	0.26	0.00	0.84	1470.41	0.01	0 12:06	0 00:00	0.00	0.00
54	MH144	27.83	0.00	1459.18	0.88	0.00	1.02	1458.83	0.53	0 12:06	0 00:00	0.00	0.00
55	MH145	18.12	0.00	1462.56	0.58	0.00	0.92	1462.20	0.22	0 12:14	0 00:00	0.00	0.00
56	MH15	6.54	0.00	1476.73	0.51	0.00	1.39	1476.73	0.51	0 00:08	0 00:00	0.00	0.00
57	MH16	6.54	0.00	1476.01	0.50	0.00	1.40	1476.01	0.50	0 00:08	0 00:00	0.00	0.00
58	MH17	6.54	0.00	1469.61	0.42	0.00	1.48	1469.61	0.42	0 00:12	0 00:00	0.00	0.00
59	MH18	6.54	0.00	1464.78	0.43	0.00	1.47	1464.78	0.43	0 00:13	0 00:00	0.00	0.00
60	MH19	6.54	0.00	1461.47	0.46	0.00	1.42	1461.47	0.46	0 00:13	0 00:00	0.00	0.00
61	MH2	18.14	0.00	1476.56	0.56	0.00	1.33	1476.21	0.21	0 12:13	0 00:00	0.00	0.00
62	MH3	18.14	0.00	1475.86	0.57	0.00	1.27	1475.50	0.21	0 12:13	0 00:00	0.00	0.00
63	MH34	7.81	0.00	1494.43	0.48	0.00	1.58	1494.20	0.25	0 12:12	0 00:00	0.00	0.00
64	MH35	7.81	0.00	1494.06	0.47	0.00	1.67	1493.83	0.24	0 12:12	0 00:00	0.00	0.00
65	MH36	7.81	0.00	1493.66	0.48	0.00	1.45	1493.43	0.25	0 12:12	0 00:00	0.00	0.00
66	MH37	7.81	0.00	1493.33	0.47	0.00	1.02	1493.10	0.24	0 12:12	0 00:00	0.00	0.00
67	MH38	7.81	0.00	1484.76	0.41	0.00	1.09	1484.56	0.21	0 12:12	0 00:00	0.00	0.00
68	MH39	7.81	0.00	1473.28	0.46	0.00	1.04	1473.06	0.24	0 12:12	0 00:00	0.00	0.00
69	MH4	18.13	0.00	1469.87	0.48	0.00	1.42	1469.56	0.17	0 12:14	0 00:00	0.00	0.00
70	MH40	7.80	0.00	1470.31	0.58	0.00	0.92	1470.03	0.30	0 12:13	0 00:00	0.00	0.00
71	MH41	7.80	0.00	1468.36	0.58	0.00	0.92	1468.08	0.30	0 12:13	0 00:00	0.00	0.00
72	MH42	7.79	0.00	1462.77	0.69	0.00	0.81	1462.44	0.36	0 12:14	0 00:00	0.00	0.00
73	MH43	7.79	0.00	1462.27	0.69	0.00	1.32	1461.94	0.36	0 12:14	0 00:00	0.00	0.00

Junction Results

SN	Element ID	Peak Inflow (cms)	Peak Lateral Inflow (cms)	Max HGL Elevation Attained (m)	Max HGL Depth Attained (m)	Max Surcharge Depth Attained (m)	Min Freeboard Attained (m)	Average HGL Elevation Attained (m)	Average HGL Depth Attained (m)	Time of Max HGL Occurrence (days hh:mm)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ha-mm)	Total Time Flooded (min)
74	MH44	7.79	0.00	1462.09	0.70	0.00	1.66	1461.76	0.37	0 12:15	0 00:00	0.00	0.00
75	MH45	7.79	0.00	1461.36	1.50	0.00	0.00	1461.35	1.49	0 00:06	0 00:00	0.00	0.00
76	MH5	18.13	0.00	1464.80	1.72	0.00	1.48	1464.53	1.45	0 12:14	0 00:00	0.00	0.00
77	MH51	5.66	0.00	1494.72	0.48	0.00	1.02	1494.28	0.04	0 12:12	0 00:00	0.00	0.00
78	MH52	5.66	0.00	1493.35	0.39	0.00	1.10	1493.00	0.04	0 12:12	0 00:00	0.00	0.00
79	MH53	5.65	0.00	1485.04	0.39	0.00	1.12	1484.69	0.04	0 12:12	0 00:00	0.00	0.00
80	MH54	5.65	0.00	1480.08	0.41	0.00	1.09	1479.71	0.04	0 12:12	0 00:00	0.00	0.00
81	MH55	5.64	0.00	1474.81	0.46	0.00	1.04	1474.39	0.04	0 12:13	0 00:00	0.00	0.00
82	MH56	5.63	0.00	1471.83	0.59	0.00	0.91	1471.30	0.06	0 12:13	0 00:00	0.00	0.00
83	MH57	5.61	0.00	1469.65	0.59	0.00	0.91	1469.12	0.06	0 12:14	0 00:00	0.00	0.00
84	MH58	7.43	0.00	1462.47	0.55	0.00	7.83	1461.98	0.06	0 12:14	0 00:00	0.00	0.00
85	MH61	5.35	0.00	1459.47	0.32	0.00	1.53	1459.18	0.03	0 12:12	0 00:00	0.00	0.00
86	MH62	5.35	0.00	1455.77	0.62	0.00	1.29	1455.21	0.06	0 12:12	0 00:00	0.00	0.00
87	MH63	5.35	0.00	1455.72	0.62	0.00	1.18	1455.16	0.06	0 12:12	0 00:00	0.00	0.00
88	MH64	7.43	2.49	1455.48	0.71	0.00	0.87	1454.84	0.07	0 12:12	0 00:00	0.00	0.00
89	MH66	8.71	0.00	1451.45	0.77	0.00	0.78	1450.75	0.07	0 12:13	0 00:00	0.00	0.00
90	MH68	10.27	0.00	1451.41	1.12	0.00	0.73	1450.66	0.37	0 12:12	0 00:00	0.00	0.00
91	MH72	1.41	0.00	1459.70	0.20	0.00	0.80	1459.51	0.01	0 12:06	0 00:00	0.00	0.00
92	MH75	1.84	0.00	1453.71	0.25	0.00	0.76	1453.48	0.02	0 12:07	0 00:00	0.00	0.00
93	MH76	1.84	0.00	1452.77	0.24	0.00	0.76	1452.54	0.01	0 12:07	0 00:00	0.00	0.00
94	MH79	3.36	0.00	1461.16	0.48	0.00	0.68	1460.71	0.03	0 12:06	0 00:00	0.00	0.00
95	MH8	24.66	0.00	1460.15	0.61	0.00	1.29	1459.90	0.36	0 12:14	0 00:00	0.00	0.00
96	MH80	3.34	0.00	1460.80	0.46	0.00	0.53	1460.37	0.03	0 12:06	0 00:00	0.00	0.00
97	MH84	18.35	0.00	1461.39	0.61	0.00	1.28	1461.39	0.61	0 00:04	0 00:00	0.00	0.00
98	MH85	18.35	0.00	1457.22	0.70	0.00	1.21	1457.22	0.70	0 00:09	0 00:00	0.00	0.00
99	MH86	18.35	0.00	1456.48	0.69	0.00	0.87	1456.48	0.69	0 00:09	0 00:00	0.00	0.00

Channel Input

SN	Element ID	Length (m)	Inlet Invert Elevation (m)	Inlet Invert Offset (m)	Outlet Invert Elevation (m)	Outlet Invert Offset (m)	Total Drop (m)	Average Slope (%)	Shape	Height (m)	Width (m)	Manning's Roughness	Entrance Losses	Exit/Bend Losses	Additional Losses	Initial Flow (cms)	Flap Gate
1	C1	90.02	1489.97	0.00	1488.00	0.00	1.97	2.1900	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
2	C10	18.45	1457.86	0.00	1457.41	0.00	0.45	2.4200	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
3	C11	104.03	1490.10	0.00	1488.09	0.00	2.01	1.9300	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
4	C12	90.08	1488.09	0.00	1486.29	0.00	1.80	2.0000	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
5	C13	89.96	1486.29	0.00	1484.30	0.00	1.99	2.2100	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
6	C14	90.01	1484.30	0.00	1482.70	0.00	1.60	1.7800	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
7	C15	90.02	1482.70	0.00	1480.96	0.00	1.74	1.9300	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
8	C16	90.03	1480.96	0.00	1478.99	0.00	1.98	2.1900	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
9	C17	90.02	1478.99	0.00	1477.09	0.00	1.90	2.1100	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
10	C18	42.59	1477.09	0.00	1476.23	0.00	0.86	2.0200	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
11	C19	42.73	1476.23	0.00	1475.51	0.00	0.72	1.6800	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
12	C2	90.03	1488.00	0.00	1486.15	0.00	1.84	2.0500	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
13	C20	199.09	1475.51	0.00	1469.19	0.00	6.32	3.1800	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
14	C21	103.23	1469.19	0.00	1464.35	0.00	4.83	4.6800	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
15	C22	109.86	1464.35	0.00	1461.01	0.00	3.34	3.0400	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
16	C23	25.56	1461.01	0.00	1460.40	0.72	0.62	2.4200	Trapezoidal	0.650	4.600	0.0150	0.7000	0.0000	0.0000	0.00	No
17	C24	133.18	1486.48	0.00	1483.50	0.00	2.98	2.2400	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
18	C25	133.26	1479.45	0.00	1473.99	0.00	5.46	4.1000	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
19	C26	133.37	1473.99	0.00	1466.17	0.00	7.82	5.8600	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
20	C27	133.21	1483.50	0.00	1479.45	0.00	4.05	3.0400	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
21	C28	40.97	1476.00	0.00	1475.29	0.00	0.71	1.7200	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
22	C29	126.28	1485.79	0.00	1482.49	0.00	3.30	2.6100	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
23	C3	90.01	1486.15	0.00	1484.06	0.00	2.09	2.3300	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
24	C30	126.34	1478.49	0.00	1473.29	0.00	5.20	4.1100	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
25	C31	126.45	1473.29	0.00	1465.93	0.00	7.37	5.8300	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
26	C32	126.30	1482.49	0.00	1478.49	0.00	4.00	3.1700	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
27	C33	101.57	1471.23	0.00	1470.29	0.00	0.94	0.9200	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
28	C34	103.23	1470.29	0.00	1466.91	0.00	3.39	3.2800	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
29	C35	101.33	1471.14	0.00	1469.99	0.00	1.15	1.1300	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
30	C36	103.50	1469.99	0.00	1466.40	0.00	3.60	3.4700	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
31	C37	25.84	1494.47	0.00	1493.95	0.00	0.52	2.0300	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
32	C38	17.79	1493.95	0.00	1493.59	0.00	0.36	2.0300	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
33	C39	19.99	1493.59	0.00	1493.18	0.00	0.40	2.0300	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
34	C4	90.01	1484.06	0.00	1482.60	0.00	1.47	1.6300	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
35	C40	196.74	1475.29	0.00	1469.39	0.00	5.91	3.0000	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
36	C41	16.11	1493.18	0.00	1492.86	0.00	0.33	2.0300	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
37	C42	245.93	1492.86	0.00	1484.35	0.00	8.51	3.4600	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
38	C43C	201.41	1484.35	0.00	1472.82	0.00	11.53	5.7200	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
39	C44	132.85	1472.82	0.00	1469.73	0.00	3.09	2.3200	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
40	C45	194.47	1469.73	0.00	1467.78	0.00	1.95	1.0000	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
41	C46	210.89	1467.78	0.00	1465.03	0.00	2.75	1.3000	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
42	C47	226.51	1465.03	0.00	1462.08	0.00	2.95	1.3000	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
43	C48	100.20	1462.08	0.00	1461.58	0.00	0.50	0.5000	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
44	C49	36.85	1461.58	0.00	1461.39	0.00	0.18	0.5000	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
45	C5	90.02	1482.60	0.00	1480.92	0.00	1.67	1.8600	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
46	C50	57.25	1461.39	0.00	1459.86	0.00	1.54	2.6800	Trapezoidal	1.000	6.500	0.0150	0.7000	0.0000	0.0000	0.00	No
47	C51	103.19	1469.39	0.00	1464.38	1.30	5.01	4.8500	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
48	C52	48.68	1494.97	0.00	1494.24	0.00	0.73	1.4900	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
49	C53	41.90	1494.24	0.00	1492.96	0.00	1.28	3.0500	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
50	C54	246.49	1492.96	0.00	1484.65	0.00	8.31	3.3700	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
51	C55	98.30	1484.65	0.00	1479.67	0.00	4.99	5.0700	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
52	C56	200.67	1479.67	0.00	1474.35	0.00	5.32	2.6500	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
53	C57	178.16	1474.35	0.00	1471.24	0.00	3.11	1.7500	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
54	C58	316.16	1471.24	0.00	1469.06	0.00	2.17	0.6900	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
55	C59	106.98	1469.06	0.00	1466.46	0.00	2.61	2.4400	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
56	C6	90.02	1480.92	0.00	1478.95	0.00	1.97	2.1900	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
57	C60	120.62	1466.46	0.00	1461.92	0.00	4.54	3.7600	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
58	C61	308.52	1461.92	0.00	1456.97	0.00	4.95	1.6000	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
59	C62	47.07	1461.94	0.00	1459.15	0.00	2.80	5.9400	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
60	C63	71.54	1459.15	0.00	1455.15	0.00	4.00	5.5900	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
61	C64	10.03	1455.15	0.00	1455.10	0.00	0.05	0.5100	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
62	C65	57.06	1455.10	0.00	1454.77	0.00	0.33	0.5800	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
63	C66	653.71	1454.77	0.00	1450.97	0.00	3.80	0.5800	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
64	C67	49.15	1450.97	0.00	1450.68	0.00	0.29	0.5800	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
65	C68	17.26	1450.68	0.00	1450.58	0.29	0.10	0.5900	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
66	C69	49.10	1450.58	0.29	1450.29	0.00	0.29	0.5800	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
67	C7	90.02	1478.95	0.00	1476.94	0.00	2.00	2.2300	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
68	C70	30.01	1460.22	0.00	1459.50	0.00	0.72	2.4000	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
69	C71	93.68	1459.50	0.00	1457.64	0.20	1.86	1.9800	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
70	C72	217.80	1457.45	0.00	1453.47	0.00	3.98	1.8300	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
71	C73	50.95	1453.47	0.00	1452.53	0.00	0.94	1.8400	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
72	C74	35.51	1460.85	0.00	1460.68	0.00	0.18	0.5000	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
73	C75	58.46	1460.68	0.00	1460.34	0.00	0.34	0.5800	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
74	C76	29.51	1460.38	0.70	1459.54	0.00	0.83	2.8200	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No

Channel Input

SN	Element ID	Length (m)	Inlet Invert Elevation (m)	Inlet Invert Offset (m)	Outlet Invert Elevation (m)	Outlet Invert Offset (m)	Total Drop (m)	Average Slope (%)	Shape	Height (m)	Width (m)	Manning's Roughness	Entrance Losses	Exit/Bend Losses	Additional Losses	Initial Flow (cms)	Flap Gate
75	C77	22.65	1460.34	0.00	1459.92	0.00	0.42	1.8600	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
76	C78	63.64	1459.92	0.00	1458.80	0.50	1.11	1.7500	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
77	C79	88.21	1461.98	0.00	1460.78	0.00	1.20	1.3600	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
78	C8	43.57	1476.94	0.00	1476.00	0.00	0.95	2.1700	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
79	C80	202.25	1460.78	0.00	1456.52	0.00	4.26	2.1000	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
80	C81	85.53	1456.52	0.00	1455.79	0.00	0.74	0.8600	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
81	C82	28.19	1455.79	0.00	1455.41	0.00	0.38	1.3300	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
82	C83	50.78	1459.54	0.00	1458.30	0.00	1.24	2.4400	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
83	C84	135.76	1471.23	0.00	1470.40	0.00	0.83	0.6100	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
84	C85	135.81	1470.40	0.00	1466.49	0.00	3.91	2.8800	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
85	C86	135.88	1471.14	0.00	1470.24	0.00	0.90	0.6600	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
86	C87	135.92	1470.24	0.00	1466.65	0.00	3.59	2.6400	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
87	C88	191.42	1494.91	0.00	1489.37	0.00	5.54	2.8900	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
88	C89	62.21	1489.37	0.00	1485.66	0.00	3.71	5.9700	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
89	C9	18.45	1458.30	0.00	1457.86	0.00	0.45	2.4200	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
90	C90	90.73	1485.66	0.00	1477.60	0.00	8.05	8.8800	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
91	C91	107.52	1477.60	0.00	1465.60	0.00	12.00	11.1600	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
92	C92	186.50	1493.15	0.00	1488.49	0.00	4.66	2.5000	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
93	C93	57.65	1488.49	0.00	1484.97	0.00	3.52	6.1000	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
94	C94	81.07	1484.97	0.00	1477.05	0.00	7.92	9.7800	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
95	C95	81.07	1477.05	0.00	1469.12	0.00	7.92	9.7700	Trapezoidal	1.000	6.000	0.0150	0.7000	0.0000	0.0000	0.00	No
96	C96	57.09	1452.53	0.00	1450.29	0.00	2.24	3.9200	Trapezoidal	0.500	4.000	0.0150	0.7000	0.0000	0.0000	0.00	No
97	C97	70.57	1463.08	0.00	1461.98	0.00	1.10	1.5600	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No
98	C98	62.23	1461.98	0.00	1459.67	0.00	2.31	3.7100	Trapezoidal	1.000	9.000	0.0150	0.7000	0.0000	0.0000	0.00	No

Channel Results

SN	Element ID	Peak Flow (cms)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cms)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (m/sec)	Travel Time (min)	Peak Flow Depth (m)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
1	C1	18.19	0 12:12	56.45	0.32	5.67	0.26	0.53	0.53	0.00		
2	C10	27.83	0 12:14	59.39	0.47	6.73	0.05	0.66	0.66	0.00		
3	C11	6.54	0 00:06	11.44	0.57	4.55	0.38	0.48	0.74	0.00		
4	C12	6.54	0 00:06	11.65	0.56	4.61	0.33	0.48	0.74	0.00		
5	C13	6.54	0 00:07	12.24	0.53	4.78	0.31	0.47	0.72	0.00		
6	C14	6.54	0 00:07	10.99	0.60	4.42	0.34	0.49	0.76	0.00		
7	C15	6.54	0 00:07	11.44	0.57	4.55	0.33	0.48	0.74	0.00		
8	C16	6.54	0 00:08	12.20	0.54	4.77	0.31	0.47	0.72	0.00		
9	C17	6.54	0 00:08	11.97	0.55	4.70	0.32	0.47	0.73	0.00		
10	C18	6.54	0 00:08	11.71	0.56	4.63	0.15	0.48	0.74	0.00		
11	C19	6.54	0 00:08	10.66	0.61	4.33	0.16	0.50	0.77	0.00		
12	C2	18.17	0 12:12	54.59	0.33	5.54	0.27	0.54	0.54	0.00		
13	C20	6.54	0 00:12	14.68	0.45	5.43	0.61	0.42	0.65	0.00		
14	C21	6.54	0 00:13	17.83	0.37	6.23	0.28	0.38	0.59	0.00		
15	C22	6.54	0 00:13	14.36	0.46	5.35	0.34	0.43	0.66	0.00		
16	C23	6.54	0 00:13	12.82	0.51	4.94	0.09	0.46	0.70	0.00		
17	C24	1.66	0 12:06	7.49	0.22	3.17	0.70	0.22	0.43	0.00		
18	C25	1.65	0 12:07	10.14	0.16	3.86	0.58	0.18	0.36	0.00		
19	C26	1.64	0 12:07	12.12	0.14	4.34	0.51	0.16	0.33	0.00		
20	C27	1.65	0 12:06	8.73	0.19	3.51	0.63	0.20	0.40	0.00		
21	C28	18.14	0 12:13	50.05	0.36	5.22	0.13	0.57	0.57	0.00		
22	C29	4.39	0 12:12	8.09	0.54	4.51	0.47	0.36	0.72	0.00		
23	C3	18.17	0 12:12	58.18	0.31	5.78	0.26	0.52	0.52	0.00		
24	C30	4.38	0 12:13	10.15	0.43	5.28	0.40	0.32	0.63	0.00		
25	C31	4.38	0 12:13	12.08	0.36	5.95	0.35	0.29	0.57	0.00		
26	C32	4.39	0 12:12	8.91	0.49	4.82	0.44	0.34	0.68	0.00		
27	C33	0.89	0 12:06	4.81	0.18	1.93	0.88	0.19	0.39	0.00		
28	C34	0.88	0 12:06	9.07	0.10	2.91	0.59	0.13	0.27	0.00		
29	C35	0.00	0 00:00	5.33	0.00	0.00		0.00	0.00	0.00		
30	C36	0.00	0 00:00	9.33	0.00	0.00		0.00	0.00	0.00		
31	C37	7.81	0 12:12	31.91	0.24	4.76	0.09	0.48	0.48	0.00		
32	C38	7.81	0 12:12	31.96	0.24	4.76	0.06	0.48	0.48	0.00		
33	C39	7.81	0 12:12	31.90	0.24	4.76	0.07	0.48	0.48	0.00		
34	C4	18.16	0 12:12	48.69	0.37	5.13	0.29	0.58	0.58	0.00		
35	C40	18.13	0 12:14	66.10	0.27	6.30	0.52	0.48	0.48	0.00		
36	C41	7.81	0 12:12	31.93	0.24	4.76	0.06	0.48	0.48	0.00		
37	C42	7.81	0 12:12	41.68	0.19	5.75	0.71	0.41	0.41	0.00		
38	C43C	7.81	0 12:12	53.62	0.15	6.83	0.49	0.36	0.36	0.00		
39	C44	7.80	0 12:13	34.16	0.23	5.00	0.44	0.46	0.46	0.00		
40	C45	7.80	0 12:13	22.45	0.35	3.72	0.87	0.58	0.58	0.00		
41	C46	7.80	0 12:13	25.59	0.30	4.08	0.86	0.54	0.54	0.00		
42	C47	7.79	0 12:14	25.59	0.30	4.08	0.93	0.54	0.54	0.00		
43	C48	7.79	0 12:14	15.85	0.49	2.90	0.58	0.69	0.69	0.00		
44	C49	7.79	0 12:15	15.83	0.49	2.89	0.21	0.69	0.69	0.00		
45	C5	18.15	0 12:13	52.04	0.35	5.36	0.28	0.55	0.55	0.00		
46	C50	7.79	0 12:15	36.71	0.21	5.24	0.18	0.44	0.44	0.00		
47	C51	18.13	0 12:14	84.06	0.22	7.39	0.23	0.42	0.42	0.00		
48	C52	5.66	0 12:12	23.65	0.24	3.99	0.20	0.48	0.48	0.00		
49	C53	5.66	0 12:12	33.81	0.17	5.14	0.14	0.39	0.40	0.00		
50	C54	5.65	0 12:12	35.53	0.16	5.34	0.77	0.38	0.38	0.00		
51	C55	5.65	0 12:12	43.58	0.13	6.13	0.27	0.34	0.34	0.00		
52	C56	5.64	0 12:13	31.50	0.18	4.89	0.68	0.41	0.41	0.00		
53	C57	5.63	0 12:13	25.57	0.22	4.22	0.70	0.46	0.46	0.00		
54	C58	5.61	0 12:14	16.05	0.35	3.04	1.73	0.59	0.59	0.00		
55	C59	5.61	0 12:14	30.20	0.19	4.73	0.38	0.42	0.42	0.00		
56	C6	18.15	0 12:13	56.48	0.32	5.67	0.26	0.53	0.53	0.00		
57	C60	7.43	0 12:14	37.53	0.20	5.99	0.34	0.43	0.43	0.00		
58	C61	7.43	0 12:15	24.51	0.30	4.44	1.16	0.54	0.54	0.00		
59	C62	5.35	0 12:12	47.17	0.11	6.37	0.12	0.32	0.32	0.00		
60	C63	5.35	0 12:12	45.73	0.12	6.24	0.19	0.32	0.32	0.00		
61	C64	5.35	0 12:12	13.80	0.39	2.67	0.06	0.62	0.62	0.00		
62	C65	5.34	0 12:12	14.76	0.36	2.81	0.34	0.60	0.60	0.00		
63	C66	7.45	0 12:13	14.76	0.50	3.16	3.45	0.71	0.71	0.00		
64	C67	8.72	0 12:13	14.76	0.59	3.21	0.26	0.77	0.77	0.00		
65	C68	8.71	0 12:13	14.80	0.59	3.21	0.09	0.77	0.77	0.00		
66	C69	10.27	0 12:12	14.74	0.70	3.35	0.24	0.84	0.84	0.00		
67	C7	18.15	0 12:13	56.91	0.32	5.70	0.26	0.53	0.53	0.00		
68	C70	1.41	0 12:06	7.75	0.18	3.06	0.16	0.19	0.39	0.00		
69	C71	1.39	0 12:06	7.05	0.20	2.87	0.54	0.20	0.41	0.00		
70	C72	1.85	0 12:07	6.77	0.27	3.07	1.18	0.24	0.49	0.00		
71	C73	1.84	0 12:07	6.79	0.27	3.05	0.28	0.24	0.49	0.00		
72	C74	3.36	0 12:06	3.53	0.95	2.32	0.26	0.48	0.97	0.00		
73	C75	3.34	0 12:06	3.81	0.88	2.45	0.40	0.46	0.93	0.00		

Channel Results

SN	Element ID	Peak Flow (cms)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cms)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (m/sec)	Travel Time (min)	Peak Flow Depth (m)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
74	C76	24.66	0 12:14	64.06	0.39	6.81	0.07	0.59	0.59	0.00		
75	C77	3.34	0 12:06	6.83	0.49	3.69	0.10	0.34	0.68	0.00		
76	C78	3.97	0 12:06	6.62	0.60	3.80	0.28	0.38	0.76	0.00		
77	C79	18.35	0 00:04	44.53	0.41	4.84	0.30	0.61	0.61	0.00		
78	C8	18.14	0 12:13	56.22	0.32	5.65	0.13	0.53	0.53	0.00		
79	C80	18.35	0 00:09	55.34	0.33	5.61	0.60	0.54	0.54	0.00		
80	C81	18.35	0 00:10	35.44	0.52	4.14	0.34	0.69	0.69	0.00		
81	C82	18.35	0 00:10	44.00	0.42	4.80	0.10	0.61	0.61	0.00		
82	C83	24.66	0 12:14	59.64	0.41	6.49	0.13	0.61	0.61	0.00		
83	C84	1.20	0 12:06	3.91	0.31	1.85	1.22	0.26	0.52	0.00		
84	C85	1.19	0 12:07	8.50	0.14	3.08	0.73	0.17	0.33	0.00		
85	C86	0.00	0 00:00	4.08	0.00	0.00		0.00	0.00	0.00		
86	C87	0.00	0 00:00	8.14	0.00	0.00		0.00	0.00	0.00		
87	C88	1.96	0 12:12	8.51	0.23	3.65	0.87	0.22	0.44	0.00		
88	C89	1.96	0 12:12	12.23	0.16	4.62	0.22	0.18	0.36	0.00		
89	C9	27.83	0 12:14	59.39	0.47	6.73	0.05	0.66	0.66	0.00		
90	C90	1.96	0 12:12	14.91	0.13	5.27	0.29	0.16	0.32	0.00		
91	C91	1.95	0 12:12	16.73	0.12	5.69	0.31	0.15	0.30	0.00		
92	C92	1.03	0 12:06	30.59	0.03	2.83	1.10	0.16	0.16	0.00		
93	C93	1.03	0 12:06	47.78	0.02	3.74	0.26	0.12	0.12	0.00		
94	C94	1.03	0 12:06	60.50	0.02	4.36	0.31	0.11	0.11	0.00		
95	C95	1.03	0 12:07	60.50	0.02	4.36	0.31	0.11	0.11	0.00		
96	C96	1.84	0 12:07	9.91	0.19	3.94	0.24	0.19	0.39	0.00		
97	C97	18.12	0 12:14	47.68	0.38	5.05	0.23	0.58	0.58	0.00		
98	C98	18.12	0 12:14	73.47	0.25	6.75	0.15	0.45	0.45	0.00		

Appendix D

CQA Report

Document Title:	**CQA Plan**
Eskom document no.:	xxxx
Contractor document no.:	366-511915 Appendix D
Document type:	Technical Report
Contractor Name:	EPCM Bonisana
Revision no.:	0.1
Prepared by	**KP Matulovich**
Package/System name:	60 Year Ash Disposal Facility
Unit/s no.:	xxxx
Contractor Name:	EPCM
Contractor no.:	xxxxx
Plant Identification codes:	

Rev	Date	Document Status	EPCM Reviewed	EPCM Approved	Client Review/ Approval
A	09-11-2023	Issued for Internal Review		Signature	Signature
B	10-11-2023	Issued for Client Review		Signature	Signature
0	21-02-2024	Issued for Construction		Signature	Signature
0.1	14-10-2024	Issued for Construction		Signature	Signature

TABLE OF CONTENTS

LIST OF ANNEXURES

LIST OF FIGURES

Kusile Power Station 60 Year ADF

SIGNATORY PAGE

	SIGNEE	ID NUMBER	CONTACT DETAILS	SIGNATURE	DATE
CLIENT	**Name:**	**ID:**	**Tel:**		
	Company:		**Email:**		
DESIGN ENGINEER	**Name:**	**ID:**	**Tel:**		
	Company:	**ECSA REG NO:**	**Email:**		
CQA OFFICER / ENGINEER	**Name:**	**ID:**	**Tel:**		
	Company:	**ECSA REG NO:**	**Email:**		

KUSILE POWER STATION ASH DISPOSAL FACILITY AND POLLUTION CONTROL DAMS

CONSTRUCTION QUALITY ASSURANCE PLAN

1. INTRODUCTION

The Ash Disposal Facility (ADF), Pollution Control Dams (PCDs) and Clean Water Dams (CWDs) liner construction quality assurance is vital to the performance of any proposed lining system. In addition to the Design Report this plan further details the construction quality assurance required for the construction of the proposed Ash Disposal Facility (ADF), Pollution Control Dams (PCDs) and Clean Water Dams (CWDs).

The proposed new ADF, PCDs and CWDs will have a single composite lining system (Class C Containment Barrier System), with the lining system comprising a base preparation layer, a geosynthetic clay liner (GCL), a HDPE geomembrane, a protection geotextile, 300mm thick 25-35MPa concrete slab (PCD's and CWD's) and 300mm thick coarse sand (ADF). A subsoil drainage system is located beneath the base preparation layer to monitor for leachate leak detection and any seasonal groundwater drainage.

The detailed requirements for a construction quality assurance plan and specifications for the manufacture, supply and installation and testing of the liner materials, will be provided in the project specifications and is further detailed below.

The purpose of this Construction Quality Assurance (CQA) Plan is to present the principles and practices of construction quality assurance to be implemented during the installation of the above specified liner materials.

Further to the CQA Plan, detailed Method Statements will be prepared, as required, for the various components of the liner system, to be submitted by the installer prior to commencement of the work on site, for approval.

2. PARTIES INVOLVED WITH CONSTRUCTION QUALITY ASSURANCE

The following section provides descriptions of the parties referred to in this CQA Plan including their responsibilities and qualifications. All parties are to complete the attached table in **Annexure E**.

Specific qualified personnel will be chosen once the work has been approved and the schedule is confirmed for the selected CQA project members. (The SANS 10409 standard specification as amended has particular reference.)

2.1 OWNER/OPERATOR

For the purpose of this CQA Plan and the PROJECT Specifications, all references to the "OWNER" shall mean the party identified as such on the title and signatory page of this document and is the license holder.

2.2 PROJECT MANAGER

The Project Manager is the official representative of the OWNER and is responsible for construction activities at the facility, including oversight and construction management. The Project Manager is responsible for coordinating construction and quality assurance activities for the project. The Project Manager shall be responsible for the resolution of all quality assurance issues that arise during the barrier system construction and must be involved in any decisions that may affect future operations of the impoundment.

For the purpose of this CQA Plan and the Project Specifications, all references to the "PROJECT MANAGER" shall mean the party identified as such on the title and signatory page of this document.

2.3 DESIGN ENGINEER

The Design Engineer, also referred to as the “Designer” or “Engineer,” is the individual or firm responsible for the design and preparation of the Construction Drawings and Construction Specifications. The Designer is responsible for approving all design and Construction Specification changes, modifications, or clarifications encountered during construction.

For the purpose of this CQA Plan and the Project Specifications, all references to the “DESIGN ENGINEER” shall mean the party identified as such on the title and signatory page of this document.

2.4 CQA OFFICER(S)

The CQA Engineer and CQA Officer(s) will be responsible for understanding this CQA Plan and shall conduct CQA testing, monitoring, documentation, and reporting as required by this CQA Plan. The CQA Engineer will be the Engineer-of-Record and will stamp the final construction report. The implementation and reporting of this CQA Plan shall be conducted under the direct supervision of an Engineering Council of South Africa registered professional engineer or technologist in the branch of civil engineering.

For the purpose of this CQA Plan and the Project Specifications, all references to the “CQA OFFICER” shall mean the party identified as such on the title and signatory page of this document.

2.5 GEOSYNTHETICS MANUFACTURER

The geosynthetics manufacturer(s), also referred to as the "Manufacturer," is responsible for production of the geosynthetic components outlined in this plan. The Manufacturer may not be aligned with the Geosynthetics Installer as prescribed in the Competition Act,

Act 89 of 1998. Each Manufacturer must pre-qualify that they are able to produce material that meets the requirements of the Project Specifications (listed in **Appendix D** and on the certified drawings).

2.6 GEOSYNTHETIC INSTALLER

The Geosynthetics Installer (Installer), also referred to as the "Geosynthetics Installation Contractor" or the "Installer", is responsible for proper installation of the geosynthetic components in accordance with the Project Drawings and Project Specifications. The Installer may not be aligned with the Manufacturer as prescribed in the Competition Act, Act 89 of 1998.

The Installer must pre-qualify by meeting the requirements outlined in the Project Specifications. The Installer shall provide a qualified Superintendent (formally known as Foreman) who will provide full-time technical guidance to the field crew. The Superintendent will represent the Installer at all site meetings and will act as the spokesman for the Installer on the project. Welding technicians shall be certified as competent by the International Association of Geosynthetic Installers, the TRI (Austin, Texas) or equal approved independent oversight body. The CQA Engineer, through the Project Manager, reserves the right to reject any welding technician whose performance is unsatisfactory.

2.7 EARTHWORKS CONTRACTOR

The Earthworks Contractor, also referred to as the "CONTRACTOR," is responsible for completion of the site work as defined by contract with the OWNER and in accordance with the Project Drawings and Project Specifications including materials provided by the Geosynthetics Manufacturer and work performed by the Geosynthetics Installer, but excluding materials provided by the OWNER.

The Earthworks Contractor will be responsible for retaining a surveyor to set lines and grades required for excavation, construction, and preparation of as-built drawings. Surveying shall be performed under the direction of a registered Surveyor.

2.8 INDEPENDENT CQA LABORATORY

The Independent CQA Laboratory (CQA Lab) is the third-party laboratory responsible for performing the quality assurance soils and/or geosynthetics laboratory testing tasks listed in this plan. The CQA Lab is directed by the CQA Officer and may be part of the CQA Consultant firm or company. The geosynthetics testing laboratory shall be accredited by the Geosynthetics Research Institute Laboratory Accreditation Program (GRI-LAP or similar). The CQA Lab shall not be affiliated with the Earthworks Contractor nor Geosynthetics Installer nor materials suppliers.

3. MEETINGS

Meetings shall be held at inception and during the construction of the project to enhance coordination among the various parties involved. Meetings will include a pre-construction meeting, progress meetings, and resolution meetings if necessary. Pre-Construction Meeting

A pre-construction meeting will be held at the site prior to the start of construction. The Project Manager, Design Engineer, CQA Officer, Installer, contractor, and others designated by the Owner shall attend this meeting. The purpose of this meeting will at a minimum:

i. Define lines of communication, responsibility, and authority
ii. Conduct a site inspection to discuss work areas, work plans, stockpiling, lay- down areas, access roads, haul roads, and related items
iii. Review the project schedule
iv. Review the Project Drawings, CQA Plan, and Project Specifications and take cognizance of the conditions in regulatory authorizations and licenses.
v. Review work area security and safety protocol

This meeting will be documented by the CQA Officer and copies of the meeting minutes will be distributed to all parties in accordance with SANS 10409 and included in the construction completion report submitted to the authorities.

3.1 PROGRESS MEETINGS

Weekly progress meetings will be held. At a minimum, these meetings will be attended by the CQA Officer, Engineer or their designee, the Project Manager, the Installer, and the Contractor. The Project Manager is responsible for organizing and conducting the progress meetings. The purpose of this meeting will be to:

vi. Review the previous weeks accomplishments and activities
vii. Review upcoming scheduled work and project milestones
viii. Discuss any problems or potential construction problems
ix. Review the results and status of CQA field and laboratory testing

This meeting will be documented by the CQA Officer and the minutes transmitted to all in attendance in accordance with SANS 10409.

3.2 RESOLUTION MEETINGS

Special meetings will be held, as needed, to discuss and resolve potential problems or deficiencies. At a minimum, these meetings will be attended by the Project Manager, Design Engineer, CQA Officer, and the Installer and/or Contractor. The meeting will be documented by the CQA Officer in accordance with SANS 10409.

When deficiencies (items that do not meet the project requirements stated in the Project Specifications) are discovered, the CQA Officer shall immediately determine the nature and extent of the problem and notify the Design Engineer and Contractor and vice versa. If unsatisfactory test results identify a deficiency, additional tests will be performed to define the extent of the deficient material or work area.

The Installer or Contractor shall correct the deficiency to the satisfaction of the Design Engineer and CQA Officer. If the remediation of the deficiency involves a design revision, the Project Manager shall also be contacted. Design revisions can only be made by the Design Engineer.

The corrected deficiency shall be re-tested and/or approved before any additional related work is performed by the Installer or Contractor. Retest results shall also be recorded by the CQA Officer and included in the final report documentation.

4. GEOMEMBRANE LINER

4.1 EARTHWORKS

The contractor shall be responsible for preparing and maintaining the base preparation layer and CCL layers (where applicable), in a condition suitable for the installation of the geomembrane liner.

4.1.1 *Construction Monitoring and Testing*

a) The CQA Officer and Design Engineer will give the Project Manager sufficient notice of anticipated completion of the construction components so that related CQA documentation may be reviewed and accepted without delay to the Contractor. Specific CQA observation and/or testing are required for the following:

- Engineered Fill
- Subgrade Preparation including subsurface drainage
- Compacted clay liner (CCL) (where applicable)
- Anchor trench backfill
- Soil protection layer
- Drainage Gravel and LCS Drainage Layer
- Operations soil layer or pioneering waste layer

b) In addition to the above components, the CQA Officer and Design Engineer will observe the construction of the aggregate base surfacing (geomembrane protection layer) and HDPE pipes for compliance with the Project Drawings and Project Specifications.

4.1.2 *Engineered Fill and Anchor Trench Backfill*

a) The CQA Officer shall observe and document the subgrade preparation prior to placement of engineered fill and shall include:

- Monitoring the stripping of vegetated soil, and growth media to be stockpiled, if directed, in the area designated by the Owner.
- Monitoring that appropriate dust control measures are implemented
- Visually inspecting the excavation for moisture seeps, soft or excessively wet areas, and unstable slopes
- Monitoring subgrade preparation and confirming that the surface of the subgrade is free of soft, organic, and otherwise deleterious materials, and that the surface is firm and unyielding and in accordance with Project Specifications (e.g. compaction density or CBR)
- Verify that the subgrade is suitable for supporting any overlying geosynthetic layers as required by the Project Specifications. Borrow materials for engineered fill and anchor trench backfill will be obtained from the excavation area within the cell or the clay stockpile. CQA observation and/or testing is required during construction to verify that the materials and construction are in accordance with the Project Specifications. The tests to be performed, including testing frequency, are shown on Table 1. The testing frequencies specified in Table 1 may be increased when construction conditions warrant additional tests. Additional tests may be recommended by the CQA Officer and approved by the Design Engineer.

b) The layer underlying the HDPE shall be free of any sharp stones greater than 5mm and shall be finished to a level standard such that no gap greater than

20mm, can be measured beneath a 3m straightedge, as directed by the Engineer on site.

Table 1: Engineered fill and anchor trench backfill construction testing

Test Designation	**ASTM Designation**	**Frequency**
Visual-Method Soil Classification	D2488	Continual during excavation and placement of soils
Moisture-Density	D1557	1 per 5,000 m^3 or each material type
Sieve Analysis	D422	1 per 1,500 m^3 or each material type
Atterberg Limits	D4318	1 per 1,500 m^3 or each material type
Nuclear Moisture/Density[1]	D6938	1 Per 500 m^3, one per lift, or one per day – whichever results in a higher number of tests
Moisture Content	D2216	1 per 20 Nuclear moisture tests
Sand	D1556 or D2937	A minimum of 1 Per 20 Nuclear Density Tests

Notes to Table 1:

1. Tests shall be performed on an even grid to provide adequate testing coverage. For large fills in small areas, the testing frequency shall be increased as necessary to ensure testing foreach lift or layer of soil placed.

4.2 SURFACE ACCEPTANCE

Immediately prior to the placement of the geomembrane liner and or geosynthetic clay liner, the surface shall be inspected and cleaned by the contractor. The installer shall provide the contractor with a written acceptance of the surface to be lined. Subsequent changes or repairs to the subgrade and/or the underlying GCL shall remain the responsibility of the contractor.

4.3 ANCHOR TRENCHES

The anchor trenches, if required, shall be excavated by the contractor to line and widths shown on the design drawings, prior to the geomembrane liner placement.

Anchor trenches excavated in clay soils susceptible to desiccation cracking should be excavated only for the length required for that day's geomembrane liner placement.

Corners in the anchor trenches shall be slightly rounded where the geomembrane liner adjoins the trench in order to minimize sharp bends in the geomembrane liner.

4.4 SCOPE

The geomembrane liner to be utilised for the ADF and PCD shall be an approved HDPE geomembrane. The HDPE geomembrane shall be as follows:

- **Primary Liner (PCDs and CWDs)-**
    - 2.0mm thick mono-textured HDPE geomembrane liner (textured side facing down)

- **Primary Liner (ADF)-**
    - 2.0mm thick mono-textured HDPE geomembrane liner (textured side facing down)

The material will not be used unless approved by the Engineer, who will require conformatory test results (including interface shear tests) before acceptance.

4.5 DEFINITIONS

For the purposes of this specification, the following definitions shall apply:

a) **Manufacturing Quality Control (MQC)**: A planned system of inspections that is used to directly monitor and control the manufacture of a material which is factory originated. MQC is normally performed by the manufacturer of geosynthetic materials and is necessary to ensure minimum (or maximum) specified values in the manufactured product. MQC refers to measures taken by the manufacturer to determine compliance with the requirements for materials and workmanship as stated in certification documents and contract specifications. For the purposes of this project, all applicable conditions of ***GRI GM13*** are to be met, unless otherwise specified.

b) **Manufacturing Quality Assurance (MQA)**: A planned system of activities that provides assurance that the materials were constructed as specified in the certification documents and contract specifications. MQA includes manufacturing facility inspections, verifications, audits, and evaluation of the raw materials and geosynthetic products to assess the quality of the manufactured materials. MQA refers to measures taken by the geomembrane lining contractor and/or engineer as applicable to determine if the manufacturer is in compliance with the product certification and contract specifications for a project.

c) **Construction Quality Control (CQC):** A planned system of inspections that is used to directly monitor and control the quality of a construction project. Construction quality control should be performed by the geomembrane lining contractor and is necessary to achieve quality in the constructed or installed system. Construction Quality Control (CQC) refers to measures taken by the installer or contractor to determine compliance with the requirements for materials and workmanship as stated in the drawings and specifications for the project.

d) **Construction Quality Assurance (CQA)**: A planned system of activities that provides the employer, engineer and permitting authorities' assurance that the facility was constructed as specified in the design. Construction Quality Assurance (CQA) includes inspections, verifications, audits and evaluations of materials and workmanship necessary to determine and document the quality of the constructed facility. Construction Quality Assurance (CQA) refers to measures taken by the engineer to assess if the geomembrane contractor is in compliance with the drawings and specifications for the project.

4.6 MATERIALS

The specific material requirements are:

The geomembrane liner shall be:

- **Primary Liner (PCDs and CWDs) –**
    - 2.0mm thick mono-textured HDPE geomembrane liner
- **Primary Liner (ADF) –**
    - 2.0mm thick mono-textured HDPE geomembrane liner

4.6.1 *Manufacture and Manufacturing Quality Control*

a) For the purposes of this Project, all applicable conditions of ***GRI GM13*** are to be met, in addition to the following conditions:

- i. Thickness to be nominal, not -5%, and the lowest individual thickness for any of the ten (10) values is to be -10%, as per ASTM D559.
- ii. Break elongation to be minimum 400%, as per ASTM D6693 Type IV.
- iii. Puncture resistance to be a minimum of 450N for 1.5mm and 600N for 2mm, as per ASTM D4833.
- iv. Standard OIT to be 200 minutes, as per ASTM D3895.
- v. HP OIT to be 600 minutes, as per ASTM D5885.

b) The test methods and frequencies used by the manufacturer for quality control/quality assurance of the above geomembrane prior to delivery shall be in accordance with the specified standard.

c) The manufacturer's geomembrane quality control certifications, including results of quality control testing of the products, must be supplied to the Owner’s Representative/CQA Officer to verify that the materials supplied for the project are in compliance with all products and/or project specifications in this Section. The certification shall be signed by a responsible party employed by the manufacturer, such as the QA/QC Manager, Production Manager, or Technical Services Manager. Certifications shall include lot and roll numbers and corresponding shipping information.

d) The Manufacturer is to provide certification that the geomembrane and welding rod supplied for the project have the same base resin and material properties.

e) If a tenderer wishes to submit any other product, full shear box test results using site appropriate materials must be submitted with the tender.

4.6.2 *Testing*

It is recognized that due to differences in the chemical constituents of materials making up the geomembrane liner, as well as the variations in the manufacturing process, the physical and chemical properties may vary. The tenderer shall thus submit the following data (Table 2) below and appropriate test results as specified to enable evaluations and comparisons of the materials to be made.

Table 2: Geomembrane testing

Property	Test Method	Unit	Testing Frequency
Nominal thickness variation	ASTM D5994 / SANS 1526 :2015	%	1 per 100 000m^2
Asperity Height (if applicable)	D 7466	mm	1 per 100 000m^2
Formulated Density	D 1505 / D792	g/cm³	1 per 100 000m^2
Puncture Resistance	D 4833	N	1 per 100 000m^2
Tear Resistance	D 1004	N	1 per 100 000m^2
Tensile Stress at Yield	D 6693	kN/m	1 per 100 000m^2
Tensile Stress at Break	D 6693	kN/m	1 per 100 000m^2
Elongation at Yield	D 6693	%	1 per 100 000m^2
Elongation at Break	D 6693	%	1 per 100 000m^2
Stress Crack Resistance	D 9357	hr	per GRI GM10
Carbon Black Content	D 4218	%	1 per 100 000m^2
Carbon Black Dispersion	D 5596	%	1 per 100 000m^2
OIT – Standard Pressure	D 3895	min	1 per 100 000m^2
OIT – High Pressure	D 5885	min	1 per 100 000m^2
Oven aging at 85°C –	D 5721, D 3895,	%	1 per 100 000m^2

Standard and High Pressure	D 5885		
UV Resistance High Pressure OIT % retained after 1600 hrs	D 7238, D 5885	%	1 per 100 000m^2

Tenderers should also note that the geomembrane liners selected should be resistant to degradation by sunlight, ultra-violet rays, ozone, airborne pollution, weathering and leachate.

It is expected that the leachate could contain a range of contaminants, including simple organic acids and alcohols, ammonia, humic and fulvic acids as well as inorganic salts high in sodium, calcium, chloride, sulphate, and iron.

4.7 SUBMITTALS

A. Prior to geosynthetic installation, the Design Engineer shall review the Geosynthetic Installer’s Quality Control submittals to confirm that materials meet the Project Specifications. The Design Engineer shall review the following submittals for each geosynthetic material specified for the Project:

1. Geosynthetic material samples, name of Manufacturer, and minimum material specifications which shall include the Manufacturer's minimum physical properties of the material, test methods (SANS and ASTM Standards) used, and factory and site seaming methods.
2. Manufacturer's Quality Control Manual followed during the manufacturing process.
3. The origin (supplier's name and production plant), identification (brand name and number) and material properties of the resin used to manufacture the product.
4. Geosynthetics Installer’s Quality Control Manual, for the installation and testing of the geosynthetic.

5. Resume (Curriculum Vitae) of the Installer Superintendent, Master Seamer, and Seamers to be assigned to this project (geomembrane only).
6. A copy of each of the Quality Control Certificates on each lot of resin issued by the resin Supplier for the specific material for this project. Geomembrane submittals shall include certification of the resin for extrusion welding rod.
7. The result of quality control testing conducted on the resin used in manufacturing the specific material for this project.
8. A listing which correlates the resin to the individual geosynthetic rolls and extruded materials.
9. A copy of the geosynthetic roll Quality Control Certificates which shall be supplied at a minimum frequency of one (1) per roll.
10. The conformance testing frequency shall be at a rate of 1 per 100 000 square metres, or one sample per lot, whichever results in the greater number of conformance tests. Samples shall be taken across the entire width of the roll and shall not include the first and last metre. The samples shall be a minimum of 1 metre wide by the roll width. The CQA Engineer shall mark the machine direction and roll number on the sample, and date the sample was obtained and forward the sample to the geosynthetic laboratory.
11. A panel layout drawing for geomembrane showing the proposed installation layout identifying field seams as well as any variance or additional details which deviate from the Project Drawings.
12. A detailed installation schedule for the project (program of works).
13. Certification that the extrusion welding rod to be used is comprised of the same resin type as the geomembrane to be used (geomembrane only).

B. Additional Submittals (In-Progress and at Completion).

1. Geomembrane installation warranty.
2. Daily written acceptance of subgrade surface.

3. Low temperature seaming procedures, if applicable.
4. Prequalification test seam samples.
5. Field seam non-destructive test results.
6. Field seam destructive test results.
7. Daily field installation reports.
8. Installation record drawing.

Non-compliance with this part of the specification may be considered sufficient grounds to reject the tender.

Samples that do not meeting the specified requirements shall result in the rejection of applicable rolls/panels. As a minimum, rolls/panels produced immediately prior to and immediately after the failed roll/panel shall be tested for the same failed parameter. Testing shall continue until a minimum of three successive rolls/panels on both sides of the original failing roll/panel pass the failed parameter.

4.8 QUALITY CONTROL

A. Manufacturer's Qualifications

The manufacturer of geomembrane of the type specified or similar product shall have at least five years' experience in the manufacture of such geomembrane. In addition, the geomembrane manufacturer shall have manufactured at least 1,000,000 m^2 of the specified type of geomembrane or similar product during the last five years.

B. Installer's Qualifications

1. The Geomembrane Installer shall be the Manufacturer, approved Manufacturer's Installer or a contractor approved by the Owner’s Representative/CQA Officer to install the geomembrane.
2. The Installer shall demonstrate, to the Engineer’s satisfaction, his/her competence in installing the 2mm HDPE geomembrane material.
3. All seaming, patching, other welding operations and testing shall be performed by qualified technicians employed by the Geomembrane Installer.

4.9 GEOMEMBRANE INSTALLATION WARRANTY

The Geomembrane Installer shall guarantee the geomembrane installation against defects in the installation and workmanship for 1 year commencing with the date of final acceptance.

4.10 GEOMEMBRANE PRE-CONSTRUCTION MEETING

A. A Geomembrane Preconstruction Meeting shall be held at the site prior to installation of the geomembrane. At a minimum, the meeting shall be attended by the Geomembrane Installer, Owner, Owner’s Representative/CQA Officer (Engineer and/or CQA Firm) and the Earthworks Contractor.

B. Topics for this meeting shall include:

1. Responsibilities of each party.
2. Lines of authority and communication. Resolution of any project document ambiguity.
3. Methods for documenting, reporting, and distributing documents and reports.
4. Procedures for packaging and storing archive samples.
5. Review of time schedule for all installation and testing.

6. Review of panel layout and numbering systems for panels and seams including details for marking on geomembrane.
7. Procedures and responsibilities for preparation and submission of as-built panel and seam drawings.
8. Temperature and weather limitations. Installation procedures for adverse weather conditions. Defining acceptable subgrade, geomembrane or ambient moisture and temperature conditions for working during liner installation.
9. Subgrade conditions, dewatering responsibilities, and subgrade maintenance plan.
10. Deployment techniques including allowable subgrade for the geomembrane.
11. Plan for controlling expansion/contraction and wrinkling of the geomembrane in accordance with 4.13.2 i.
12. Covering of the geomembrane and cover soil placement.
13. Measurement and payment schedules.
14. Health and safety.

C. The meeting shall be documented by a person designated at the beginning of the meeting and minutes shall be transmitted to all parties as per SANS 10409.

4.11 CONSTRUCTION QUALITY CONTROL - INTRODUCTION

This specification addresses the minimum measures that must be incorporated by the contractor in his Construction Quality Control Programme to ensure the quality of workmanship and the installation integrity of the geomembrane liners.

It is recognised that careful and specific documentation of the installation procedure is required to substantiate this Construction Quality Control Programme. The onus shall be on the contractor to ensure that his Quality Control Co-coordinators carries out this task to the satisfaction of the engineer.

To ensure conformance of the materials, initial testing as specified must be done on the manufactured materials before they are shipped. Once the materials arrive on site, another set of testing is required to confirm that the material tested before shipping is in fact the same materials that have arrived on site.

4.12 MATERIAL DELIVERY

The engineer or his representative should be present, whenever possible, to observe the material delivery and unloading on site. The engineer or his representative is to note any material received in a damaged state and to remove any necessary conformance samples. Upon mobilisation on site, the contractor and CQA Officer shall:

a. verify that the equipment used on site is adequate and does not present a risk of damage to the geomembrane or other materials,
b. mark rolls or portions of rolls which appear damaged,
c. verify that storage of materials ensures adequate protection against dirt, theft, vandalism, passage of vehicles and that the storage area is dry, ventilated and not exposed to direct sunlight,
d. rolls shall be off-loaded using the appropriate equipment and straps,
e. rolls shall not be placed directly on the ground,
f. ensure that rolls are properly labelled, and that labelling corresponds with quality control documentation,
g. ensure that roll numbers, date, roll size and any damage are logged on the material delivery checklist.
h. rolls shall not be stacked more than three (3) high for geomembranes.

4.13 GEOMEMBRANE INSTALLATION

4.13.1 *Subgrade Preparation*

c) The subgrade shall be prepared in accordance with the project specifications. The geomembrane subgrade shall be uniform and free of all sharp or angular objects that may damage the geomembrane prior to installation of the geomembrane.

d) The Geomembrane Installer and Owner’s Representative/CQA Officer shall inspect the surface to be covered with the geomembrane on each day's operations prior to placement of geomembrane to verify suitability.

e) The Geomembrane Installer and Owner’s Representative/CQA Officer shall provide daily written acceptance for the surface to be covered by the geomembrane in that day's operations. The surface shall be maintained in a manner, during geomembrane installation that ensures subgrade suitability.

f) All subgrade damaged by construction equipment and deemed unsuitable for geomembrane deployment shall be repaired prior to placement of the geomembrane. All repairs shall be approved by the Owner’s Representative/CQA Officer and the Geomembrane Installer. The definitions regarding damage, repair and the responsibilities of the contractor and Geomembrane Installer shall be defined in the preconstruction meeting.

4.13.2 *Geomembrane Placement*

No geomembrane shall be deployed until the applicable certifications and quality control certificates listed in this Plan are submitted to and approved by the Owner’s Representative/CQA Officer. Should geomembrane material be deployed prior to approval by the Owner’s Representative/CQA Officer, it will be at the sole risk of the

Geomembrane Installer and/or Contractor. If the material does not meet project specifications it shall be removed from the work area at no cost to the owner.

a) The geomembrane shall be installed to the limits shown on the project drawings and essentially as shown on approved panel layout drawings.

b) No geomembrane material shall be unrolled and deployed if the material temperatures are lower than five (5) degrees Celsius unless otherwise approved by the Owner’s Representative/CQA Officer. The specified minimum temperature for material deployment may be adjusted by the Owner’s Representative/CQA Officer based on recommendations by the manufacturer. Temperature limitations should be defined in the preconstruction meeting. Typically, only the quantity of geomembrane that will be anchored and seamed together in one day should be deployed.

c) No vehicular traffic shall travel on the geomembrane other than an approved low ground pressure All-Terrain Vehicle or equivalent.

d) Sandbags or equivalent ballast shall be used as necessary to temporarily hold the geomembrane material in position under the foreseeable and reasonably expected wind conditions. Sandbag material shall be sufficiently close-knit to prevent soil fines from working through the bags and discharging on the geomembrane. Sand filling to be approved by the Engineer prior to placing.

e) Geomembrane placement shall not be done if moisture prevents proper subgrade preparation, panel placement or panel seaming. Moisture limitations should be defined in the preconstruction meeting.

f) The Contractor shall only lay as much geomembrane as can be seamed by the end of each working day.

g) Damaged panels or portions of the damaged panels, which have been rejected, shall be marked and their removal from the work area recorded.

h) The geomembrane shall not be allowed to "bridge over" voids or low areas in the subgrade. In these areas, the geomembrane shall be installed so as to allow it to rest in intimate contact with the subgrade.

i) Wrinkles caused by panel placement or thermal expansion should be minimised in accordance with Section 4.7 B.11. Wrinkle heights should not exceed 2 times the width of the wrinkle (i.e., H < 2W), as recommended by Toepfer G.W. (2015), *The Complete Field Guide to Ensuring Quality Geosynthetics Installation, Volume 1*. Should the wrinkle exceed this (during the time stipulated for placement of the cover material), remedial measures indicated by the CQA Engineer must be performed prior to placement of the cover material.

j) Considerations on Site Geometry: In general, seams shall be oriented parallel to the line of the maximum slope. In corners and odd shaped geometric locations, the total length of field seams shall be minimised. Seams shall not be located at low points in the subgrade unless geometry requires seaming at such locations and if approved by the Owner's Representative/CQA Officer.

k) Overlapping: The panels shall be overlapped prior to seaming to whatever extent is necessary to affect a good weld and allow for proper testing. In no case shall this overlap be less than 75mm.

l) The method and equipment used to deploy the panels must not damage the geomembrane liner or the supporting subgrade surface.

m) No personnel working on the geomembrane liner will wear shoes that can damage the geomembrane liner or engage in actions which could result in damage to the geomembrane liner.

n) When using welding/seaming equipment, a protection sheet shall be placed on the geomembrane liner and used as a working surface. All tools and equipment shall be placed on this sheet when not in use.

o) Adequate temporary loading and/or anchoring (i.e., sand bags, tyres), which will not damage the geomembrane liner, will be suitably placed to prevent wind lifting up the geomembrane liner.

p) The geomembrane liner will be deployed with enough slack to allow for typical thermal effects. Measures should be taken to prevent and / or accommodate wrinkling of the geomembrane liner resulting from any possible dimensional instability.

q) Any area of a panel seriously damaged (torn, twisted or crimped) will be marked and repaired in accordance with clauses elsewhere in of this specification.

r) The use of steel pegs driven through the geomembrane liner, as a means of securing it in anchor trenches, will not be permitted.
s) Irregular panels shall be cut so as to allow adequate overlaps for seaming.

t) The geomembrane is to be covered as soon as possible after installation, to prevent extended exposure times. Exposure times are to be approved by the Owner’s Representative/CQA Officer at the pre-construction meeting. If exposed geomembrane cannot be permanently covered timeously, it shall be temporarily covered to prevent thermal conductivity impacts.

u) During the geomembrane placement, interface shear testing as per test method ASTM D5321 is to be undertaken, at a frequency (but no less than three (3) No. tests) and as directed by the Engineer on site, to verify/confirm that the minimum design shear strengths are being met.

4.13.3 *Seaming Procedures*

a) No geomembrane material shall be seamed when the ambient temperature is below 5°C or above 45°C; if the temperatures are above 5°C but frost, ice, etc. is visible on the geomembrane then the overlap needs to be visually inspected and approved by the Owner's Representative/CQA Officer prior to the commencement of the pre-qualification test seam.

The following conditions need to be complied with:

i. The Geomembrane Installer shall submit to the Owner's Representative/CQA Officer for approval, detailed procedures for seaming at low temperatures, possibly including the following:

   - Preheating of the geomembrane.
   - The provision of a tent or other device if necessary, to prevent heat losses during seaming and rapid heat losses subsequent to seaming.
   - Number of test welds to determine appropriate seaming parameters.

ii. Upon inspection if it is noted that there is ice, frost or moisture on the geomembrane overlap due to cold conditions, welding must be stopped until the geomembrane has defrosted sufficiently to be dried.

b) No geomembrane material shall be seamed when the sheet temperature is above 75°C as measured by an infrared thermometer or surface thermocouple unless otherwise approved by the Owner's Representative/CQA Officer. This approval will be based on recommendations by the manufacturer and on a field demonstration by the Geomembrane Installer using prequalification test seams to demonstrate that seams comply with the specification.

c) If seaming operations are conducted at night, lighting equipment shall be sufficient to allow the Installer and CQA Officer to adequately and safely perform their duties.

d) Seaming shall primarily be performed using automatic fusion welding equipment and techniques. Extrusion welding shall be used where fusion welding is not possible, such as at pipe penetrations, patches, repairs and short (less than a roll width) runs of seams.

e) Fishmouths or excessive wrinkles at the seam overlaps, shall be minimised and when necessary, cut along the ridge of the wrinkles back into the panel so as to effect a flat overlap. The cut shall be terminated with a keyhole cut (nominal 10 mm diameter hole) so as to minimise crack/tear propagation. The overlay shall subsequently be seamed. The keyhole cut shall be patched with an oval or round patch of the same base geomembrane material extending a minimum of 150 mm beyond the cut in all directions.

f) The panels of the geomembrane liner shall be overlapped by 150mm prior to welding.

g) The seam area must be cleaned prior to seaming to ensure the area is clean and free of moisture, dust, dirt, or debris of any kind.

h) The panels must be adjusted so that seams are aligned with the fewest possible number of wrinkles and "fishmouths".

i) When extrusion welding is required:

- Whenever possible, the sheets shall be bevelled prior to heat tacking commencement.
- The panels of the geomembrane liner shall be overlapped a minimum of 75mm.
- Using a hot air device, the panels of the geomembrane liner to be welded should be temporarily tacked, taking care not to damage the geomembrane liner by overheating.
- The seam area must be cleaned prior to seaming to assure the area is clean and free of moisture, dust, dirt and debris of any kind.
- The seam overlap should be ground prior to welding within one (1) hour of the welding operation in a manner that does not damage the geomembrane liner. Grind marks should be covered with extrudate whenever possible. In all cases, grinding should not extend more than 5mm past the edge of the area covered by the extrudate during welding.
- The extruder should be purged prior to beginning the seam in order to remove all heat degraded extrudate from the barrel and care should be taken not to dispose of hot extrudate on the geomembrane liner.
- The welding rod should be kept clean and dry.

4.13.4 *Pipe and Structure Penetration Sealing System*

a) Provide penetration-sealing system as shown in the Project Drawings.

b) Penetrations shall be constructed from the base geomembrane material, flat stock, prefabricated boots, and accessories as shown on the Project Drawings. The prefabricated or field fabricated assembly shall be field welded to the geomembrane as shown on the Project Drawings so as to prevent leakage. This assembly shall be tested as outlined in section 4.13.5 b). Alternatively, where field non-destructive testing cannot be performed, attachments will be field spark tested by standard leak detectors in accordance with ASTM 6365.

c) Spark testing should be done in areas where both air pressure testing and vacuum testing are not possible.

   i) Equipment for spark testing shall be comprised of, but not limited to: A handheld holiday spark tester and conductive wand that generates a high voltage.
   ii) The testing activities shall be performed by the Geomembrane Installer by placing an electrically conductive tape or wire beneath the seam prior to welding (if necessary). A trial seam containing a non-welded segment shall be subject to a calibration test to ensure that such a defect (non-welded segment) will be identified under the planned machine settings and procedures. Upon completion of the weld, enable the spark tester and hold approximately 25mm above the weld moving slowly over the entire length of the weld in accordance with ASTM 6365. If there is no spark, the weld is considered to be leak free.
   iii) A spark indicates a hole in the seam. The faulty area shall be located, repaired, and retested by the Geomembrane Installer.
   iv) Care should be taken if flammable gases are present in the area and the gases are to be tested.

4.13.5 *Field Quality Control*

The Owner's Representative/CQA Officer shall be notified prior to all prequalification and production welding and testing, or as agreed upon in the preconstruction meeting.

a) Prequalification Test Seams

i. Test seams shall be prepared and tested by the Geomembrane Installer to verify that seaming parameters (speed, temperature and pressure of welding equipment) are adequate.

ii. Test seams shall be made by each welding technician and tested in accordance with ASTM D 4437 at the beginning of each seaming period. Test seaming shall be performed under the same conditions and with the same equipment and operator combination as production seaming. The test seam shall be a minimum of 3.3 m long for fusion welding and 1 m long for extrusion welding with the seam centred lengthwise.

iii. The Installer shall perform pre-weld testing at the beginning of each crew shift and immediately following any work stoppage (e.g., for lunch, weather, etc.) of 30 minutes or more. Seaming operation shall not commence until the CQA Officer has determined that the seaming process meets the Project Specifications.

iv. Three 25 mm wide specimens shall be cut by the Geomembrane Installer from the test seam. These specimens shall be tested by the Geomembrane Installer using a field tensiometer testing both tracks for peel strength and also for shear strength. Each specimen shall fail in the parent material and not in the weld, "Film Tear Bond" (FTB failure). Seam separation equal to or greater than 10% of the track width shall be considered a failing test.

v. The minimum acceptable seam strength values to be obtained for all specimens tested are listed in subsection 4.13.5 c) iv) of this Section. All four specimens shall pass for the test seam to be a passing seam.

vi. If a test seam fails, an additional test seam shall be immediately conducted. If the additional test seam fails, the seaming apparatus shall be rejected and not used for production seaming until the deficiencies are corrected and a successful test seam can be produced.

vii. A 300mm sample from each test seam shall be labelled. The label shall indicate the date, geomembrane temperature, number of the seaming unit, technician performing the test seam and pass or fail description. The sample shall then be given to the Owner's Representative/CQA Officer for archiving.

viii. The CQA Officer shall record the trial seam test results on a trial seam log form.

b) <u>Field Seam Non-destructive Testing</u>

i. All field seams shall be non-destructively tested by the Geomembrane Installer over the full seam length before the seams are covered. Each seam shall be numbered or otherwise designated. The location, date, test unit, name of tester and outcome of all non-destructive testing shall be recorded and submitted to the Owner's Representative/CQA Officer.

ii. Testing should be done as the seaming work progresses, not at the completion of all field seaming, unless agreed to in advance by the Owner's Representative / CQA Officer. All defects found during testing shall be numbered and marked immediately after detection. All defects found should be repaired, re-tested and remarked to indicate acceptable completion of the repair.

iii. Non-destructive testing shall be performed using vacuum box, air pressure or spark testing equipment.

iv. Non-destructive tests shall be performed by experienced technicians familiar with the specified test methods. The Geomembrane Installer shall demonstrate to the Owner's Representative/CQA Officer all test methods to verify the test procedures are valid.

v. Extrusion seams shall be vacuum box tested by the Geomembrane Installer in accordance with ASTM D 4437 and ASTM D 5641 with the following equipment and procedures:

- Equipment for testing extrusion seams shall be comprised of, but not limited to: a vacuum box assembly consisting of a rigid housing, a transparent viewing window, a soft rubber gasket attached to the base, porthole or valve assembly and a vacuum gauge; a vacuum pump assembly equipped with a pressure controller and pipe connections; a rubber pressure/vacuum hose with fittings and connections; a plastic bucket; wide paint brush or mop; and a soapy solution.
- The vacuum pump shall be charged and the tank pressure adjusted to approximately 35 kPa (5 psig).
- The Geomembrane Installer shall create a leak tight seal between the gasket and geomembrane interface by wetting a strip of geomembrane approximately 0.3m by 1.2m (length and width of box) with a soapy solution, placing the box over the wetted area, and then compressing the box against the geomembrane. The Geomembrane Installer shall then close the bleed valve, open the vacuum valve and maintain initial pressure of approximately 35 kPa for approximately 5 seconds. The geomembrane should be continuously examined through the viewing window for the presence of soap bubbles, indicating a leak. If no bubbles appear after 5 seconds, the area shall be considered leak free. The box shall be depressurised and moved over the next adjoining area with an appropriate overlap and the process repeated.
- All areas where soap bubbles appear shall be marked, repaired and then retested.

- At locations where seams cannot be non-destructively tested, such as pipe penetrations, alternate non-destructive spark testing (as outlined in section 4.13.4 b)) or equivalent should be substituted.
- All seams that are vacuum tested shall be marked with the date tested, the name of the technician performing the test and the results of the test.

vi. Double fusion seams with an enclosed channel shall be air pressure tested by the Geomembrane Installer in accordance with ASTM D 5820, ASTM D 4437 and the following equipment and procedures:

- Equipment for testing double fusion seams shall be comprised of, but not limited to: an air pump equipped with a pressure gauge capable of generating and sustaining a pressure of 210 kPa mounted on a cushion to protect the geomembrane; and a manometer equipped with a sharp hollow needle or other approved pressure feed device.
- The Testing activities shall be performed by the Geomembrane Installer. Both ends of the seam to be tested shall be sealed and a needle or other approved pressure feed device shall be inserted into the tunnel created by the double wedge fusion weld. The air pump shall be adjusted to a pressure of 210 kPa, and the valve closed. Allow 2 minutes for the injected air to come to equilibrium in the channel and sustain pressure for 5 minutes. If pressure loss does not exceed 28 kPa after this five-minute period, the seam shall be considered leak tight. Release pressure from the opposite end verifying pressure drop on needle to ensure testing of the entire seam. The needle or other approved pressure feed device shall be removed and the feed hole sealed.

- If loss of pressure exceeds 28 kPa during the testing period or pressure does not stabilise, the faulty area shall be located, repaired and retested by the Geomembrane Installer.
- Results of the pressure testing shall be recorded on the liner at the seam tested and on a pressure testing record.

vii. The CQA Officer shall record all non-destructive test locations on the vacuum test and pressure test log forms.

c) <u>Destructive Field Seam Testing</u>

i. One destructive test sample per 150 linear metre seam length of each seaming apparatus or another predetermined length in accordance with GRI GM 14 shall be taken by the Geomembrane Installer from a location specified by the Owner's Representative/CQA Officer. The Geomembrane Installer shall not be informed in advance of the sample location. In order to obtain test results prior to completion of geomembrane installation, samples shall be cut by the Geomembrane Installer as directed by the Owner's Representative/CQA Officer as seaming progresses.

ii. All field samples shall be marked with their sample number and seam number. The sample number, date, time, location and seam number shall be recorded. The Geomembrane Installer shall repair all holes in the geomembrane resulting from obtaining the seam samples. All patches shall be vacuum box tested or spark tested. If a patch cannot be permanently installed over the test location the same day of sample collection, a temporary patch shall be tack welded or hot air welded over the opening until a permanent patch can be affixed.

iii. The samples shall be taken centred over the seam and prioritized as follows:

- All areas identified as suspect during non-destructive testing/monitoring

- Seams that appear suspect to the CQA Officer
- A minimum of one sample per day
- A minimum of one sample for each geomembrane seaming apparatus
- A minimum of one sample for each representative working conditions (e.g. weather condition)
- A minimum of one sample every 150 metres of seaming for each apparatus

iv. Two types of samples shall be obtained at each location. The first sample shall consist of two specimens, each cut approximately 25 mm wide by 200 mm long, taken 1 m apart. These specimens shall be tested for peel and shear strength in the field by the Installer using a calibrated field tensiometer capable of quantitatively measuring peel and shear strengths. The CQA Officer shall observe all field tests and record the test results.

v. If one or both of the specimens fail, the Installer shall take additional test samples 3m from the point of the failed test in each direction and repeat the field test procedure. If these additional tests fail, then the procedure shall be repeated until the length of the poor-quality seam is established.

vi. If the initial field tests pass, the second type of sample shall be taken between the passing specimens. The second sample type shall be approximately 1 m along and 300 mm across.

The sample shall be divided into three equal sections and distributed and tested as follows:

- One sample - Manufacturer/Installer for their use
- One sample - CQA Officer for destructive testing
- One sample - CQA Officer for site archives

vii. Each sample shall be subject to the following destructive tests at a GRI-LAP accredited CQA geosynthetics laboratory (or similar approved by CQA Officer) or at the CQA Site Office and tested per ASTM D6392 with appropriate calibrated equipment:

- Seam shear strength (five tests)
- Seam peel strength (five tests)

viii. For fusion seams, one peel strength test refers to testing of both sides of the seam. A passing test must have all five passing tests for the shear test and peel test.

ix. Failed destructive tests shall be subject to additional testing until a passing area is found. The Installer shall take another test sample 3 m from the point of the failed test in each direction and repeat the field test procedure. If subsequent tests fail, then the procedure is repeated until the length of the poor-quality seam is established. Once the field tests have passed, a second sample shall be taken between the passing specimens and tested by the Independent CQA Laboratory.

x. Failed seams shall be tracked according to the welding apparatus and the machine operator. All failed seams shall be bounded by locations from which passing Independent CQA Laboratory tests have been taken.

xi. The Installer shall be responsible for patching all areas cut for test samples in accordance with the Project Specifications and the Manufacturer's recommended procedures, and for non-destructive testing (e.g., vacuum box, spark testing etc.) of the patched seams. The CQA Officer shall record all test locations, results, actions taken in conjunction with destructive test failures, and repairs.

xii. The CQA Officer shall observe and document that all repair materials, techniques, and procedures used for repairs are approved in advance and meet the requirements of the Project Specifications.

xiii. The CQA Officer shall verify that all repairs are marked, recorded, repaired, tested, and that wrinkles are addressed, prior to being covered by other materials; and that repairs are performed as specified, including specified type of repair according to type of damage and proper patch size or dimension.

xiv. The CQA Officer shall record defects and repairs on repair log forms.

xv. Daily Field Installation Reports: At the beginning of each day's work, the Installer shall provide the Engineer with daily reports for all work accomplished on the previous workday. Reports shall include the following:

- Total amount and location of geomembrane placed.
- Total length and location of seams completed, name of technicians doing seaming and welding unit numbers.
- Results of prequalification test seams.
- Results of non-destructive testing.
- Results of vacuum testing of repairs.

4.13.6 *Construction Quality Assurance*

The engineer, or his representative, shall have full access to all test results carried out by the contractor. In addition, he shall be entitled to be present whenever such tests are carried out.

Should it be deemed necessary, additional tests may be called for by the engineer and the contractor shall give full co-operation in obtaining samples for such tests.

The CQA Officer shall approve areas of the geomembrane prior to coverage of the geomembrane by other materials. Acceptance of areas shall follow these procedures:

xvi. As-built panel layout survey

xvii. Full documentation of all seams
xviii. Full documentation of non-destructive testing on all seams and repairs
xix. Full documentation of repairs on all defects
xx. Full documentation of passing destructive tests
xxi. A final “walk-over” of the area to observe any subsequent damages or non-addressed items
xxii. All submittals required by this CQA Plan or the Project Specifications

5. GEOSYNTHETIC CLAY LINER (GCL)

5.1 SCOPE

This Plan covers the supply and installation of a High Shear grade geosynthetic clay liner (GCL) in the leachate liner system. The installation of the GCL is to be undertaken by an approved specialist Contractor qualified in such installation and proof of such experience must be submitted with the Tender.

5.2 DEFINITIONS

For the purposes of this specification, the following definitions shall apply:

a) **Manufacturing Quality Control (MQC):** A planned system of inspections that is used to directly monitor and control the manufacture of a material that is factory originated. MQC is normally performed by the manufacturer of geosynthetics materials and is necessary to ensure minimum (or maximum) specified values in the manufactured product. MQC refers to measures taken by the manufacturer to determine compliance with the requirements for materials and workmanship as stated in certification documents and contract specifications. For the purposes of this Project, all applicable conditions of ***GRI GCL3*** must be met.

b) **Manufacturing Quality Assurance (MQA):** A planned system of activities that provides assurance that the materials were constructed as specified in the certification documents and contract specifications. MQA includes manufacturing facility inspections, verifications, audits and evaluation of the raw materials and geosynthetics products to assess the quality of the manufactured materials. MQA refers to measures taken by the GCL installation contractor and/or Engineer as applicable to determine if the manufacturer is in compliance with the product certification and contract specifications for a project.

c) **Construction Quality Control (CQC):** A planned system of inspections that is used to directly monitor and control the quality of a construction project. Construction quality control should be performed by the GCL installation contractor and is necessary to achieve quality in the constructed or installed system. Construction Quality Control (CQC) refers to measures taken by the installer or contractor to determine compliance with the requirements for materials and workmanship as stated in the drawings and specifications for the project.

d) **Construction Quality Assurance (CQA):** A planned system of activities that provides the Employer, Engineer and permitting authorities assurance that the facility was constructed as specified in the design. Construction Quality Assurance (CQA) includes inspections, verifications, audits and evaluations of materials and workmanship necessary to determine and document the quality of the constructed facility. Construction Quality Assurance (CQA) refers to measures taken by the Engineer to assess if the GCL installation contractor is in compliance with the drawings and specifications for the project.

e) **Geosynthetic Clay Liner (GCL):** A factory manufactured hydraulic barrier consisting of Sodium Bentonite clay sandwiched between, supported and encapsulated by two geotextiles, held together by needle punching.

f) **Woven or nonwoven fabrics:** used to contain the Bentonite used in a GCL.

g) **Sodium Bentonite:** The high swelling clay component of GCL's consisting primarily of the mineral Montmorillonite.

h) **Needle punching:** A GCL manufacturing process whereby boards of barbed needles incorporate the staple fibres from a nonwoven geotextile, through a Sodium Bentonite clay layer, into the matrix of a second or more geotextile layers.

5.3 MATERIALS

The GCL must comply with the following specifications:

a) The GCL shall consist of new, first-quality products designed and manufactured specifically for the purpose of this work which shall have been satisfactorily demonstrated by prior testing to be suitable and durable for such purposes.

b) The GCL shall conform to the project specifications and shall include, at a minimum, a layer of bentonite between two geosynthetic layers. The GCL shall be manufactured by mechanically bonding the cover and carrier geotextiles using a needle punching process to enhance frictional and internal shear strength characteristics.

c) In order to maintain these characteristics, no glues, adhesives or other non-mechanical bonding processes shall be used instead of the needle punching process.

d) The needle punched GCL shall be locked to prevent fibre pull out under continuous, long-term strain. The lock process must set the nonwoven fibres where they protrude from the carrier geotextile (woven or nonwoven depending upon product) to secure the reinforcement in place more permanently.

e) No other manufacturing techniques shall be approved unless it can be suitably demonstrated that the GCL exhibits uniform shear strength characteristics across the entire width of the panel. Isolated sewn, stitched or stapled rows do not constitute uniform reinforcement for the purposes of this specification.

g) A minimum overlap of 300mm shall be achieved by the installer. The CQA officer is to inspect all overlaps to ensure to ensure the minimum overlap is achieved.

h) The minimum acceptable dimensions for the GCL panels shall be 4.5 metres wide and 30 metre long. Short rolls (rolls less than 30 metres long) may be supplied, but at a rate not to exceed 5% of the total product area produced for this project.

i) To demonstrate the uniformity of the manufacturing process, no delamination of the geotextile components from the Bentonite core shall occur when samples of the GCL are immersed in tap water at ambient temperature of one hour.

j) The Bentonite should be a natural sodium bentonite rather than calcium bentonite that has been activated to become sodium bentonite. The GCL rolls shall be undamaged and sealed in the original manufacturers packaging.

k) A copy of the geosynthetic roll Quality Control Certificates which shall be supplied at a minimum frequency of one (1) per every five thousand (5 000) square metres of geosynthetic material continuously produced and supplied to the project unless otherwise presented in the Project Specifications.

l) Samples that do not meeting the specified requirements shall result in the rejection of applicable rolls/panels. As a minimum, rolls/panels produced immediately prior to and immediately after the failed roll/panel shall be tested for the same failed parameter.

m) Testing shall continue until a minimum of three successive rolls/panels on both sides of the original failing roll/panel pass the failed parameter.

Table 3: GCL Specification and Testing

STANDARD GRADES		UNIT	MEDIUM SHEAR	HIGH SHEAR	M Q TESTING (m^2)	TEST METHOD
GEOTEXTILE COVER LAYER	PP non-woven white	g/m²	200	200	4 000	ASTM D5261
GEOTEXTILE CARRIER LAYER	PP slit film woven	g/m²	110	110		
	PP non-woven white	g/m²	N/A	200		
	Composite	g/m²	N/A	310		
BENTONITE LAYER	Quality	Montmorillonite content > 75%, Sodium Cation Na^+ > 60%				
(Bentonite mass at 0% moisture content)	Sodium Bentonite Powder	g/m²	3700	3700	2 500	ASTM D5993
	Fluid Loss	ml	≤ 18		Per 25 or 50 tonnes	ASTM D5891
	Swell index	ml/2g	≥ 24			ASTM D5890
GCL MASS PER UNIT AREA		g/m²	4000	4210	2 500	ASTM D5993
BONDING PROCESS		Fully Needle-punched and thermally locked				
GRAB STRENGTH	Machine	N	600	1500	4 000	ASTM D4632
	Across	N	600	1500		
CBR BURST	Strength	N	1400	2500	20 000	ASTM D6241
	Elongation	%	≥ 15	≥ 50		
HYDRAULIC CONDUCTIVITY		m/s	≤ 2.56 x 10^{-11}	≤ 1.92 x 10^{-11}	20 000	ASTM D5887
PEEL STRENGTH		N/m	≥ 360	≥ 600	4000	ASTM D6496

Notes:

1) Minimum Average Roll Values (MARV) are reported unless otherwise stated.
2) A high shear grade GCL is required for this Contract.

Acceptable GCL's for this Contract include any needle punched GCL's that meet all the requirements of this specification.

Before considering an alternative GCL material to that specified in the Contract Documents, the Contractor shall submit with his Tender certified test results and statements of quality from the proposed GCL supplier to the Engineer, indicating without exception that the proposed GCL meets the requirements of the specification.

5.4 QUALITY CONTROL

a) The GCL shall be tested for compliance with conformance with the GRI GCL3 and GRI GCL5 as amended Standard specifications and the Project Specifications by the test methods indicated on the material specification. During production needle punched GCL's shall be continuously inspected for broken needles using an in-line metal detector and broken needles shall be removed. GCL's produced on a line that is not equipped with on-line needle detection facilities will not be considered for acceptance. Candidate GCL materials may be tested and pre-approved at the manufacturing location.

b) As a minimum, the CQA Monitor shall independently confirm the mass per unit area of bentonite at zero percent (0%) gravimetric moisture content and the swell index of the bentonite in the GCL.

c) The GCL manufacturer shall issue Quality Control Certificates to the Project Engineer, CQA Inspector or other designated party for each delivery of material. The certifications shall be signed by the quality control manager of the GCL manufacturer or other responsible party and shall include the following information:

   i) Shipment Packing List. A list indicating the rolls shipped on a particular truckload.
   ii) Bill of Loading. The shipping documents for the truck used for the shipment.
   iii) Letter of Certification. The letter indicating the material is in conformance with the physical properties specified.
   iv) Physical Properties Sheet. The material specification for the GCL supplied in accordance with this specification.

d) Manufacturer Quality Control Submittal. The GCL manufacturer shall issue Quality Control submittals to the Project Engineer, CQA Inspector or other designated party for each lot of material if necessary. The submittals shall include the following information:

   i) Bentonite Manufacturer Certification. Bentonite manufacturer quality documentation for the particular lot of clay used in the production of the rolls delivered.
   ii) Geotextile Manufacturer Certification. Geotextile manufacturer quality control documentation for the particular lots of geotextiles used in the production of the rolls delivered.
   iii) GCL Manufacturer Tracking List. Cross-referencing list delineating the corresponding geotextile and Bentonite lots for the materials used in the production of the rolls delivered.

iv) Manufacturing Quality Control Data. The manufacturing quality control test data indicating the actual test values.

e) Packaging. All GCL rolls shall be packaged in opaque moisture resistant plastic sleeves. The roll cores shall be sufficiently strong to resist collapse during transit and handling.

f) Roll Identification and Labelling. Before shipment, the manufacturer shall label each roll, both on the GCL roll and on the surface of the plastic protective sleeve. Labels shall be resistant to fading and moisture degradation to ensure legibility at the time of the installation. At a minimum, the roll labels shall identify the following:

   i) Product name and grade
   ii) Length and width of roll
   iii) Total weight of roll
   iv) Production lot number and individual roll number

g) Any accessory Bentonite used for sealing seams, penetrations or repairs, shall be high-quality powdered Sodium Bentonite from a recognized producer.

5.5 INSTALLATION

The following operational procedures are as specific as possible while recognising that the specific requirements of the project may necessitate minor modifications. Significant deviations from these procedures shall be pre-approved by the Project Engineer or other designated party.

A copy of the manufacturer's installation guidelines must be supplied to the CQA Officer and Engineer with the GCL product.

In addition to the manufacturer's installation guidelines, cognisance will be taken of the Standard Guide for Installation of Geosynthetic Clay Liners (ASTM D 6102-97).

a) Shipping and Handling Equipment. The party responsible for unloading the GCL shall contact the supplier before shipment to determine the correct unloading methods and equipment if different from the pre-approved and specified methods.

b) GCL's must be supported during handling to ensure worker safety and prevent damage to the product. Under no circumstances should the rolls be dragged, lifted from one end, lifted with only the forks of a lift truck or dropped on to the ground from the delivery vehicle.

c) The QCA Inspector shall verify that proper handling equipment exists which does not pose any danger to installation personnel or risk of damage or deformation to the liner material itself. Suitable handling equipment is described below:

   i) Spreader Bar Assembly. A spreader bar assembly shall include both a core pipe or bar and a spreader bar beam. The core pipe shall be used to uniformly support the roll when inserted through the GCL core, while the spreader bar beam will prevent chains or straps from chafing the roll edges.

   ii) Carpet Spike. A carpet spike is a rigid pipe or rod with one end directly connected to a forklift or other handling equipment, and the other end rounded off to allow easy insertion into roll material cores. If a carpet spike is used, it should be at least 3.0 metres long and inserted to its full length into the roll core to prevent excessive bending of the roll when lifted.

iii) Roller Cradles. Roller cradles consist of two large diameter rollers spaced approximately 75mm apart, which both support the GCL roll and allow it to unroll freely. The use of roller cradles shall be permitted if the rollers support the entire width of the GCL roll.

iv) Straps. Straps may be used to support the ends of spreader bars *but are not recommended as the primary support mechanism.* As straps may damage the GCL where wrapped around the roll and generally do not provide sufficient *uniform* support to prevent roll bending or deformation, great care must be exercised when this option is used.

d) GCL Inspection upon Delivery. Each roll shall be visually inspected when unloaded to determine if any packaging or material has been damaged during transit.

i) Rolls exhibiting damage shall be marked and set aside for close examination during deployment.

ii) Minor rips or tears in the plastic packaging shall be repaired with moisture resistant tape before being placed in storage to prevent moisture damage.

iii) The presence of free-flowing water within any roll packaging shall require that roll to be set aside for further examination to ascertain the extent of any damage.

  iv) GCL rolls delivered to the project site shall be only those indicated on GCL manufacturing quality control certificates.

e) Storage / Stockpiling / Staging. Storage of the GCL rolls shall be the responsibility of the installer or other designated party. All GCL rolls shall be stockpiled and maintained dry in a well-drained flat location area away from high-traffic areas, but sufficiently close to the active work area to minimise handling.

  i) Rolls shall not be stacked on uneven or discontinuous surfaces, in order to prevent bending, deformation, and damage to the GCL or cause difficulty inserting the carpet spike or core pipe.

  ii) GCL's should be stored no higher than four rolls high, or limited to the height at which installation personnel may safely manoeuvre the handling apparatus. Stacks or tiers of rolls should be situated in a manner that prevents sliding or rolling by chocking the bottom layer of the rolls.

  iii) An additional tarpaulin or plastic sheet shall be used over the stacked rolls to provide extra protection for GCL material stored outdoors.

  iv) Bagged Bentonite material shall be stored under cover. Bags shall be stored on pallets or other suitably dry surfaces that will prevent pre-hydration.

f) Manufacturing Quality Assurance Documentation. Third party GCL MQA sampling and testing for compliance with this specification shall be co-ordinated by the third party CQA inspector as necessary to support the manufacturer's MQC data.

g) No horizontal joints shall be allowed on any slope during installation of the GCL.

5.6 SUBGRADE PREPARATION

The surfaces upon which the GCL is to be laid shall be suitable for the placement of GCL material, subject to the specification below.

a) Subgrades. The surface upon which the GCL material will be installed shall be inspected by the CQA inspector and certified by the earthworks Contractor to be in accordance with the requirements of this specification.

  i) The subgrade soil shall be well graded containing less than 20% gravel 50mm in diameter.

  ii) In applications where the GCL is the sole barrier and will be subjected to a hydraulic head that exceeds the confining stress, subgrade surfaces consisting of gravel or granular soils may not be appropriate due to their large void content. For these applications, the top 150mm of the subgrade soil should possess a particle size distribution where at least 80% of the soil is finer than 0.2mm (#60 sieve).

  iii) Site specific compaction requirements should be followed in accordance with the project drawings and specifications. At a minimum, the level of compaction should be such that no rutting is caused by installation equipment or other construction vehicles that traffic the area of deployment.

iv) The surfaces to be lined shall be smooth and free of any debris, vegetation, roots, sticks, sharp rocks, or other deleterious materials larger than 5mm in diameter, as well as free of any voids, large cracks or standing water.

v) Directly before deployment of the GCL, the subgrade shall be final-graded to fill remaining voids or desiccation cracks, and proof-rolled to eliminate sharp irregularities or abrupt elevation changes. The surfaces to be lined shall be maintained in this smooth condition.

vi) On a continuing basis, the project CQA inspector and site supervisor shall certify acceptance of the subgrade before GCL placement.

5.7 PLACEMENT

GCL material shall be placed in general accordance with the procedures specified below, or modified to account for site specific conditions.

a) Panel Placement forms must be submitted to the Engineer and approved before commencement of the installation.

b) GCL Orientation. In the absence of specific guidelines, GCL panels should be placed with the nonwoven side up on slopes to maximise shear strength characteristics.

c) In base or flat areas, the GCL does not require any particular orientation, however, in composite liner applications, intimate contact may be facilitated by placing the woven face of the GCL against the overlying FML.

d) GCL Panel Position. Where possible, all slope panels should be installed running down the slope, while panels installed in flat areas require no particular orientation.

e) Panel Deployment. GCL materials shall be installed in general accordance with the procedures set forth in this section, subject to site specific conditions that would necessitate modifications.

f) Deployment should proceed from the highest elevation to the lowest to facilitate drainage in case of precipitation.

g) The GCL may be deployed on slopes by pulling by hand the material from a suspended roll or securing a roll end into an anchor trench and unrolling each panel by hand while slowly moving backwards. The roll must not be allowed to roll down the slope freely without any form of restraint. All care must be taken not to damage the underlying geosynthetics, where applicable.

h) Deployment on flat areas shall be conducted in the same manner as that for the slopes. However, care should be taken to minimise “dragging” the GCL. Slip-sheet may be used to facilitate positioning of the liner while ensuring the GCL is not damaged by underlying harsh surfaces. All care must also be taken not to damage the underlying geosynthetics, where applicable.

i) Overlaps shall be a minimum of 300mm and be free of wrinkles, folds or “fish-mouths”.

j) The Contractor shall only install as much GCL as can be covered at the end of each working day. Only those GCL panels that can be anchored and covered in the same day shall be unpacked and installed. If exposed GCL cannot be permanently covered before the end of a working day, it shall be temporarily covered with plastic or other waterproof material to prevent hydration. No GCL shall be left exposed overnight. Exposed edges of the GCL shall be covered by temporary water-resistant sheeting until work commences again.

k) Anchoring. All GCL material installed on slopes greater than 7h: 1v shall be anchored to prevent potential GCL panel movement.

l) Standard Anchor. The GCL shall be placed into and across the base of the excavated trench, stopping at the back wall of the excavation.

m) “Run-Out” Anchor. On gentle slopes or locations where it is difficult to create an anchor trench, the GCL may alternatively be anchored by a material run-out past the crest of the slope. The length of the run-out shall be pre-approved by the Project Engineer before the use of this method.

n) Overlap seams shall be a minimum of 300mm on panel edges and 300mm on panel ends.

o) Bentonite paste manufactured in accordance with the GCL supplier’s specification should be placed between panels at a minimum rate of 900 grams per linear metre of seam. Where a product is claimed to be self-sealing along the edges, the manufacturer shall provide proof of this claim. This impregnation is to extend inward 500mm (minimum 300mm) from the edges of the roll, in the long direction.

p) Detailing. Detail work, defined as the sealing of the liner to pipe penetrations, foundation walls, drainage structures, spillways, and other appurtenances shall be performed as recommended by the GCL manufacturer.

q) Cutting of the GCL should be performed using a sharp utility knife. Frequent blade changes are recommended to avoid damage to the geotextile components of the GCL during the cutting process.

r) Although direct vehicular contact with the GCL is to be avoided, lightweight, low ground pressure vehicles (such as light weight loader with rubber wheels) may be used to facilitate the installation of the overlaying HDPE membrane.

s) Care should be taken to minimise “dragging” the GCL. Slip-sheet maybe used to facilitate positioning of the liner while ensuring the GCL is not damaged by underlying harsh surfaces.

t) Do not leave GM/GCL composite liners exposed to the atmosphere for extended periods of time (site and condition specific). Backfilling (as per the design specification) in a timely manner should be adequate to prevent movement of GCL due to shrinkage.

5.8 DAMAGE REPAIR

Before cover material placement, damage to the GCL shall be identified and repaired by the installer. Damage is defined as any rips or tears in the geotextiles, delamination of geotextiles or a displaced panel.

a) Rip and Tear Repair (flat surfaces). Rips or tears may be repaired by completely exposing the affected area, removing all foreign objects or soil, and by then placing a patch cut from unused GCL over the damage (damaged material may be left in place), with a minimum overlap of 300mm on all edges.

b) Accessory Bentonite paste should be placed between the patch edges and the repaired material at a rate of 900 grams per lineal metre of edge spread in a continuous 150mm wide fillet, 5mm thick.

c) Rip and Tear Repair (slopes). Damaged GCL material on slopes shall be repaired by the same procedures above. However, the edges of the patch should also be adhered to the repaired liner with a suitable adhesive to keep the patch in position during backfill or cover operations.

d) Displaced Panels. Displaced panels shall be adjusted to the correct position and orientation. The adjusted panel shall then be inspected for any geotextile damage or Bentonite loss. Damage shall be repaired by the above procedure.

e) Premature Hydration. If the GCL is prematurely hydrated, the installer shall notify the QA/QC technician and Project Engineer for a site-specific determination as to whether the material is acceptable or if alternative measures must be taken to ensure the quality of the design, *dependent upon the degree of damage.*

5.9 WEATHER CONDITIONS FOR INSTALLATION

No GCL shall be installed during any rainfall, including light rainfall (<5mm / hr intensity). Heavy direct raindrop impact should also be avoided. The panels can be covered during rainfall events with a tarpaulin or plastic sheet if there is not enough time to complete the cover placement.

6. GEOTEXTILE (PROTECTION, FILTRATION AND SEPARATION)

6.1 SCOPE

This Plan shall be extended to cover the geotextile components required for the protection layer above the HDPE geomembrane liner and the separation layers above the stone leachate collection system and above the leachate leak detection layer.

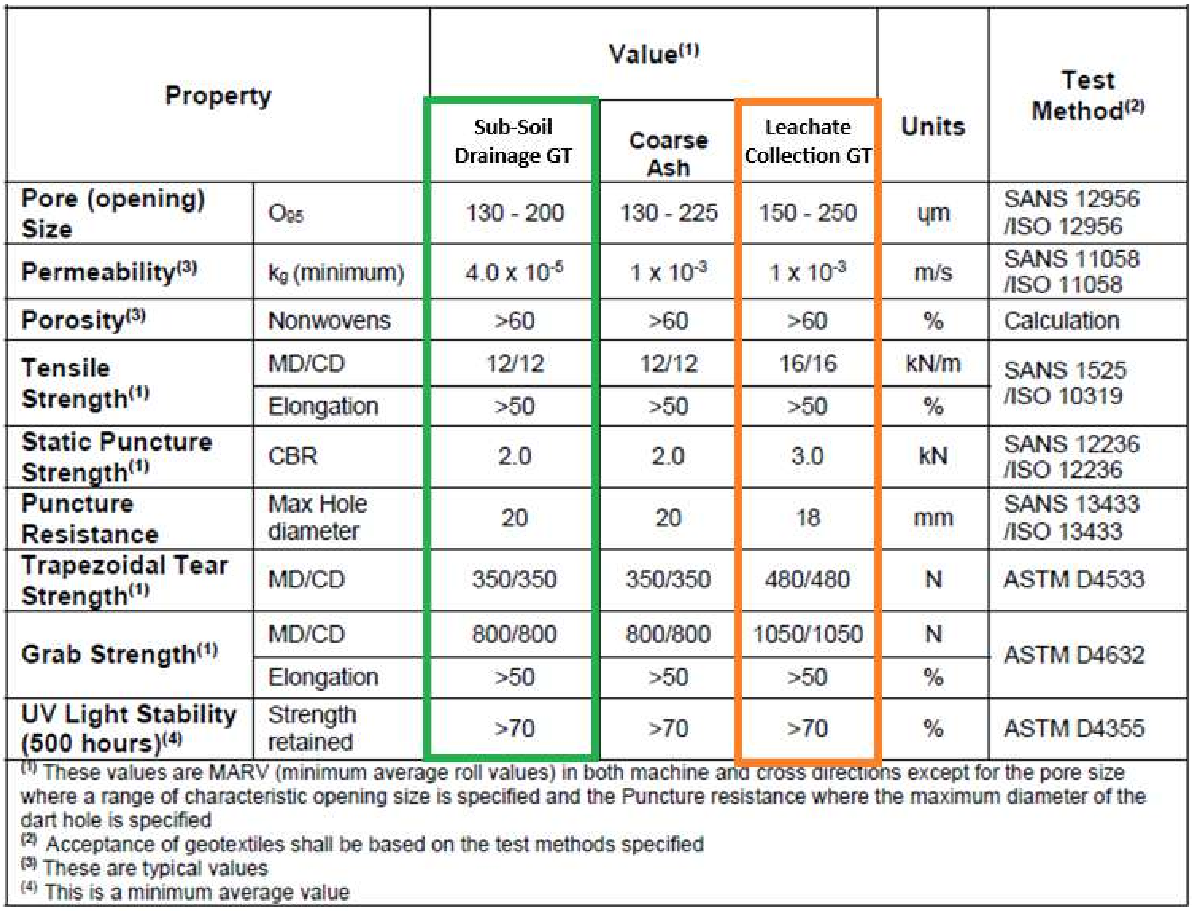
6.2 MATERIAL PROPERTIES AND CONFORMANCE TESTING

6.2.1 *Filtration and Separation Geotextile*

a) **Non-woven needle punched geotextile.**

The geotextile used for the separation layers shall be a non-woven polypropylene or polyester geofabric with a nominal minimum mass of 200g/m^2. All specified non-woven separation geotextiles for the **sub-soil herringbone** drainage system, as well as the **leachate collection** system shall satisfy the below geotextile filter specifications:

Table 4: Properties of 200g/m^2 Geotextile



Property		Value(1)			Units	Test Method(2)
		Sub-Soil Drainage GT	Coarse Ash	Leachate Collection GT		
Pore (opening) Size	O_{95}	130 - 200	130 - 225	150 - 250	ųm	SANS 12956 /ISO 12956
Permeability(3)	k_g (minimum)	4.0 x 10^{-5}	1 x 10^{-3}	1 x 10^{-3}	m/s	SANS 11058 /ISO 11058
Porosity(3)	Nonwovens	>60	>60	>60	%	Calculation
Tensile Strength(1)	MD/CD	12/12	12/12	16/16	kN/m	SANS 1525 /ISO 10319
	Elongation	>50	>50	>50	%	
Static Puncture Strength(1)	CBR	2.0	2.0	3.0	kN	SANS 12236 /ISO 12236
Puncture Resistance	Max Hole diameter	20	20	18	mm	SANS 13433 /ISO 13433
Trapezoidal Tear Strength(1)	MD/CD	350/350	350/350	480/480	N	ASTM D4533
Grab Strength(1)	MD/CD	800/800	800/800	1050/1050	N	ASTM D4632
	Elongation	>50	>50	>50	%	
UV Light Stability (500 hours)(4)	Strength retained	>70	>70	>70	%	ASTM D4355

(1) These values are MARV (minimum average roll values) in both machine and cross directions except for the pore size where a range of characteristic opening size is specified and the Puncture resistance where the maximum diameter of the dart hole is specified

(2) Acceptance of geotextiles shall be based on the test methods specified

(3) These are typical values

(4) This is a minimum average value

Notes:

(a) All values are Minimum Average Roll Value (MARV) unless otherwise indicated.

(b) Evaluation to be on 50 mm strip tensile specimens after 500 hours exposure unless otherwise indicated.

Where products are tested under other test methods, the methods and results should accompany the tender. The geotextile must be stable in the presence of chemicals typically found in a landfill and should be resistant to attack from these chemicals.

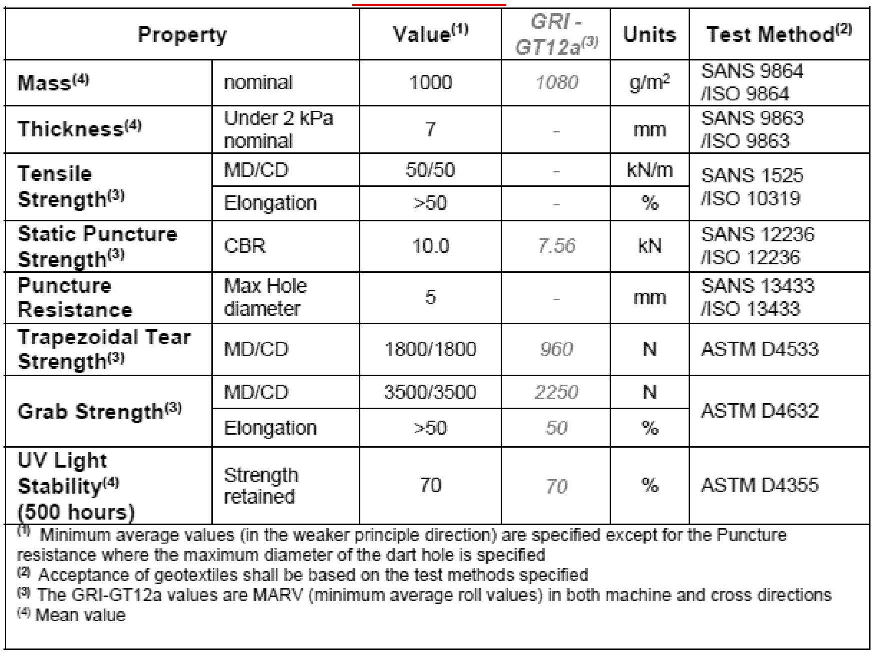
All geotextiles should be stable at a temperature of 100 °C.

6.2.2 *Protection Geotextile*

b) Non-woven needle punched geotextile.

The geotextile used for the protection layer shall be a non-woven polypropylene or polyester geofabric with a minimum nominal mass of 1000g/m^2 and shall have the following properties:

Table 5: Properties of 1000g/m^2 Geotextile



Property		Value[(1)]	*GRI - GT12a[(3)]*	Units	Test Method[(2)]
Mass[(4)]	nominal	1000	*1080*	g/m^2	SANS 9864 /ISO 9864
Thickness[(4)]	Under 2 kPa nominal	7	-	mm	SANS 9863 /ISO 9863
Tensile Strength[(3)]	MD/CD	50/50	-	kN/m	SANS 1525 /ISO 10319
	Elongation	>50	-	%	
Static Puncture Strength[(3)]	CBR	10.0	*7.56*	kN	SANS 12236 /ISO 12236
Puncture Resistance	Max Hole diameter	5	-	mm	SANS 13433 /ISO 13433
Trapezoidal Tear Strength[(3)]	MD/CD	1800/1800	*960*	N	ASTM D4533
Grab Strength[(3)]	MD/CD	3500/3500	*2250*	N	ASTM D4632
	Elongation	>50	*50*	%	
UV Light Stability[(4)] (500 hours)	Strength retained	70	*70*	%	ASTM D4355

(1) Minimum average values (in the weaker principle direction) are specified except for the Puncture resistance where the maximum diameter of the dart hole is specified
(2) Acceptance of geotextiles shall be based on the test methods specified
(3) The GRI-GT12a values are MARV (minimum average roll values) in both machine and cross directions
(4) Mean value

Notes:

(a) All values are Minimum Average Roll Value (MARV) unless otherwise indicated.

(b) Evaluation to be on 50 mm strip tensile specimens after 500 hours exposure unless otherwise indicated.

Where products are tested under other test methods, the methods and results should accompany the tender.

The geotextile must be stable in the presence of chemicals typically found in a landfill and should be resistant to attack from these chemicals.

All geotextiles should be stable at a temperature of 100 °C.

6.2.3 *Conformance Testing*

The geotextile manufacturer shall provide production test certificates for rolls delivered to site demonstrating that the test values specified for the proposed product have been attained. Test methods employed shall be in accordance with those stated below unless otherwise agreed by the CQA Engineer. Certificates relevant to a batch of geotextile shall be furnished to the CQA Engineer prior to that batch of geotextile being incorporated in the works.

Table 6: Geotextile Testing

Property	Test Method SANS / ASTM	Unit	Testing Frequency
Thickness (at 2kPa)	SANS 9863:13	mm	1 per 100 000m^2
Mass per	SANS 10221:07	g/m^2	1 per 100 000m^2

Tensile Strength – 200mm wide strip (weaker direction)	SANS 1525:13	kN/m	1 per 100 000m^2
CBR Puncture Strength	SANS 12236:13	kN	1 per 100 000m^2
Puncture Resistance Diameter of hole (max)	SANS 13433:13	mm	1 per 100 000m^2

The Contractor shall also conduct conformance testing at the frequencies detailed in Table 6 above or one sample per lot, whichever results in the greater number of conformance tests. Samples shall be 1.0m wide by the width of the roll and shall not include the first metre.

Samples shall be split into three: 1 No. for the CQA Engineer, 1 No. for the Contractor and 1 No. for conformance testing. The Contractor shall supply prior to commencement of the works a statement of which geosynthetic laboratory he proposes to use.

All of the parameters listed in Table 6 above will be tested to ensure the material is in accordance with the quoted Test Values. The Contractor shall supply to the CQA Engineer a copy of the laboratory test results immediately on receipt. If testing shows that the geotextile is not in accordance with any one of the quoted Test Values, then this may be cause for rejection of the material from the works.

The CQA Engineer shall mark the machine direction and roll number on the sample, and date the sample was obtained and forward the sample to the geosynthetic laboratory.

6.3 PLANT

The Contractor will be required to provide plant specific to the geotextiles and geosynthetics used to prevent damage and/or reduction of the geotextiles and geosynthetics properties specified.

Due to the nature of the geosynthetics, "Bobcat" compact track like plant will need to be used to prevent damage of the geosynthetics during the installation thereof and the construction and installation of the layers above. The Contractor is to ensure that the plant utilized for the spreading of aggregate is not to exceed 5tonnes operating weight and is to be track mount plant. The proposed plant is to be submitted to the engineer prior to placement of aggregate for written approval.

6.4 CONSTRUCTION

6.4.1 *Handling and Placement*

The installer's personnel shall handle the geotextiles in such a manner as to minimize damage and shall comply with the following:

a) A copy of the manufacturer's installation guidelines must be supplied to the CQA Officer and Engineer with the geotextile.

b) The geotextile shall be delivered to site in rolls covered with an opaque plastic sheet to prevent damage from sunlight and should be stored as per the supplier's specification.

c) Panel Placement forms must be submitted to the Engineer and approved before commencement of the installation.

d) The method of installation shall ensure that the geotextile is in continuous contact with the surface subgrade. The geotextile shall not be stretched or bridged over hollows or humps.

e) On slopes, the geotextile shall be securely anchored and then rolled down the slope in such a manner as to keep the geotextile panel in tension.

f) No horizontal joints shall be allowed on any slope during installation of the geotextile.

g) The geotextile shall be held in place with sandbags to prevent wind uplift. Sandbags shall be installed during the placement and shall remain until replaced with the overlying layer/s. Sandbags shall be filled with fine grained material and must be handled with care to prevent rupture.

h) Geotextiles shall be kept continually under tension to minimize to presence of wrinkles in the geotextile.

i) Care should be taken not to drag the geotextile on the HDPE geomembrane and the leachate drainage layer, as this could damage the material.

j) Where the geotextile is being placed onto the geomembrane and underlying geosynthetics, it shall be deployed by hand so as not to damage the geomembrane and geosynthetics in any way. Special care shall be taken by the Installer to prevent damage of the geomembrane and underlying geosynthetics.

k) Geotextiles shall be cut using an approved geotextile cutter only (i.e., upward cutting hook blade).

l) After installation, the entire surface of the geotextile shall be examined, and harmful foreign objects, shall be removed.

m) A minimum thickness of 300mm of cover shall be kept between heavy equipment and the geotextile at all times.

n) No construction traffic shall be allowed directly on any of the laid geotextile.

o) All laid and approved geotextile is to be covered within fifteen (15) days to prevent damage due to UV exposure.

6.4.2 *Seams and Overlap*

a) Overlap widths are site specific and generally at the discretion of the CQA engineer.

b) On side slopes, the geotextile shall be securely anchored in the anchor trench, then unroll to prevent wrinkles and folds. End of roll overlaps shall be lapped upslope over down slope, and the overlap shall be a minimum of 300mm. Adjacent end of roll overlaps (down slope) should be offset by a minimum of 200mm.

c) All rolls (placed alongside one another or end-on-end) shall overlap by a minimum of 300mm or be sewn with a polyester thread or shall be heat bonded along overlapping edges, or all three methods, as per the supplier’s specification.

d) For curves, the geotextile should be folded or cut and overlapped in the direction of the turn (previous geotextile on top).

6.4.3 *Repairs*

a) Holes in the geotextile shall be patched with geotextile of the same unit weight and material.

b) Sufficient overlap shall be provided to ensure that a suitable thermal seam can be produced, that will not come apart and, when used as a filter, will contain soil.

c) Patches shall be placed over the damaged area and extend 200mm beyond the perimeter of the damaged area and be thermally bonded.

d) Care shall be taken to remove any soil or other material which may have penetrated the torn geotextile.

The Contractor shall submit a summary of the manufacturer's qualifications and a copy of the manufacturer's quality control manual together with the Tender Document. The geotextile manufacturer shall provide a qualified and experienced representative to be available on an as needed basis during construction. The representative shall visit the site for consultation at least twice during construction, or as requested by the Contractor.

One properly identified 600 by 600 mm minimum size geotextile sample is to be submitted at the beginning of the Contract. The geotextile sample is intended for visual demonstration prior to product delivery.

7. SAND BALLAST LAYER / COARSE ASH (FOR ASH DISPOSAL FACILITY)

7.1 SCOPE

This plan shall be extended to cover the sand ballast layer on the base and side slopes of the ADF, above the geomembrane and protection geotextile.

7.2 MATERIALS

Sand Ballast Layer

This layer shall be clean river sand to the following specification:

a)	**Maximum Particle Size**	3.0mm
b)	**Minimum Particle Size**	0.5mm

c) The sand must be washed to remove the fine particles.

Mixed Ash Ballast Layer

The placement of this layer (in place of the washed sand ballast layer) shall only be done once the ADF is able to receive contaminated ash and all supporting infrastructure (PCDs and Pump Stations) are commisioned.

This layer shall be mixed ash from Kusile Power Station to the following specification (in accordance with the approved ash layer for the existing 10 year co-disposal extension):

a) **Average Permeability** $1x10^{-5}$cm/s

b) Grading as per table below:

Sieve Size (mm)	Mixed Ash Sample Averages
53.0	**100**
37.5	**100**
26.5	**100**
19.0	**100**
13.2	**100**
9.5	**100**
6.7	**100**
4.75	**100**
2.0	**99**
1.0	**98**
0.425	**95**
0.250	**93**
0.150	**85**
0.075	**69**
0.060	**62**
0.050	**57**
0.035	**47**
0.020	**33**
0.006	**14**
0.002	**7**
GM	**0.36**

Atterberg Limits

LL (%)	**-**
PI (%)	**NP**
LS (%)	**0.0**

7.3 PROCESS CONTROL TESTING

Both pre-construction and construction testing are required for these materials.

Pre-construction testing consists of testing proposed materials from samples obtained at the aggregate or on-site borrow source.

Construction testing consists of testing performed from samples obtained during delivery of materials during the module or layer construction.

The tests to be performed, including testing frequency, for each material type are presented in Table 7 The testing frequencies specified in Table 7 may be increased when construction conditions warrant additional tests. Additional testing may be performed on suspect materials as recommended by the Design Engineer.

Table 7: Ballast layer construction testing

Test Designation	ASTM	Designation Frequency
Sieve Analysis	D422	1 per 1,500 m^3 or each material type

Construction observation and monitoring required during the ballast layer includes:

a) Verification that all pre-construction testing has been performed and that laboratory test results indicate compliance with the Project Specifications. The CQA Officer shall assure that the Project Manager and the Contractor receive prompt notification of material conformance.

b) Observe that care is taken when placing the sand / ash ballast layer that the geomembrane is not punctured or damaged during placement operations. Frequent video records should augment observations to reflect the method of placement.

c) Observe and document that appropriate light ground pressure equipment is used and that such equipment avoids sharp turns.

d) Observation and monitoring of hauling equipment and spreading equipment to verify that the minimum thickness is maintained for spreading and hauling equipment above the HDPE geomembrane.

e) Collect and transmit to the laboratory the required number of samples for testing.

f) Communicate with the laboratory to verify that the materials tested comply with the Project Specifications.

g) Visually observe the sand material to inspect for any variability in the material including variation in gradation, excess fines or any deleterious material present.

h) Visually observe the ash material to inspect for any variability in the material including variation in gradation, excess fines or any deleterious material present.

i) Verify that the CQA Survey has been completed and that the Record Drawings furnished by the surveyor indicates compliance with the lines, grades, elevations, and tolerances as indicated by the Project Drawings and Specifications

If the equipment or material placement procedures do not comply with the Project Specifications, the geomembrane shall be exposed and inspected for potential damage.

7.4 CONSTRUCTION

a) Where applicable, carefully place the sand ballast layer on the liner ensuring any potential for geomembrane damage is minimised. CQA Officer is to ensure proper documentation of this process by means of video recordings and dated photographs.

b) Where applicable, any wrinkles that do not meet the specification must be remedied prior to the placement of the ballast layer.

c) Under no circumstances may a machine or truck track be directly on the protection geotextile, separation geotextile or the geomembrane liner once laid to its specification. The minimum thickness of protective material as specified in the design specifications must be adhered to and verified by the Engineer prior to any vehicles driving on the protective layers above the geomembrane.

d) Where the slope, upon which the sand / ash ballast layer is to be constructed, is steeper than 1V:5H, construction is to proceed from the lower level, in an upwards direction only.

e) The sand / ash shall be placed by trucking the material over previously placed sand / ash and unloading on the previously placed material.

f) The sand / ash layer shall be no less than 200mm thick where wheeled vehicles are to traverse the sand. The method of working shall comprise end-tipping and spreading from previously placed material, or a method agreed with the Engineer prior to work commencement.

g) Spreading of the sand / ash to the required 300mm thick layer, where applicable, shall be carried out with low bearing pressure equipment. Unless otherwise agreed, the maximum permitted machinery weight for the drainage layer placement shall be 5 tons and at no point shall tracked placement equipment operate on a working platform of less than 200 mm thickness of material above the protection geotextile.

h) Locking of tracks of dozers shall be prohibited. Stopping shall be carried out slowly. To prevent any possible rotational strain on the underlying geosynthetics, no turns over a tight radius will be permitted. Where sudden stoppage of a vehicle or tight turns are witnessed, the underlying geosynthetics shall be exposed to assess if damage has occurred.

i) The Contractor may elect to prepare a trial pad to allow the Engineer to assess the proposed equipment and/or plant.

8. CONCRETE (FOR POLLUTION CONTROL DAMS AND CLEAN WATER DAMS)

8.1 SCOPE

This Plan shall be extended to cover the 25MPa and 35MPa concrete to be used as ballasting layer to ensure intimate contact between the HDPE liner and the subgrade on the slopes and base of the PCDs and CWDs, above the geotextile as well as in the trapezoidal stormwater drains, where shown on drawings or as directed by the Engineer.

8.2 MATERIALS

The concrete ballasting layer shall consist of a standard river sand, 19mm aggregate and cement to make up the desired 25MPa and 35MPa concrete as directed by relevant drawings.

In addition to the above, the layers shall satisfy the relevant clauses of SANS 1200 Part GA with regard to plain concrete small works and SANS 1200 Part H with regards to reinforced concrete.

8.3 CONSTRUCTION

At no time during construction shall pegs be used to support shuttering, reinforcing mesh or other construction materials within the PCDs and CWDs.

Concrete within the dams is to be constructed from the base upwards and shall be placed by means of overhead concrete boom pumps. At no time shall a concrete truck or plant (with the intent of delivering and placing concrete) be driven within the dam area on top of the liner system.

9. SUB-SOIL / LEAKAGE DETECTION FIN DRAINS

9.1 SCOPE

This Plan shall be extended to cover the subsoil drainage / leak detection finger drains to be used as part of the leakage detection system.

9.2 PROGRESS CONTROL TESTING

Both pre-construction and construction testing are required for these materials. Pre-construction testing consists of testing proposed materials from samples obtained at the aggregate or on-site borrow source.

Construction testing consists of testing performed from samples obtained during delivery of materials during the module or layer construction.

The tests to be performed, including testing frequency, for each material type are presented in Table 8 The testing frequencies specified in Table 8 may be increased when construction conditions warrant additional tests. Additional testing may be performed on suspect materials as recommended by the Design Engineer.

Table 8: Subsoil Drainage and LCS drainage layer construction testing

Test Designation	ASTM	Designation Frequency
Sieve Analysis	D422	1 per 1,500 m3 or each material type
Aggregate Crushing Value	SANS3001-AG10	1 per 1,500 m3 or each material type
Flakiness Index	SANS 5847	1 per 1,500 m3 or each material type

Construction observation and monitoring required during the drainage gravel and LCS drainage layer includes but not limited to the following:

a) Verification that all pre-construction testing has been performed and that laboratory test results indicate compliance with the Project Specifications. The CQA Officer shall assure that the Project Manager and the Contractor receive prompt notification of material conformance.

b) Verify that the material upon which the cushion/protection soil and drainage aggregate layers will be placed (HDPE geomembrane) has been installed in accordance with the Project Drawings and Specifications, and that all required testing, and as-built documentation have been completed.

c) Observe that care is taken when placing the soil cushion layer and LCS drainage layer on the HDPE geomembrane and that the geomembrane is not punctured or damaged during placement operations. Frequent video records should augment observations to reflect the method of placement.

d) Observe and document that appropriate light ground pressure equipment is used and that such equipment avoids sharp turns.

e) Observation and monitoring of hauling equipment and spreading equipment to verify that the minimum thickness is maintained for spreading and hauling equipment above the HDPE geomembrane.

f) Collect and transmit to the laboratory the required number of samples for testing.

g) Communicate with the laboratory to verify that the materials tested comply with the Project Specifications.

h) Visually observe the soil cushion and drainage materials to inspect for any variability in the material including variation in gradation, excess fines or any deleterious material present.

j) Verify that the CQA Survey has been completed and that the Record Drawings furnished by the surveyor indicates compliance with the lines, grades, elevations, and tolerances as indicated by the Project Drawings and Specifications

If the equipment or material placement procedures do not comply with the Project Specifications, the geomembrane shall be exposed and inspected for potential damage.

9.3 CONSTRUCTION

a) Properly install the perforated pipe in a well-prepared trench with a suitable bedding material to ensure proper alignment, support, and drainage efficiency.

b) The crushed rock aggregate must be inspected and approved by the Design Engineer and CQA Engineer prior to being placed within the sub-soil drainage fin drains.

c) Where the slope, upon which the sub-soil drainage layer is to be constructed, is steeper than 1V:5H, construction is to proceed from the lower level, in an upwards direction only.

d) The Contractor shall take care to ensure that the aggregate is worked around the leachate collection pipes to provide uniform support around the pipes.

10. HDPE PIPES – LEACHATE COLLECTION SYSTEM

10.1 SCOPE

This Plan shall be extended to cover the HDPE pipes to be used as the leachate collection and leachate carrier pipes as part of the leachate management system.

10.2 MATERIAL

10.2.1 *MEDIUM PRESSURE PIPES*

a) HDPE pipes shall be type Class 16 unless otherwise specified. HDPE fittings shall be manufactured from HDPE type PE100 PN18 pipe unless otherwise specified.

b) All puddle flanges on HDPE fittings shall be thrust bearing with a minimum thickness of 75 mm unless otherwise specified.

c) HDPE pipes that are to be used for the leachate carrier pipelines are to have all joints reamed both internally and externally.

10.2.2 *LEACHATE COLLECTION SYSTEMS*

a) The HDPE pipes shall conform to SABS 533 Part II of 1982. The sizes referred to in both specifications and drawings are the minimum outside diameter.

b) Size of Perforations

The apertures in the pipes shall not exceed the following filter criteria:

For slots $\frac{\text{D85 (filter)}}{\text{slot width}} < 1.2$

For circular holes $\frac{\text{D85 (filter)}}{\text{slot width}} < 1.0$

c) Jointing procedures for HDPE pipes shall be as follows:

PIPE DIAMETER	REQUIRED JOINTING METHOD
50	“Plasson” type compression coupling
110/160/200	Electrofusion welding on site

10.3 CONSTRUCTION

a) Where applicable, carefully place the leachate drainage layer on the liner ensuring any potential for geomembrane damage is minimised. CQA Officer is to ensure proper documentation of this process by means of video recordings and dated photographs.

b) Where applicable, any wrinkles that do not meet the specification must be remedied prior to the placement of the leachate drainage layer.

c) The crushed rock aggregate must be inspected and approved by the Design Engineer and CQA Engineer prior to being placed within the leachate collection herringbone fins.

d) Under no circumstances may a machine or truck track be directly on the protection geotextile, separation geotextile or the geomembrane liner once laid to its specification.

The minimum thickness of protective material as specified in the design specifications must be adhered to and verified by the Engineer prior to any vehicles driving on the protective layers above the geomembrane.

e) Where the slope, upon which the leachate drainage layer is to be constructed, is steeper than 1V:5H, construction is to proceed from the lower level, in an upwards direction only.

f) The drainage aggregate shall also be placed around the leachate collection pipes as shown on the Drawings. The Contractor shall take care to ensure that the aggregate is worked around the leachate collection pipes to provide uniform support around the pipes.

g) The Contractor may elect to prepare a trial pad to allow the Engineer to assess the proposed equipment and/or plant.

11. EROSION PROTECTION

11.1 SCOPE

This Plan shall be extended to cover a woven 100% jute yarn components required for the soil protection for the unlined clean water stormwater drains (earth drains) – or similar approved product.

11.2 MATERIAL

Woven 100% natural jute yarn compromising a coarse 65% open mesh. The erosion protector will have the following properties as listed in the below table 9:

Property	Value
Length	68.5m lapped lengths in bale
Width	1.22m
Thickness	5mm
Open Area	65% approximate
Openings	10 x 10mm
Weight	292 g/m^2
Construction	10.8 Warp Threads/ 100mm
Plain open weave, single yarn	12.0 Weft Threads/100mm

11.3 CONSTRUCTION

a) Level the area, removing rocks, roots etc. to allow maximum soil contact with the erosion protection blanket.

b) Lay the erosion protection blanket loosely to closely match topsoil taking care not to stretch between anchor points.

c) Covering a sloped area is easily achieved by unfolding the the erosion protection blanket from the top of the slope. Successive lapped lengths should be overlapped by 150mm or sewn together with sisal twine.

d) When unfolding the erosion protection blanket, the beginning of the lapped length should be anchored in a lock trench.

e) Peg down the erosion protection blanket at 1m grid intervals both down and across the slope. Wooden pegs of 300mm or longer, notched to snag the fabric are recommended.

f) Caution, always avoid openings between adjacent lengths

12. SCRAP MATERIAL

On completion of liner installation, the liner Installer shall dispose of all trash and scrap material in a location as approved by the Engineer/Owner, remove all equipment used in connection with the work herein and shall leave the premises in a neat acceptable manner. No scrap material shall be allowed to remain on the lined surface.

13. LINER ACCEPTANCE

The earthworks, the GCL, the HDPE geomembrane liner, the protection geotextile, the coarse sand layer and concrete, as well as the sub-soil drains will be accepted by the Owner’s Representative/CQA Officer when:

a) The entire installation is completed or an agreed-upon subsection of the installation is completed.

b) All Installer’s QC documentation is completed and submitted to the Owner’s Representative/CQA Officer.

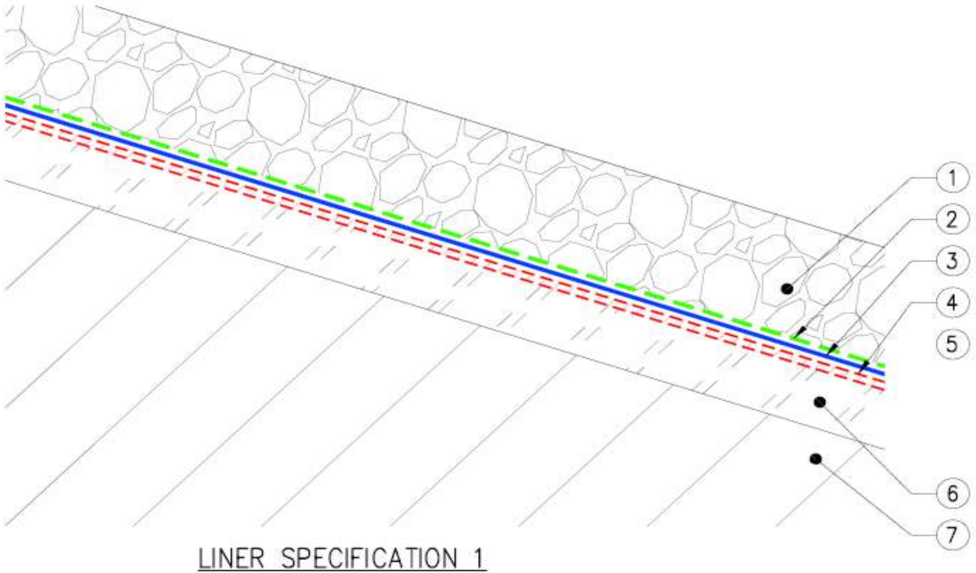
c) Verification of the adequacy of all field seams and repairs and all associated liner testing is complete.

14. PROPOSED ASH DISPOSAL FACILITY LINER DETAIL

As per the Design Report, the proposed ADF liner detail is illustrated below.

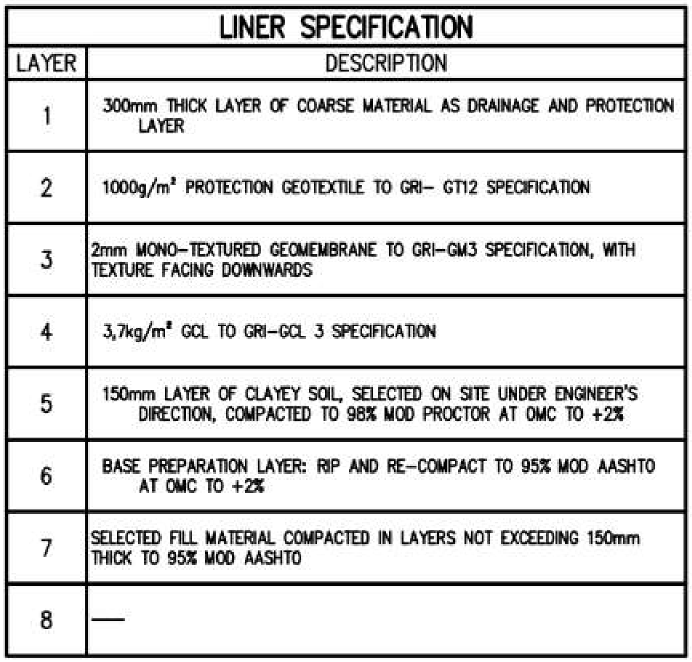
The containment barrier system comprises of the following layer works listed from the ADF surface at the top to the in-situ based prepared layer:

- 300mm thick layer of coarse sand / ash as drainage and protection sand layer
- Geotextile Protection Layer – Woven geotextile of nominal mass 1000g/m^2
- Primary Geomembrane Liner – 2.0mm Mono-Textured HDPE Geomembrane Liner to GRI-GM13 specifications, with texture facing downwards.
- Geosynthetic Clay Liner – 3.7kg/m^2 Geosynthetic Clay Liner conforming to GRI-GCL 3 specifications to the approval of the Engineer.
- 150mm Layer of clayey soil, selected on site under Engineer's direction, compacted to 98% MOD Proctor at OMC +2%
- Base Preparation Layer – Rip and recompact 300mm thick layer of in-situ material to 95% MOD AASHTO Density at OMC +2%



LINER SPECIFICATION 1
1:10

Figure 1: Proposed ADF Containment Barrier System Detail (Label and Keys on Figure 2)



LINER SPECIFICATION

LAYER	DESCRIPTION
1	300mm THICK LAYER OF COARSE MATERIAL AS DRAINAGE AND PROTECTION LAYER
2	1000g/m² PROTECTION GEOTEXTILE TO GRI– GT12 SPECIFICATION
3	2mm MONO–TEXTURED GEOMEMBRANE TO GRI–GM3 SPECIFICATION, WITH TEXTURE FACING DOWNWARDS
4	3,7kg/m² GCL TO GRI–GCL 3 SPECIFICATION
5	150mm LAYER OF CLAYEY SOIL, SELECTED ON SITE UNDER ENGINEER'S DIRECTION, COMPACTED TO 98% MOD PROCTOR AT OMC TO +2%
6	BASE PREPARATION LAYER: RIP AND RE–COMPACT TO 95% MOD AASHTO AT OMC TO +2%
7	SELECTED FILL MATERIAL COMPACTED IN LAYERS NOT EXCEEDING 150mm THICK TO 95% MOD AASHTO
8	—

Figure 2: Keys and Labels (Please refer to Figure 1)

15. PROPOSED POLLUTION CONTROL AND CLEAN WATER DAM LINER DETAIL

As per the Design Report, the proposed PCD and CWD liner detail is illustrated below. The containment barrier system comprises of the following layer works listed from the PCD / CWD surface at the top to the in-situ based prepared layer:

- 300mm thick 35MPa Concrete Slab on base and up to 1m height on side slopes, reinforced with one layer of mesh ref 395
- Geotextile Protection Layer – Woven geotextile of nominal mass 1000g/m^2
- Primary Geomembrane Liner – 2.0mm Mono-Textured HDPE Geomembrane Liner to GRI-GM13 specifications, with texture facing downwards.
- Geosynthetic Clay Liner – 3.7kg/m^2 Geosynthetic Clay Liner conforming to GRI-GCL 3 specifications to the approval of the Engineer.
- 150mm Layer of clayey soil, selected on site under Engineer’s direction, compacted to 98% MOD Proctor at OMC +2%
- Base Preparation Layer – Rip and recompact 300mm thick layer of in-situ material to 95% MOD AASHTO Density at OMC +2%

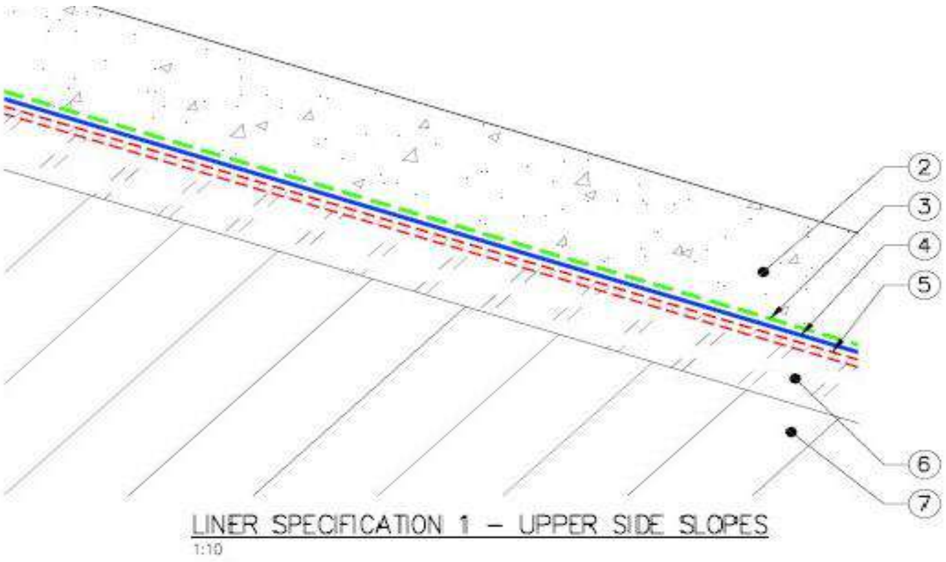
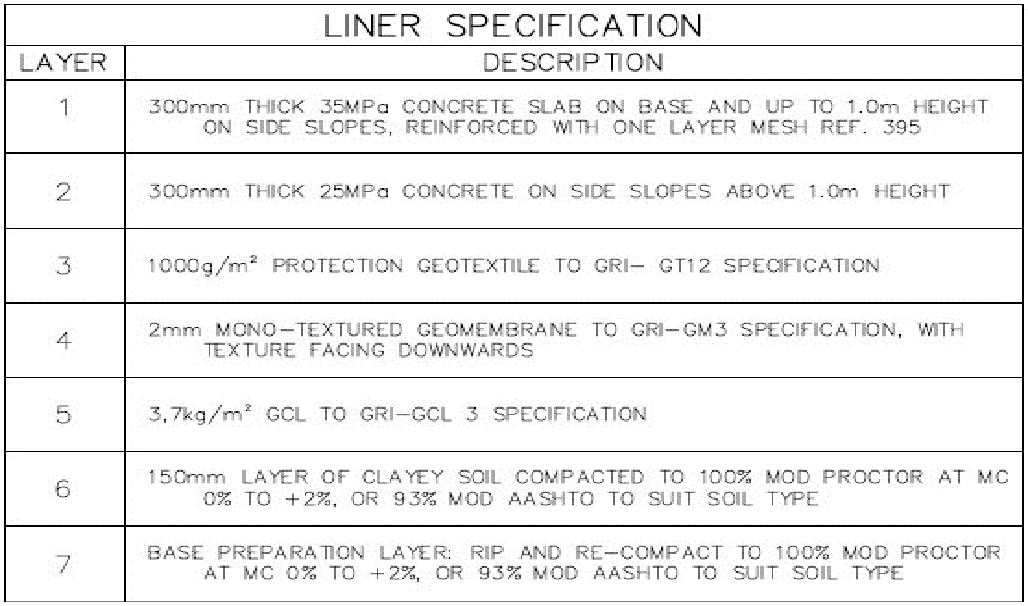


Figure 3: Proposed PCD / CWD Containment Barrier System Detail (Label and Keys on Figure 4)



LINER SPECIFICATION	
LAYER	DESCRIPTION
1	300mm THICK 35MPa CONCRETE SLAB ON BASE AND UP TO 1.0m HEIGHT ON SIDE SLOPES, REINFORCED WITH ONE LAYER MESH REF. 395
2	300mm THICK 25MPa CONCRETE ON SIDE SLOPES ABOVE 1.0m HEIGHT
3	1000g/m² PROTECTION GEOTEXTILE TO GRI– GT12 SPECIFICATION
4	2mm MONO–TEXTURED GEOMEMBRANE TO GRI–GM3 SPECIFICATION, WITH TEXTURE FACING DOWNWARDS
5	3,7kg/m² GCL TO GRI–GCL 3 SPECIFICATION
6	150mm LAYER OF CLAYEY SOIL COMPACTED TO 100% MOD PROCTOR AT MC 0% TO +2%, OR 93% MOD AASHTO TO SUIT SOIL TYPE
7	BASE PREPARATION LAYER: RIP AND RE–COMPACT TO 100% MOD PROCTOR AT MC 0% TO +2%, OR 93% MOD AASHTO TO SUIT SOIL TYPE

Figure 4: Keys and Labels (Please refer to Figure 3)

16. DOCUMENTATION

An effective Quality Assurance program depends on thorough monitoring and documentation of all construction activities during all phases of construction and as a minimum shall comply with SANS 10409 as amended and the all-important inception meeting. Documentation shall consist of daily record keeping (including minutes of meetings), construction problem resolutions, design and specification changes, photographic records, weekly progress reports, chain of custody forms for test sample tracking, and a certification and summary report. During construction, all documentation shall be kept on-site and will be available for review by the Project Manager, Design Engineer, or CQA Officer.

No section of the barrier system may be covered up until the CQA Officer and Design Engineer (as appropriate) observes and approves the completed section of the barrier system and assures that all CQA documentation has been completed.

16.1 DAILY RECORD KEEPING

Daily records shall consist of field notes, observation and testing data sheets, summary of the daily meeting with the Installer and Contractor, and reporting of construction problems and resolutions. This information shall be submitted weekly along with a weekly summary to the CQA Officer. Copies of all CQA documentation shall be maintained at the site and be made available for review by the Project Manager.

16.2 SOILS OBSERVATION AND TESTING DATA SHEETS

Soils observation and testing data sheets generally include the following information:

- Date, project name, location, and weather data

- A reduced-scale site plan, or full-scale plots, showing work areas and test locations
- Descriptions of ongoing construction
- Summary of test results and samples taken, with locations and elevations
- Off-site materials received including quarry certificates
- Test equipment calibrations, if necessary
- Signature or initials of the CQA Officer

16.3 GEOSYNTHETIC OBSERVATION AND TESTING FORMS

Geosynthetic observation and testing forms generally include the following information:

- Date, project name, location, and weather data
- Identification of panel or seam number
- Numbering system identifying test or sample number
- Location and identification of repairs and date of repair
- Length and/or thickness measurements for geomembrane panels or seams
- Welding machine temperatures and settings
- Welding machine and technician identifications
- Location of tests and test results
- Identification of testing technicians and time of tests
- Signature or initials of the CQA Officer

16.4 CONSTRUCTION PROBLEM AND RESOLUTION DOCUMENTATION

Any construction problem which cannot be resolved between the Installer, Contractor, and CQA Officer may require a special meeting in order to resolve the problem. The problem should be discussed with the Project Manager, and Design Engineer if a design issue is involved. Specific written documentation of that problem should be prepared, if warranted, and will generally include the following information:

- Detailed description of the problem
- Location and cause of the problem
- How and when the situation or deficiency was identified
- How the problem was resolved
- Any measures taken to prevent similar problems in the future
- Signature of the Design Engineer and CQA Officer

Copies of all Construction Problem and Resolution correspondence will be submitted to the Project Manager.

16.5 PHOTOGRAPHIC DOCUMENTATION

All phases of construction shall be sufficiently photographed and/or audio video recorded by the CQA Officer. Photographs shall be identified by separate photographic log by location, time, date, and name of the person taking the photograph. A camera which records the time and date shall be used. Representative photographs will be included in the certification report.

16.6 DESIGN AND SPECIFICATION CHANGES

If it is necessary to address Project Drawings and/or Projects Specification changes, modifications, or clarifications during construction, the CQA Officer or Design Engineer will inform the Project Manager. Project Drawing and Project Specification changes shall only be made with written agreement from the Project Manager and Design Engineer, and approval of the regulatory authorities if required.

16.7 CONSTRUCTION REPORT

At the completion of construction, a construction report shall be prepared and signed by the CQA Officer and Design Engineer to certify that the work has been performed in compliance with the license conditions, Project Drawings and Project Specifications and will contain the following general information:

- Summary of construction activities
- Observation and test data summary sheets, inclusive of a table reflecting statistical analyses i.e. for each test method on all materials the number of tests; minimum, maximum and mean values; standard deviation; number of non-compliances and rectification shall be included (sample table attached in **Annexure B**)
- Sampling, testing locations, and test results
- Confirmation of interface shear strength parameters (peak and residual) using the actual geosynthetic materials supplied and installed on site
- A description of significant construction problems and the resolution of these problems
- Changes to the Project Drawings or Project Specifications and the justification for these changes
- Record drawings
- Specific barrier performance confirmation tests, which may include amongst others:
    - Electric leak location survey according to ASTM D7007 or D8265 or similar amendment to encourage competitive procurement provided the

  independence and performance of the survey are not compromised as agreed with the lead authority in writing prior to implementation;
  - Differential Scanning Calorimeter (DSC) test results on the geomembrane installed for the sample identified by the regulator or agreed agent when required;
  - In-situ geomembrane tensile strain confirmation for the construction method employed (the test usually been undertaken in the trial pad or initial footprint area), or laboratory test simulating site loading conditions on site specific material or similar approved
  - The destructive test specimens of installed geomembrane welds shall be retained for visual and physical inspection by the regulator, with photographic record and weld dimensions included in the report.

- Detail of instrumentation that is provided to test, measure and confirm assumed parameters used in design and construction performance assessments e.g. settlement beacons, flow gauges, vibrating wire piezometers, strain gauges, inclinometers and similar, including co-ordinates and elevation.
- A certification statement signed and certified by the Design Engineer and CQA Officer, by whom the CQA activities were supervised and work performed in responsible charge.

The Record Drawings shall be prepared by the Surveyor and shall accurately locate all construction items including the lines, grades, and thickness of all soil components for the barrier system.

17. ELECTRIC LEAK LOCATION TESTING FOR GEOMEMBRANES COVERED WITH EARTHEN MATERIAL

17.1 SCOPE

This specification is written in conjunction with ASTM D7007 and ASTM D6747 for the proposed Electric Leak Location (ELL) testing of the primary geomembrane liner.

17.2 DESCRIPTION

Among the various types of electrical leak location methodologies used to evaluate geosynthetic lining systems in the field, the dipole method has a unique distinction in that it is the only ASTM standardized ELL practice that can locate damage to covered geomembranes after cover material placement.

The dipole geo-electric method according to ASTM D7007 uses the intrinsic insulation properties of geomembranes to localize perforations that enable water to pass from one side of the geomembrane to the other (see Figure 5). A current of approximately 300-500V is injected into the covering material, and a grounding electrode is placed outside of the geomembrane limits. The current must pass through the leak in order to reach the ground, which generates a distinct electrical field that can be identified and located by a specialized technician.

For surveys with earthen materials on the geomembrane, the earthen materials shall have adequate moisture to provide a continuous path for electrical current to flow through the leak.

A high voltage is applied to the cover material with a positive electrode. The power source is grounded to the subgrade underneath the geomembrane. Voltage measurements are taken in a grid pattern throughout the survey area using a dipole instrument. Leak locations cause a sine wave pattern in the voltage measurements as the dipole instrument travels across a hole location.

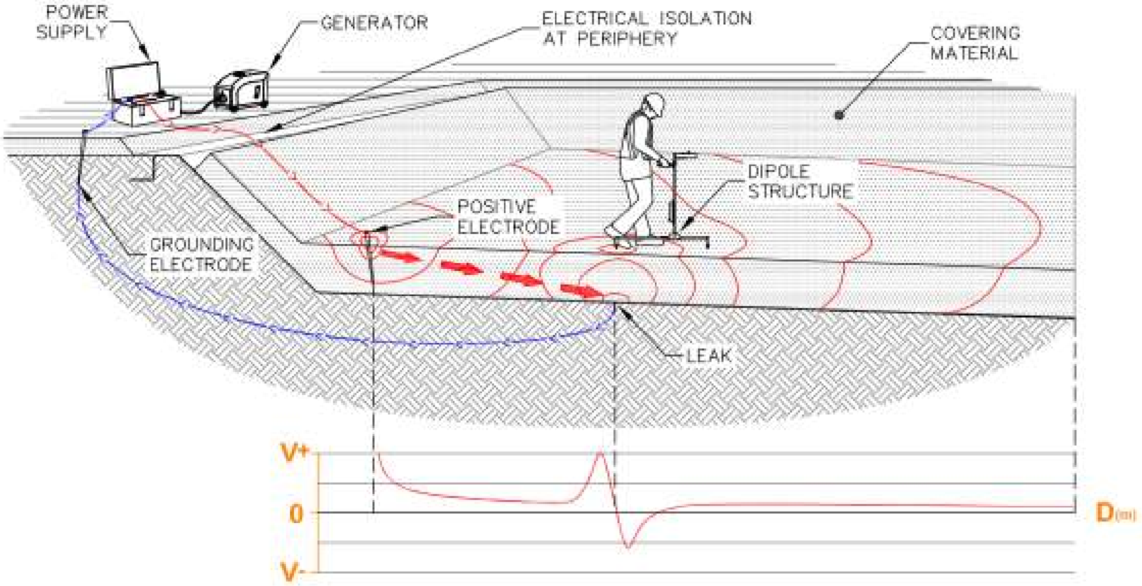


Figure 5: Diagram of the Electrical Leak Location Method for surveys for covered geomembrane

17.3 MATERIALS / EQUIPMENT

- Calibrated excitation power supply
- Current electrodes (positive and negative)
- Electric Cable (insulted)
- Dipole Tester (with measurement electrodes)
- Calibrated Data Logger (measuring device)

- Water Tanker (with hose and fittings)
- Power Supply (Generator)
- Handheld GPS
- Mapping Software

17.4 SITE / DESIGN REQUIREMENTS

- The maximum cover, over the geomembrane to be tested, should not exceed 600mm thick and the cover material to be as homogenous as possible (to be included in material specification).
- The wrinkle sizes on the geomembrane should be reduced as far as possible and the specification should cater for the placement of the cover material to reduce the wrinkles. It is almost impossible to detect damages without intermit contact between the geomembrane and the conductive layer below.
- **Most Critical** – The survey area must be electrically isolated, no exceptions.
  - Anchorage of the geomembrane - The loose end of the geomembrane to exit the anchor trench and form a flap after the trench has been backfilled. This is the best way to ensure complete isolation of the area to be tested.
  - Drainage material or any material covering the geomembrane to only be filled on the inner side of the anchor trench (no contact with in-situ soil).
  - No access ramps or internal berms to be constructed over the perimeter of the cell unless they are isolated by means of trenches or other isolation methods. Ideally any fill earthworks structures that continue outside the lined area of the cell should only be completed after the ELL survey. No fill over the geomembrane shall exceed 600mm before the test has been concluded.

- The protection geotextile over the geomembrane must be kept within the confined isolated area. Wet geotextiles become conductive so this must be considered during placement.
- The CCL moisture content should be a minimum of 10% to ensure conductivity. This should be carefully specified for double composite systems.
- If the CCL has been exposed for an extended period of time it should be hydrated prior to covering.
- The in-situ soil conductivity and moisture content should be tested by an accredited laboratory. The results obtained from the ELL survey depends on the in-situ site conditions.
- In climates were high rainfall or extreme rain events are expected, a rain flap on the geomembrane should be considered to prevent overflow of the runoff and hence maintain isolation.
- Conductive paths such as metal pipes penetrations, pump grounds and batten strips on concrete should be isolated or insulted from the earthen material on the geomembrane whenever practical.
- If the leak detection layer is not a gravel, but a geocomposite material then keep in mind that a lot more water will be required as it will need to be saturated before commencing the test.
- For the primary geomembrane:
    - Material under the primary geomembrane must be conductive (if it is a CCL it must have sufficient moisture).
    - The conductive layer under the geomembrane must be accessible for and earth (negative electrode).
    - The covering layer on the primary geomembrane, again, must be properly isolated from the underlying layer.

17.5 GENERAL

- The lining contractor shall be available on site in the event that repairs are required.
- Complete isolation between the cell being constructed and exiting adjacent cells must be maintained.
- A water tanker must be available on site during each day of the ELL survey. If possible the tanker should be stationed inside the lined area of the cell to be tested. If not possible any connections from the water tank (hose pipes etc.) must be isolated (i.e. there should be no leaks in the hose and the hose material should be non-conductive).
- The programme and planning for the installation of the lining system must allow for the ELL survey to be conducted on the Secondary and Primary geomembranes.

17.6 WARNING

The electrical methods used for geomembrane electric leak location testing uses high voltage (up to 500v), resulting in the potential for electric shock or electrocution. Adequate safety measures must be taken to prevent potential incidents during the survey.

18. ELECTRIC LEAK LOCATION TESTING FOR EXPOSED GEOMEMBRANES- ARC TESTING

18.1 OVERVIEW

The arc testing method is generally preferred for bare geomembranes, since no water is required to perform the test and it can be more sensitive than the water-based methods because the leak detection does not depend on water getting through the leak. The minimum sensitivity is a 1 mm diameter leak per ASTM D7953, but leaks smaller than that have regularly been located.

A high voltage power supply is applied to a test wand above the geomembrane and is grounded to the underlying conductive layer. The area is swept with a test wand and an electrical arc is formed in the presence of a leak. When the system senses the discharge current arc, it is converted into visual and audio alarms. The test wand can be custom sizes and shapes for specific applications.

This type of test requires that the geomembrane is in contact with the subgrade. If the separation distance is greater than 3 cm, such as on a wrinkle or other “poor contact” conditions, the instrument is not likely to arc. The surface of the geomembrane must be clean and dry.

18.2 METHOD STATEMENT

Method statement for Electric Leak Location (ELL) Testing of Exposed Geomembranes – ARC Testing

18.3 APPLICABLE STANDARD

ASTM D7953 – 14 – Standard Practice for Electrical Leak Location on Exposed Geomembranes Using the Arc Testing Method.

18.4 MATERIALS / EQUIPMENT

- Battery Power supply (800V – 35 000V)
- Current electrode (Negative grounding electrode)
- Electric cable (insulated)
- Arc testing wand (Positive electrode)
- Spray paint
- Handheld GPS

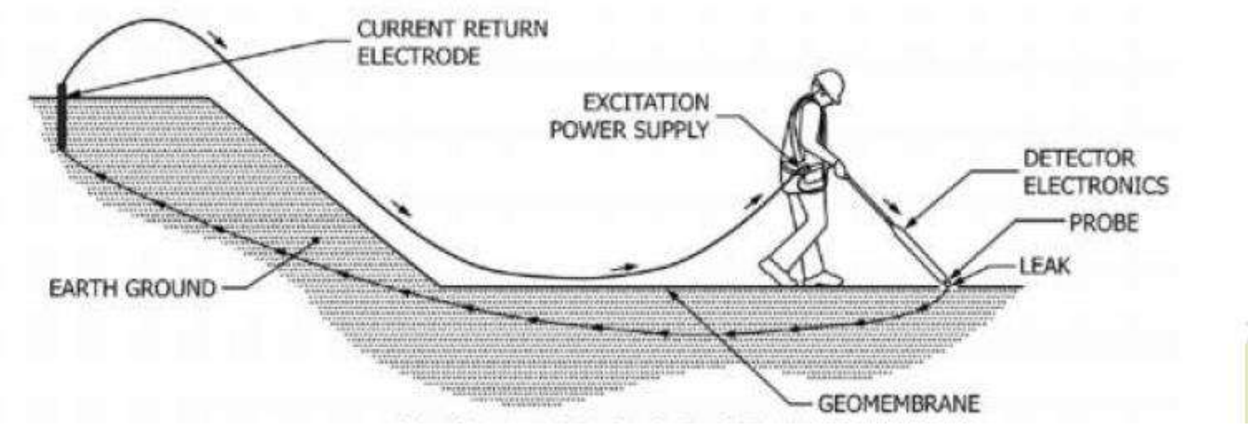


Figure 6: Diagram of the Electrical Leak Location Method for surveys for exposed geomembrane

18.5 CALIBRATION OF ARC TESTING EQUIPMENT

A test pad is constructed on site, replicating the layer works in the design, to the level of the exposed geomembrane layer to be tested. Punctures to the geomembrane are made at random locations and to varying sizes. The arc testing equipment is then set up and adjusted to successfully locate the punctures on the liner. Once all the punctures are located and adjustments made to the sensitivity of the arc tester, if any, the arc tester is calibrated.

18.6 SURVEY METHODOLOGY

The site will be visually inspected by the Electronic Leak Location (ELL) Technician to assess the site-specific conditions. A critical requirement for ELL testing methods is complete isolation of the survey area, and this will be checked and confirmed by the ELL Technician prior to the commencement of testing. In addition to the site being isolated, the exposed geomembrane must be clean and dry as stated in the ASTM Standard D7953 for arc testing to commence.

The negative electrode will be positioned at suitable location into the earthen material in the ground near the area to be tested. The arc testing wand serves as the positive electrode. The electrodes are connected to a battery power supply to generate a voltage potential above and below the HPDE geomembrane. The battery power supply is strapped around the waist of the ELL Technician. Before the testing commences, the ELL technician will inspect the area to ensure that it is clean and dry. No personal or equipment is allowed in the testing area during the testing process. The ELL technician will use the arc testing wand to sweep through the surface of the exposed geomembrane until the entire area requiring testing has been covered.

If a leak is detected, the ELL technician will mark and label the leak using spray paint, log the position using a handheld GPS and take a picture for record keeping. The ELL technician will then notify the relative site personal of the damages found in order for repairs to be done to the geomembrane liner.

18.7 GENERAL

- The lining contractor shall be available on site in the event that repairs are required.
- Repairs are to be carried out in accordance with Section 4.13.3 procedures.
- Testing of the repair seams shall be captured and documented.
- Complete isolation of the cell/dam being constructed, and any existing adjacent cells must be maintained.
- No site personal is allowed to handle any ELL testing equipment except the ELL technician.

18.8 SAFETY

The electrical methods used for geomembrane electric leak location testing uses high voltage (up to 35 000v), resulting in the potential for electric shock or electrocution. Adequate safety measures must be taken to prevent potential incidents during the survey.

18.9 RISK ASSESSMENT

18.9.1 *Risk Assessment of Electric Leak Location (ELL) Testing*

Identification of Hazards

The electric leak location (ELL) testing uses high voltage (up to 35 000v), resulting in the potential for electric shock or electrocution.

People at Risk

On-site workforce working in the vicinity where the ELL testing is being conducted. If any personnel on site makes direct contact with the equipment being used during the testing process, they can experience an electric shock or electrocution.

Precautions

No personnel will be allowed in the areas where the ELL testing is being conducted. Only the experienced ELL Technician is allowed to set up and handle all ELL equipment. The ELL Technician will notify the relevant site personnel before the commencement of testing and after the completion of testing.

In the event of an incident on site, the ELL Technician will report the incident to the contractor immediately.

19. REFERENCES

National Norms and Standards for Disposal of Waste to Landfill, Notice No. R. 636, 23rd August 2013

SANS 1526:2015 Edition 3 - Thermoplastics polyolefin sheeting for use as a geomembrane

SANS 10409:2005 Design, selection and installation of geomembranes

GRI GM13, Geosynthetic Research Institute

GRI GM9, Geosynthetic Research Institute

GRI GCL3, Geosynthetic Research Institute

GRI GCL5, Geosynthetic Research Institute

ASTM D6747-15 - Standard Practices for selection of techniques for electrical leak location of leaks in geomembranes

ASTM D7007-16 - Standard Practices for electrical methods for locating leaks in geomembrane covered with water or earthen materials

Toepfer, G.W. 2015. The Complete Field Guide to Ensuring Quality Geosynthetic Installations, Volume 1: Fundamentals & Geomembrane

Environment Agency UK, Report LFE5 - *Using geomembranes in landfill engineering*

(BAM), *Certification Guidelines for Plastic Geomembranes used to Line Landfills and Contaminated Sites*, English translation of the Second Revised edition, 09/99.

GSI Webinar Series - Quality Control and Quality Assurance of Geosynthetics in Solid and Liquid Waste Containment Systems, Dr R Koerner, 2015

Hsuan, Y. G. & Koerner, R. M. 1995. Long term Durability of HDPE Geomembranes Part 1 – Depletion of Antioxidants. *GRI Report no 16*

Koerner, R. M. & Koerner, G. R. 2012. Lifetime Prediction of Laboratory UV Exposed Geomembranes: Part 1 – Using a Correlation factor. *GRI Report no 42*

Chief Directorate: Engineering Services. 2020. Construction Quality Assurance Plan for Government Waterworks Waste Disposal Facility Pollution Control Works

Chief Directorate: Engineering Services. 2020. Technical Advisory Note: Construction Quality Assurance for Water Conservation and Pollution Control Barrier Systems.

For EPCM

ANNEXURE A –

FIELD INSPECTION TEMPLATES

Certificate of Acceptance of Soil Subgrade Surface

CONTRACT NAME ________________________________
CONTRACT NUMBER ________________________________
CLIENT ________________________________
LOCATION ________________________________

I, the undersigned, a duly appointed representative of _________________________________ have visually observed the soil subgrade surface described below, and found it to be an acceptable surface finsh on which to install the geomembrane.

This certification is based on observations of the surface of the subgrade only. No subterranean inspections or tests have been performed by _________________________________ and no representations or warranties regarding conditions which might exist below the surface of the subgrade are made. _______________________ accepts no responsibility for conformance of the subgrade to this project's specifications.

Area being accepted : ________________________________
____
____
____

Comments : ________________________________
____
____
____
____
____

INSTALLATION CONTRACTOR'S REPRESENTATIVE		CIVIL CONTRACTOR'S REPRESENTATIVE	
Date	____________	Date	____________
Signature	____________	Signature	____________
Name	____________	Name	____________
Title	____________	Title	____________

CQA REPRESENTATIVE		ENGINEER'S REPRESENTATIVE	
Date	____________	Date	____________
Signature	____________	Signature	____________
Name	____________	Name	____________
Title	____________	Title	____________

REVISED 21.02.2024

Material Delivery / Inventory Checklist

Page ______ of ______

DATE ____________________ TRUCK NO ____________________

GOODS RECEIVED BY ____________________

GOODS CHECKED BY (optional) ____________________

CONTRACT NAME ____________________

LOCATION ____________________ MATERIAL TYPE ____________________

ITEM	COMPLETE ROLL NUMBER	BATCH NUMBER	ROLL SIZE	DAMAGE / REMARKS (a)
1				
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				
13				
14				
15				
16				
17				
18				
19				
20				

a Extent of damage shall be noted in detail.

NB Photographs shall be taken of all materials during offloading and storage placement for record purposes

REVISED 21.02.2024

Trial Seam Information Report Form

Page ______ of ______

CONTRACT NAME ____________________ CONTRACT NUMBER ____________________

MATERIAL DESCRIPTION ____________________

DATE/TIME	TRIAL NO	AMBIENT TEMP	QC INITIALS	MACHINE NO	SEAMER INITIALS	EXTRUSIONS WELDS				WEDGE WELDS					PASS/ FAIL	VALUES		
						BARREL TEMP		PREHEAT TEMP		WEDGE TEMP		WEDGE PRESSURE		SPEED SETTING		SHEAR	PEEL	
						SET	ACTUAL	SET	ACTUAL	SET	ACTUAL	SET	ACTUAL					

REVISED 21.02.2024

Certificate of Acceptance of Surface Finish

CONTRACT NAME ______________________________
CONTRACT NUMBER ______________________________
CLIENT ______________________________
LOCATION ______________________________

I, the undersigned, a duly appointed representative of ___________________________________ have visually inspected the surface finish described below, and found it to be an acceptable surface finsh on which to install the geomembrane.

This certification is based on observations of the surface finish only. No subterranean inspections or tests have been performed by ___________________________________ and no representations or warranties regarding conditions which might exist below the surface finish are made. ________________________ accepts no responsibility for conformance of the subsurface to this project's specifications.

Area being accepted : ______________________________

Comments : ______________________________

INSTALLATION CONTRACTOR'S REPRESENTATIVE

Date ______________________________

Signature ______________________________
Name ______________________________
Title ______________________________

EARTHWORKS CONTRACTOR'S REPRESENTATIVE

Date ______________________________

Signature ______________________________
Name ______________________________
Title ______________________________

OWNER'S REPRESENTATIVE

Date ______________________________

Signature ______________________________
Name ______________________________
Title ______________________________

ENGINEER'S REPRESENTATIVE

Date ______________________________

Signature ______________________________
Name ______________________________
Title ______________________________

REVISED 21.02.2024

Panel Placement Form

Page ______ of ______

CONTRACT NAME ____________________ CONTRACT NO ____________________

MATERIAL DESCRIPTION ____________________

DATE/TIME	PANEL NO	ROLL NO	PANEL LENGTH	PANEL WIDTH	COMMENTS/PANEL LOCATION

REVISED 21.02.2024

Eskom

epcm

Panel seaming form

Page ______ of ______

CONTRACT NAME ________________________________ CONTRACT NO ____________________

MATERIAL DESCRIPTION ____

DATE/TIME	SEAM NO	PANEL NO	SEAM LENGTH	SEAMER INITIALS	MACHINE NO	SPEED SETTING	TEMP SETTING	PRESSURE SETTING/ READING	WINDS	TEMPERATURE		TEST RESULT PASS / FAIL	COMMENTS
										AMBIENT	LINER		

REVISED 21.02.2024

Non Destructive Testing Form

Page ______ of ______

CONTRACT NAME ______________________ CONTRACT NO ______________________

MATERIAL DESCRIPTION ______________________

DATE/TIME	SEAM NO	TESTER INITIALS	INSTALLER SEAM REF	AIR TESTING						V/BOX + SPARK TEST PASS / FAIL	REMEDIAL ACTION / COMMENT
				PRESSURE			TIME		PASS/FAIL		
				START	END	VARIANCE	START	END			

REVISED 21.02.2024

Field Destructive Testing Form

Page ______ of ______

CONTRACT NAME ______________________________ CONTRACT NO ______________________

MATERIAL DESCRIPTION ____

DATE	SAMPLE I.D.	SEAM NO	MACHINE NO	SEAMER INITIALS	PEEL VALUES (a) SHEAR VALUES (a)					PASS/ FAIL	DATE TO LAB/ PACK SLIP NO	LAB P/F (b)	LOCATION/COMMENTS

a These columns required only if destructive tests are not being carried out at a laboratory.

b This column is for verification of results, if required. Actual results will be recorded separately on the in-house results form of the laboratory.

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epcm

Repair Report Form

Page _______ of _______

CONTRACT NAME _______________ CONTRACT NO _______________

MATERIAL DESCRIPTION _______________

FIELD SEAM NO	PANEL NO	REPAIR DATE	REPAIR CREW	MACHINE NO	TEST DATE	TEST CREW	TEST PASS/ FAIL	INSTALLER SEAM REF	LOCATION / COMMENTS

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epcm

Daily Report

CONTRACT NAME ______________________________

CONTRACT NUMBER ______________________________

MATERIAL DESCRIPTION ______________________________

WEATHER CONDITIONS	DATE
SUBMITTALS RECEIVED / ASSESSMENT	MATERIALS DELIVERY TO SITE
SUB-GRADE INSPECTIONS / ACTION TAKEN	GCL INSTALLATION: INSPECTION
ANCHOR TRENCH INSPECTION	PLACEMENT & SEAMING OF GSM
COVERING UP OF PLACED / SEAMED GSM	TEST SAMPLES
OTHER OBSERVATIONS / PROBLEMS / RECTIFICATION WORK	DISCUSSIONS WITH ENGINEER / CONTRACTOR / GSM INSTALLER

SIGNED (CQA OFFICER) ______________________________

SIGNED (ENGINEER) ______________________________

REVISED 21.02.2024

ANNEXURE B –

SAMPLE TESTING TRACKING SPREADSHEET

CONSTRUCTION QUALITY ASSURANCE PLAN: RECORD OF MATERIAL CONTROL TEST

Layers	PARAMETER	TEST METHOD	PERFOMANCE CRITERIO	MINIMUM FREQUENCY	Min	Max	MIN VALUE	NUMBER OF TESTS	STANDARD DEVIATION	NUMBER OF FAILURES
Base Preparation	Layer Thickness (Rip & Compact)	Survey or Excavation and Measure	Min of 150 mm	1 Test per 1000m2						
	Moisture Content (OMC)	SANS 3001-GR30:2015	OMC 2+2%							
	Compaction Density	(a). Troxler (Nuclear Density Gauge Test)	95% STD Proctor MDD	1 Test per 1000m2						
		(b). ASTM (Sand Replacement)								
	Max Particle Sizes	(a) ASTM D6913 / D6913M - 17 Sieve Analysis	Maximum Size of 25 mm	1 Test per 1000m2						
		(b). ASTM D7928 - 17 Hydrometer Test								
CLAY LINERS										
a) CCL	Layer Thickness	Survey or Excavation and Measure	150 mm Layers	1 Test per 1000m2						
	Permeability (cm/s)	ASTM D3385 (In-situ-Double Ring Inflitrometers)	1 x 10-6 cm/s	1 Test per 1 000m2						
	Grading (Particle Size Distribution)	ASTM D6913 / D6913M - 17 Sieve Analysis	Maximum size of 0.002 mm	1 Test per 1000m2						
	Density	Proctor Test (ASTM D1557-91)	95% STD Proctor MDD							
	Moisture Content (OMC)	SANS 3001-GR30:2015	OMC 2+2%	1 Test per 1000m2						
	Plasticity Index (PI)	ASTM D4318-Atterburg Limits	Min of 10 to Max that will not result in excessive desiccation cracking	1 Test per 1000m2						
	Surface Smoothness	SANS 10409								
b) GCL	Mass of GCL (g/m2)(	ASTM D5993	4000	4000m2						
	Moisture content (%) min	ASTM D5993	35	4000m2						
	Swelling index (ml/2g) min	ASTM D5890	24	50 Tons						
	Tensile str., MD (N/m)	ASTM D6768	4000	4000m2						
	Peel strength (N/m) (min)	ASTM D6496	4000	4000m2						
	GCL permeability (on pure bentonite not polymer modified)									
	Permeability (m/s) max @35 Kpa	ASTM D6766	1x10^-8	yearly						
	Permeability (m/s) max @500 Kpa	ASTM D6766 mod.	5x10^-10	yearly						
	Component Durability									
	Geotextile & reinforcing yarns (% strength retained)	ASTM D5721/ASTM D6768,	65	yearly						
	Overlap	GRI GCL3	Min of 300 mm	all edges						
HDPE GM	Thickness	ASTM D5994 / SANS 1526 :2015		Per roll						
	Density	ASTM D1505/D792	0.940g/cc	90 000kg						
	Tensile Properties	ASTM D6693								
	Yield strength		11,15,18,22,29,37,44 kN/m	9 000kg						
	Break strength		20,27,33,40,53,67,80kN/m	9 000kg						
	Yield elongation		12%	9 000kg						
	Break elogation	ASTM D6693 (Type 5)	700%	9 000kg						
	Tear resistance	ASTM D1004	93 N, 125 N, 156 N, 187 N, 249 N, 311	20 000kg \1 per site						
	Puncture resistance	ASTM D4833	240 N,320 N ,400 N,480 N,640 N,800 N	20 000kg						
	Asperity Height - Side 1	ASTM D7466	0.9mm							
	Asperity Height - Side 2	ASTM D7466	0.4mm							
	Stress crack Resistance	ASTM D5397	500hr	Per GRI GM10						
	OIT (oxidative Induction Time (min.ave)									
	_(a) STD OIT or	ASTM D 3895	100min	1 per site or 1 every 90 000kg						
	_(b) High Pressure OIT	ASTM D 5885	400min	1 per site or 1 every 90 000kg						
	Oven Aging at 85°C**	ASTM D 5721								
	_(a) STd OIT (min.ave)-%retained after 90 days or...	ASTM D 3895	55%	1 per site or 2 every 90 000kg						
	_(b) High Pressure OIT (min.ave)-% retained after 90 days	ASTM D 5885	80%	1 per site or 2 every 90 000kg						
	**(It is also recommended to evaluate samples at 30 and 60 days to compare with the 90 day response)									
	UV Resistance	ASTM D 7238								
	_(a) STd OIT (min.ave)	ASTM D 3895	Not recommended(unrealistic results due to high temp)	–						
	_(b) High Pressure OIT (min.ave)-% retained after 1600 hrs	ASTM D 5885	50% (for all thickness)	20hr uv cycles @ 75 C followed by 4hr condensation at 60C						
	Seaming	GRI GM 14 &19(Vacuum box/Spark Testing/Seam Pressure Test)	Test	Test Every Seam						
	Peel Tensile Test : Seaming	GRI GM 19/ASTM D7747	90 % of parent material strength							
GM Protection Layer										
a) Sand Layer										
	Thickness	Excavation & Mesuare or Survey	min 100 mm	1 Test per 1000m2						
	Particle Size Distribution	ASTM D6913 / D6913M - 17 Sieve Analysis	max 4.75 mm	1 Test per 1000m2						
	Grading distribution			1 Test per 1000m2						
	Compaction	Sand Replacement Test	95% STD Proctor MDD	1 Test per 1000m2						
b) Geotextile	**(Protection or Cushioning Materials)**									
	Mass /unit area (g/m2)	ISO 09864	35,040,060,080,010,000,000	9 000kg						
	Tensile Properties									
	Strength	ISO 10319	16,21,27,32,36,45	9 000kg						
	Strain at Max. Load	ISO 10319	50%							
	Trapezoidal Tear Strength	ISO 13434	0.42,0.51,0.66,0.89,0.96,1.32							
	CBR Puncture (kN/mm)			9 000kg						
	Max. Force	ISO 12236	3.1,3.6,4.1,4.9,7.6,11.0							
	Elongation at Max Force	ISO 12236	38							
	UV Str. Ret. after 500 lt. HRS EXPOSURE	ASTM D7238	70%							
	Joint Strength		90 % Strength of Parent Material							
	Strip Tensile	Strip Tensile Test	90 % of Parent Material							
	Overlap	Measure	Min of 120 mm							
Protection Geotextile										
	Thickness (at 2kPa)	SANS 9863:13	7mm	1 per 15 000m^2						
	Mass per	SANS 10221:07	1200g/m2	1 per 15 000m^2						
	Tensile Strength – weaker direction	SANS 1525:13	75kN/m	1 per 15 000m^2						
	CBR Puncture Strength	SANS 12236:13	14	1 per 15 000m^2						
	Puncture Resistance	SANS 13433:13	0	1 per 15 000m^2						
Filtration Geotextile										
	Thickness (at 2kPa)	SANS 9863:13	0.8mm	1 per 15 000m^2						
	Mass per	SANS 10221:07	200g/m2	1 per 15 000m^2						
	Tensile Strength – weaker direction	SANS 1525:13	11kN/m	1 per 15 000m^2						
	CBR Puncture Strength	SANS 12236:13	2	1 per 15 000m^2						
	Puncture Resistance	SANS 13433:13	23	1 per 15 000m^2						
Leachate Collection System										
a) Aggregate	Layer Thickness	Survey or Excavation and Measure	150 mm -200 mm per Design Drawing	1 Test per 1000m2						
	Grading	SANS 3001-AG:2014-Sive analysis	38-53 mm with 2% less than fines							
	Aggregate crushing value (ACV)	SANS 3001-AG10 Aggregate Crushing Value		1 Test per 10 000m2						
	Flakiness index	SANS 5847 : Flekiness Index	Max 35%	1 Test per 10 000m2						
b) HDPE Pipe	Crushing Strength	ASTM D2412		Test Every Pipe Lenghts						
	Orifice Size	Measure/Inspection	10 mm	Min of 1 hole per meter						
	Slope	Survey	Min 2%	As per Detailed Design Drawings						
Filter Geotextile	Mass per Unit Area									
	Maximum Opening Size (o95)	ASTM D4751	max 0.423mm	40 000m2						
	Permittivity	ASTM D4491								
	Trapezoid Tear Strength	ASTM D 4533	0.8x1.2 kn	7500 m2						
	CBR Puncture Strength	ASTM D 6241	1.2 kn	7500 m2						
	Apparent Opening Size	ASTM D 4751	max 0.423mm	40 000m2						
	Ultraviolet Stability	ASTM D 7238	65%	1 per site/ every 40 000kg						
	Overlap	Inspection	Min of 150 mm	All Overlaps						
Pioneer Waste layer										

ANNEXURE C –

DESIGN DRAWINGS

Refer to Design Report Drawings

ANNEXURE D –

STANDARD SPECIFICATIONS

(SANS 1200; 1526; 10409(2020); GRI FOR GCLS AND GT CUSHION LAYERS; ASTM D SERIES FOR AND GC3 AND GN2 FOR GEOCOMPOSITES ETC.)

Standard Specification	**Description**
SANS 1200 DE	Small earthworks
SANS 1526 (2015)	HDPE
SANS 10409 (2020)	Design, selection and installation of geomembranes
SANS 1083 (2014)	Aggregate
BS8007	Concrete water retaining structures
GRI-GN2 and GRI-GC13 (2012)	Joining and attaching Geonets and Drainage
EPA 9090, ASTM D5747 and ASTM D5721	Durability
ASTM D7007 and ASTM D6747 or D8265	Form of electric leak location survey to be undertaken post placement of cover material
ASTM D7953	Form of electric leak location survey to be undertaken on exposed liners

ANNEXURE E –

PARTIES INVOLVED IN THE CQA IMPLEMENTATION

COMPANY NAME	**REPRESENTATIVES NAMES**	**EMAIL ADDRESS**	**CONTACT NUMBER**
Owner / Operator			
Project Manager			
Design Engineer			
CQA Monitor			
Geosynthetics Manufacturer			
Geosynthetics Installer			
Earthworks Contractor			
Independent CQA Laboratory			

Appendix E

Operating Plan

Document Title:	Kusile 60yr ADF Operations and Maintenance Manual
Eskom document no.:	xxxx
Contractor document no.:	366-513519
Document type:	Operations and Maintenance Manual
Contractor Name:	EPCM
Revision no.:	0.2
Prepared by	KP Matulovich
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Project Title: Kusile 60yr ADF Operations and Maintenance Manual

TABLE OF CONTENTS

SECTION **PAGE**

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

This Operations and Maintenance (O&M) handbook serves as a comprehensive guide for the efficient operation of the Kusile 60 Year ash disposal facility while ensuring cost-effectiveness. Its primary purpose is to clarify the rationale behind design assumptions and decisions to all personnel engaged in the ash disposal facility's operation.

Initiating with an overview of the underlying philosophy governing the ash dump's various components, the Operations Handbook subsequently outlines the operational and maintenance requirements for these components. It delves into pollution control methodologies, monitoring and maintenance procedures, as well as rehabilitation requisites. The document concludes by summarising the legal and safety aspects associated with dry ash disposal.

It is anticipated that this manual will undergo periodic updates and revisions throughout the facility's lifespan, adapting to evolving circumstances and technological advancements. To facilitate this ongoing process, consistent communication between design and site staff is imperative, ensuring that all stakeholders remain well-informed about the continuous development of the ash dump.

1.1 TERMS OF REFERENCE

Eskom Holdings SOC Ltd. (Eskom) has enlisted the professional services of EPCM Holdings for the Construction Tender Design of the 60 Year Ash Disposal Facility (60yr ADF) at the Kusile Power Station.

The power station is set to adopt dry ashing facilities for ash disposal, as studies indicate that the existing ash/gypsum co-disposal facility, granted Environmental Authorization alongside the power station in March 2008, lacks the capacity to accommodate the anticipated ash volume during the station's operational life. The airspace capacity of the existing co-disposal facility is expected to diminish to less than 10 years if both gypsum and ash are deposited on the facility.

To address this, an additional facility is required to manage ash disposal for the 60-year design life of the station. The envisioned facility will incorporate various components, including:

- a conveyor belt system for ash transportation,
- a barrier containment system,
- contaminated and clean water management systems,
- site services,
- office facilities for operational contractors,
- and auxiliary supporting systems such as Dust Suppression, Irrigation, Electrical, Control, and Instrumentation.

Following the completion of the first phase of the ADF, ash and gypsum waste streams will be permanently separated, and the existing co-disposal facility will be exclusively used for gypsum storage throughout the 60-year design life.

Eskom's goal is to construct and commission the proposed ADF to handle ash generated by the power station over its 60-year design life. Project execution aims to minimize co-disposal at the existing facility, preserving its capacity for gypsum storage over the power station's design life.

The new facility's design adheres to the Integrated Environmental Authorization from the Department of Environmental Affairs (DEA) for the new ash area and Water Use Licence requirements, soon to be issued by the Department of Water and Sanitation (DWS). It is crucial that construction and operations comply with these requirements.

1.2 DESIGN DATA

The design criteria listed in the Table 1 forms the basis for the design of the Kusile ADF.

Table 1: Design Criteria

Design parameter description	Value of design parameter	Source of information
Ash Characteristics		
Ash grading		
4.75mm	100%	WSP Golder Lab Testing - 2022
2.00mm	100-99%	WSP Golder Lab Testing - 2022
0.425mm	98-96%	WSP Golder Lab Testing - 2022
0.075mm	87-78%	WSP Golder Lab Testing - 2022

Design parameter description	Value of design parameter	Source of information
0.002mm	5-6%	WSP Golder Lab Testing - 2022
Grading Modulus	0.15 – 0.27	WSP Golder Lab Testing - 2022
Plastic Index	-	WSP Golder Lab Testing - 2022
Ash dry density	800 kg/m^3	J&W Detailed concept design
Ash moisture content	20-33%	Kendal Operations manual
Ash Bulk Density	1193.156 kg/m^3	WSP Golder Lab Testing - 2022
Specific Gravity	2.202 – 2.254	WSP Golder Lab Testing - 2022
Ash angle of repose:		
Upper third	40 degrees	Kendal Operations manual
Middle third	33.7 degrees	Kendal Operations manual
Lower third	22.6 degrees	Kendal Operations manual
Average slope for geometric modelling	30.5 degrees	Determined from survey
Ash permeability	2.1x10^{-5} cm/s to 1.4x10^{-5} cm/s	Results from 10-year co-disposal testing and Civilab Results (2024)
Ash Production		
Annual tonnage/ unit	1 182 600 t/year	J&W detailed concept design
Number of units in total	6	J&W detailed concept design
Total tonnage/ year	7 095 600 t/year/6units	J&W detailed concept design
Phase 1 pre-deposition works construction date		TBD
Operating life of facility	63 years (at 100% production full time)	J&W detailed concept design
Annual airspace required per unit?	1 004 499 m3/year	Quantity estimation based on dry density
Annual airspace required at full production	6 026 997 m3/year	Quantity estimation based on dry density
Total airspace estimated for life of facility	381 961 514 m3	Based on available space
Geometric characteristics		
Overall maximum height	138m	Geometric modelling
Modelled operational side slopes	1V:2H	
Platform ramp slopes	1V:10H	
Modelled rehabilitated side slopes	1V:5H	

Design parameter description	Value of design parameter	Source of information
Maximum bottom stacker back stack	12m	Geometric modelling
Maximum top stacker back stack	12m	Geometric modelling
Conveyor Parameters		
Target system utilisation (Top:Bottom)	50:50 Split	TBC
Top Extendable Conveyor:		
Maximum length	2768m	TBC
Maximum allowable grade	1V:10H	TBC
Bottom Extendable Conveyor:		
Maximum length	2672m	TBC
Maximum allowable grade	1V:10H	TBC
Top Shiftable Conveyor		
Maximum length	1772m	Eskom GTE
Maximum grade	1V:20H	Operations manual
Shift distance	79.50m	Eskom GTE
Ashing reach	92.8m	TBC
Safe edge distance	15m	Stability analysis
Bottom Shiftable Conveyor		
Maximum length	2356m	Eskom GTE
Maximum allowable grade	1V:20H	Operations Manual
Shift distance	79.52m	Eskom GTE
Ashing reach	92.8m	TBC
Safe edge distance	15m	
Stacker machine parameters		
Link conveyor length	50m	Communication between ZJV and Eskom
Stacker boom length	35m	Communication between ZJV and Eskom
Minimum angle between link conveyor and shiftable conveyor	25°	Based on Majuba Stacker System
Slope Stability Analysis		

Design parameter description	Value of design parameter	Source of information
Operational Static (FoS)	1.2	
Final Static (FoS)	1.5	

1.3 Description of Ashing System

1.3.1 Conveyor System

The ash transport process from the power station to the ADF involves the utilization of two transverse conveyor systems (TAC 1&2), conveying the ash in a south-western direction. The ash is discharged onto two new overland conveyor systems (OLC 1&2) at existing transfer house 8. Additionally, new overland conveyor sections will be added from the existing transfer house 9 position, extending the current overland link conveyor system (OLC 1&2). These overland conveyors discharge the ash to both the top and bottom extendable conveyors (EC 1&2) at transfer houses 11 and 12, respectively. The newly installed overland conveyor sections cover a length of approximately 1,377m between existing transfer houses 9 and 12, with transfer house 11 positioned at chainage 1004m from transfer house 9.

At transfer house 12, the bottom extendable conveyor (EC 2) receives ash from the overland conveyors and conveys it upward onto the bottom stacker platform, facilitating discharge onto the bottom shiftable conveyor (SC 2). The extension of the bottom extendable conveyor aligns with the requirements of the bottom shiftable conveyor shift position.

Similarly, for the top stacker extendable conveyor (EC 1), ash received from the overland conveyors at transfer house 11 is conveyed upward onto the top stacker platform for discharge onto the top shiftable conveyor (SC 1). The extension of the top extendable conveyor accommodates the requirements of the top shiftable conveyor shift position. Both the bottom and top stacker shiftable conveyors discharge their loads onto a crawler-mounted stacker machine using a traveling tripper car and link conveyor.

Figure 1 below shows the layout of the conveyor systems:

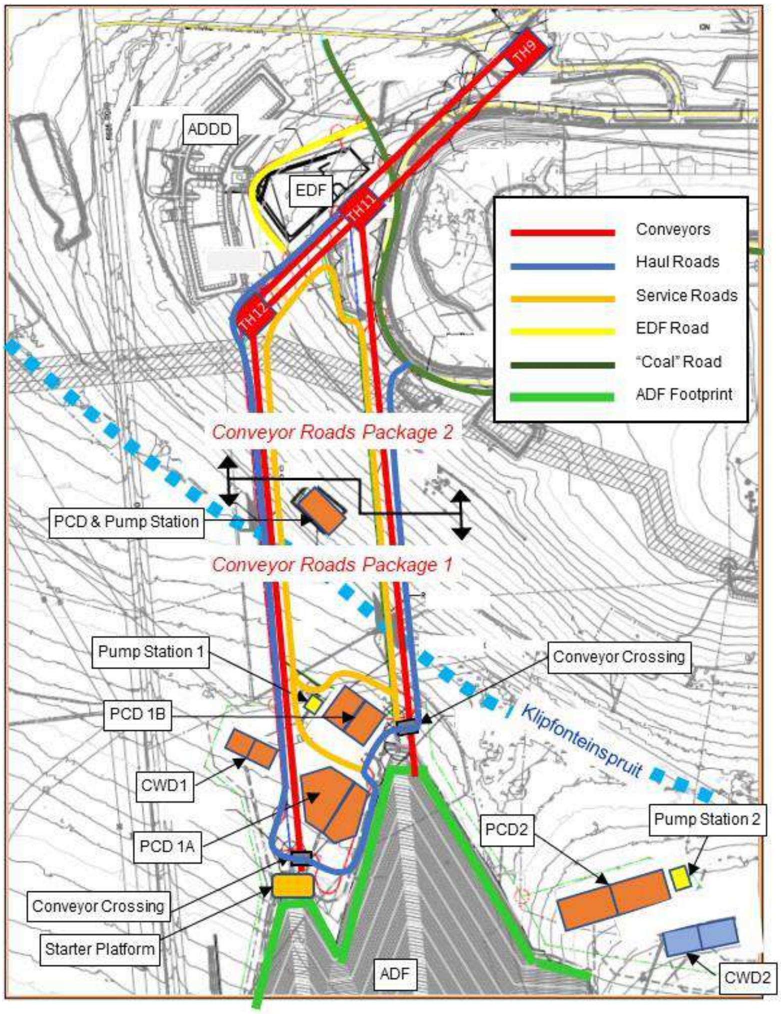


Figure 1: Conveyor Layout

1.3.2 Stacker Machines

Ash is transported along the shiftable conveyors as far as a tripper car which is situated on each of the shiftable conveyor lines. At the tripper car the conveyor belt is raised, and the ash diverted onto a link conveyor. The link conveyor "links" the shiftable conveyor to the stacker machines.

The link conveyor used at the stackers are part of the stacker machine and is capable of slewing and luffing.

The stacker machines receive ash from a tripper car and deposit it in cones and windrows from its conveyor. It consists of separate link and boom conveyors, each of which is capable of slewing independently of the other. This capability plus the extra length of reach of a stacker make it a versatile and cost-effective machine.

1.3.3 Mobile Equipment Operations

Ash should be placed from the stackers as close as possible to the required dump profile. However, there are occasions when mobile equipment is required to operate on the dump.

A dozer is required to move ash to positions outside the reach of the stacker machines, carry out trimming and final profiling of the dump surface, side slopes, and conveyor platforms, as well as shift conveyors, head and tail stations, and transformers during conveyor shifts. A self-propelled or towed roller can be used to achieve a nominal compaction of the dump surface to aid with dust suppression.

2. ASH DUMP DESIGN PHILOSOPHY

2.1 Life & Capacity of The Ash Dump

The ash dump will serve as a disposal site for approximately 382 million m3 of ash during the life of the Power Station. This will provide the Power Station with sufficient capacity until at least 2069.

The actual size of the main ash dump will depend on the lifespan of Kusile Power Station and shall be a function of the load factor during the time of operation. The dump was designed to have a maximum final height of 138m.

2.2 Rate of Development of the Ash Dump

The ADF growth plan provides detail on ash platform development by means of trucking or stacking on a time and shift by shift basis to illustrate the development of the ADF for the duration of the operational life of the facility. The growth plan takes into account constraints such as:

- Stacker system utilisation:
- Mechanical constraints of the conveyors and stacker systems;
- Initial pre-deposition works;
- Ramp up production of the power station;

- Ultimate ash production; and
- Decommission time.

The ash is conveyed from the power station to the ADF via two (2) transverse conveyor systems (TAC 1&2) in a south western direction and discharges onto two (2) new overland conveyor systems (OC 1&2) at existing transfer house 8. The new overland conveyor sections will be installed from existing transfer house 9 position as an extension of the existing overland link conveyor system (OLC 1&2). The overland conveyors discharge to the top and bottom extendable conveyors (EC 1&2) at transfer house 11 and 12 respectively. The new overland conveyor sections to be installed are approximately 1 377m in length between existing transfer house 9 and 12 at the conveyor head end, with transfer house 11 located at chainage 1004m from transfer house 9.

The bottom extendable conveyor (EC 2) receives ash from the overland conveyors at transfer house 12 and conveys the ash up onto the bottom stacker platform in order to discharge onto the bottom shiftable conveyor (SC 2). The bottom extendable conveyor is extended on the top of the stacker platform as per the requirement of the bottom shiftable conveyor shift position.

Similarly, for the top stacker extendable conveyor (EC 1), which receives ash from the overland conveyors at transfer house 11, the ash is conveyed up onto the top stacker platform in order to discharge onto the top shiftable conveyor (SC 1). The top extendable conveyor is also extended on the top of the stacker platform as per the requirement of the top shiftable conveyor shift position. The bottom and top stacker shiftable conveyors both discharge onto a crawler mounted stacker machine utilizing a traveling tripper car and link conveyor.

A total of 382Mm3 of ash will be deposited on the ADF footprint, with a maximum height of 138m.

2.2.1 **Pre-shift dump development**

The pre-shift deposition works essentially consists of the construction and operational works that takes place before parallel shifting of the shiftable conveyor systems. The first phase of pre-deposition is for the first five (5) years of ash deposition and consist of:

- ADF pre-deposition construction activities such as topsoil stripping, starter embankment construction, base and clay liner preparation, geomembrane placement and drainage layer installation;
- Conveyor corridor civil works and Stacker Erection Platform 2 (SEP 2) construction; and
- Conveyor and stacker assembly in order to convey ash to the ADF.

A description of the work required for the pre-deposition dump development is best explained referring to the following:

2.2.2 General Dump Development

The top shiftable conveyor is shifted at an approximate constant distance of 79.5m which is acceptable as the crawler mounted stacker machine's reach is 92.8m which includes the cone bottom width of 7.8m for a 6.5m high cone. The safe edge distance for the top stacker machine is 30m for the extension section of the top stacker platform and only once parallel shifting commences is 15m acceptable. The stacker machine can only be as close as 23.5m from the shiftable conveyor.

The shiftable conveyor length varies between 536m and 2356m in length and has a maximum grade of 1V:20H during the operating life of the facility. The shift duration is determined by the ash production split and the shiftable conveyor system's reach and length. The shift duration for the top shiftable conveyor system varies between 10.0 months and 2.3 years depending on its position during the operating life of the facility.

The total volume of ash placed with the front and back stack methodologies of the bottom conveyor system EC2 is 194.4Mm^3 while the volume of ash placed with the top conveyor system EC1 is 187.5Mm^3.

2.3 Slope Stability

The stability assessment of two sections as indicated on Figure 2 were carried out. The layout indicates a valley to the north of the ADF which falls outside of the ADF footprint and is not considered. The valley indicated on the drawing runs underneath the ADF and is the valley referred to in the sections below. The top stacker platform extension is towards the east from its initial position on the western side of the site.

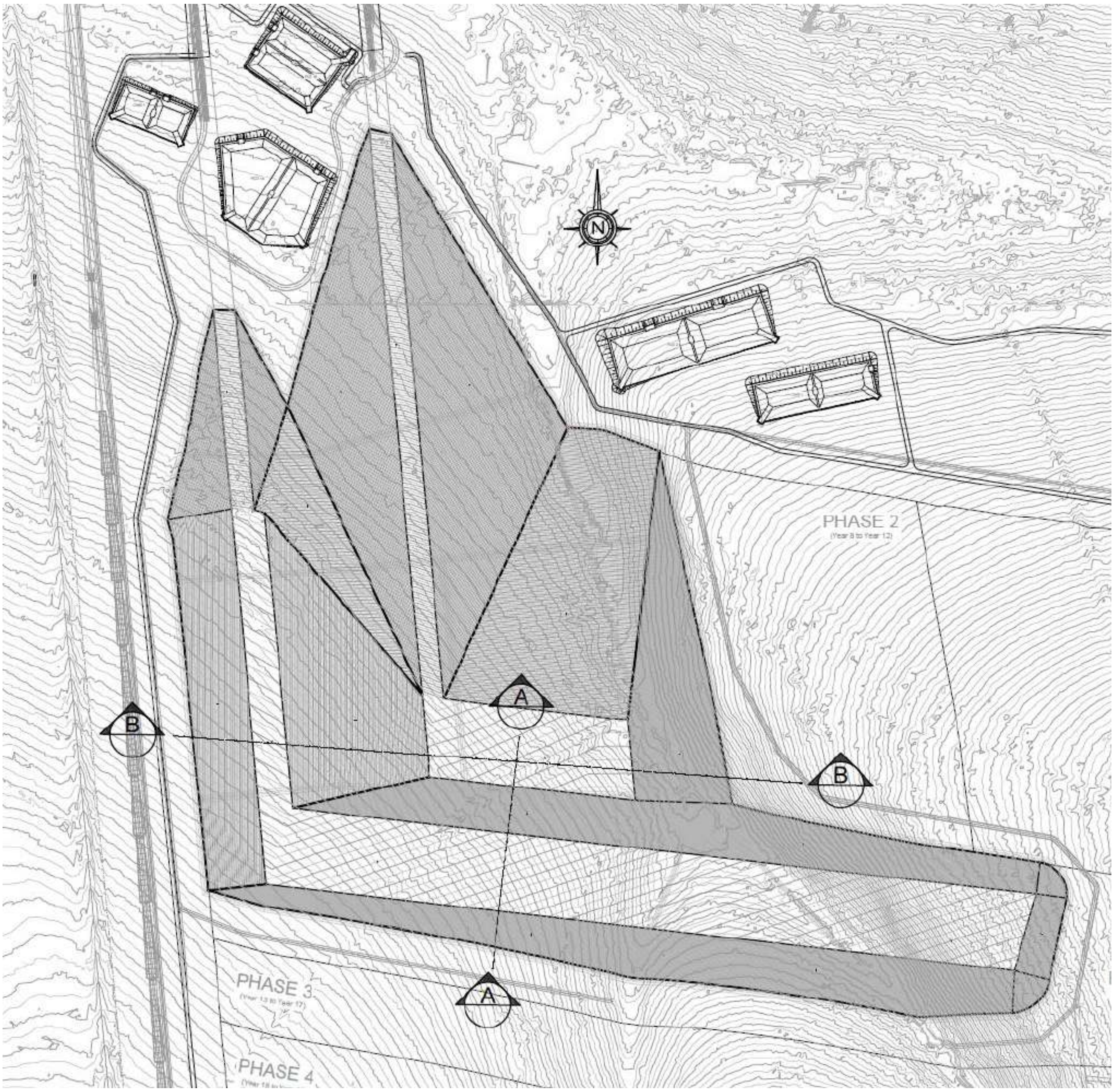


Figure 2: Layout Showing Stability Section Lines

The sections analysed were deemed to be representative of the critical scenarios during operation and final shaping of the ADF with:

- Section AA: Typical steep working face at a slope of 1:2 with stacking surcharge on edge
- Section BB: Longest steep slope from top stack platform to base at valley area.

Other operational sections have been analysed, but only the above, critical sections have been reported on as these are representative of the overall stability.

2.3.1 **Available information**

The following information was reviewed as part of the stability assessment:

- Final geotechnical investigation report prepared Envitech and EPCM which included:
  - Laboratory testing results on in-situ material;
  - Logs of in-situ material.
- Proposed ADF geometry (as per basic engineering level design); and
- Reference paper on the properties of dry dumped fly ash (Fourie et. al, 1997).

2.3.2 **Assumptions**

This section summarises the critical assumptions made as part of the stability assessment:

Water Levels

- Current and post-closure static water tables were modelled at 300mm above the lined basal level throughout the ADF body.

Material Properties and Founding Conditions

- The material properties defined in terms of a Mohr Coulomb material model are provided in the table below. The material properties are based on laboratory testing carried out by EPCM (2022) and ZJV (Report No.: 1651909-307604-1, dated 16 January 2017) and values taken from literature as discussed below. These values are reflected in Table 2.

Table 2: Material Properties used in the Stability Assessment

Material	Unit Weight γ (kN/m^3)	Cohesion c' (kPa)	Friction Angle ϕ' (degrees)
Ash	8	0	35
HDPE Liner	10	0	8.7
Clay	16.5	0	21.3
Soft Rock	22	500	0
Bedrock	22	1200	0

- The Mohr-Coulomb strength parameters of the ash have been adjusted (with reference to Fourie et. al (1997)) such that a slip surface with a factor of safety of 1.0 is apparent near to the surface of the top two-thirds of the advancing face, which conforms to the method by which the ash is placed (i.e. at or near angle of repose).
- The Mohr-Coulomb strength parameters for the liner material were assessed based on research conducted by Koerner and Narejo (2005), which represents the residual shear strength of an HDPE geomembrane in contact with a saturated cohesive material. In reality, this value will be larger, as it is expected that the interface will not be completely saturated.
- The founding conditions, based on test pit profiles, were taken as a 2 m thick clay layer succeeded by a 3 m soft rock layer. A bedrock layer is present below the soft rock layer. It should be noted that these delineations are considered to be conservative, as stripping during construction will reduce the clay layer depth.
- Ash dumped within the ash dump for the power station’s lifecycle will be similar to that of the characteristics of the ash samples tested.
- The angles at which the ash will naturally settle after dry placement were taken as*:
  - 22.6° for the bottom third of the face
  - 33.7° for the middle third of the face
  - 40° for the top third of the face

Analysis

Some notes on the analysis performed:

- The development of water-filled surface cracks was omitted.
- Surcharge loading from plant and equipment other than the conveyors and stackers were ignored, a distributed load of 85kPa is used at the location of the stacker (loading is based on Majuba power station stacker load)
- A half-sine ratio between the horizontal and vertical inter-slice forces was specified for the Morgenstern-Price method of slices.
- A safe edge distance of 15m was used for the ash stacked and a distributed load of 85kN was used over 20m for the stacker weight on the ADF body.

2.3.3 **Methodology**

Stability analyses were carried out using the following software.

GEO5 2023, a slope stability program which computes the stability of slopes and embankments with circular or polygonal slip surfaces. In terms of polygonal or non-circular slip surfaces, the programme can perform the Sarma, Spencer, Janbu or Morgenstern-Price, Shahunyants and ITF methods. We have used the Sarma and Morgenstern-Price methods.

The Sarma (1979) method falls within a category of general sliced methods of limit states. It is based on fulfilling the force and moment equilibrium conditions on individual blocks. The blocks are created by dividing the region above the potential slip surface by planes, which in general have a different inclination.

Morgenstern-Price (1967) is a general method of slices developed on the basis of limit equilibrium. It requires satisfying equilibrium of forces and moments acting on individual blocks. The blocks are created by dividing the soil above the slip surface by vertical dividing planes.

The stability is expressed in terms of a Factor of Safety (FoS) against failure, which can be defined as follows for static loading conditions (taken from the SAICE Code of Practice on Lateral Support in Surface Excavations, 1989):

- FoS < 1,0 - The stability is inadequate, and failure is imminent.

- 1,0 < FoS < 1,3 - Stability is marginal.
- FoS > 1,3 - The short term / operational stability is acceptable.
- FoS > 1,5 - The long term / closure stability is acceptable.

2.3.4 Stability Analyses Results

The results of the stability analyses are given in the table below.

Table 3: Stability Analysis Results

		Global	**Figure reference***
Analysis number	**Operational**		
1.	Section AA Initial – Block failure upper working failure	1.43	Figure 1
2.	Section AA Initial – Composite failure surface	1.45	Figure 2
3.	Section BB East – Working face block failure	1.29	Figure 3
	Post-closure		
4.	Section BB Closure – Lower Platform	2.50	Figure 9
	Section BB Closure – Upper Platform	3.41	

* The figures presenting the failure surfaces are attached in the Design Report 366-511915.

2.3.5 Discussion

The FoS of slip surfaces intersecting the ash stacking machinery are within the upper limit of the marginal stability category (close to 1.5) or satisfy operational stability (Figures 1 to 3). The stability is, therefore, considered to be marginal and will require continuous monitoring. The dump profile has been shown to work in practice at Kendal Power Station and it is expected that the FoS of the dump should increase as time progresses, due to the cementation phenomenon engendered by the free lime content in the ash. Additional to this, some shear strength gain will be realised through matric suction caused by the partially saturated conditions experienced in the ash.

The FoS post-closure is well above 1.5 (Figure 9) and the rehabilitated profile is appropriate for closure.

For the conditions analysed in this analysis, the ADF is considered to be safe, although only marginally in certain scenarios. It is essential that safe edge distances are adhered to by the conveyor stackers.

Should any additional information become available on any pertinent aspects of the ADF (machinery changes, additional data on the ash properties, etc.) the stability analysis should be repeated with consideration of these aspects.

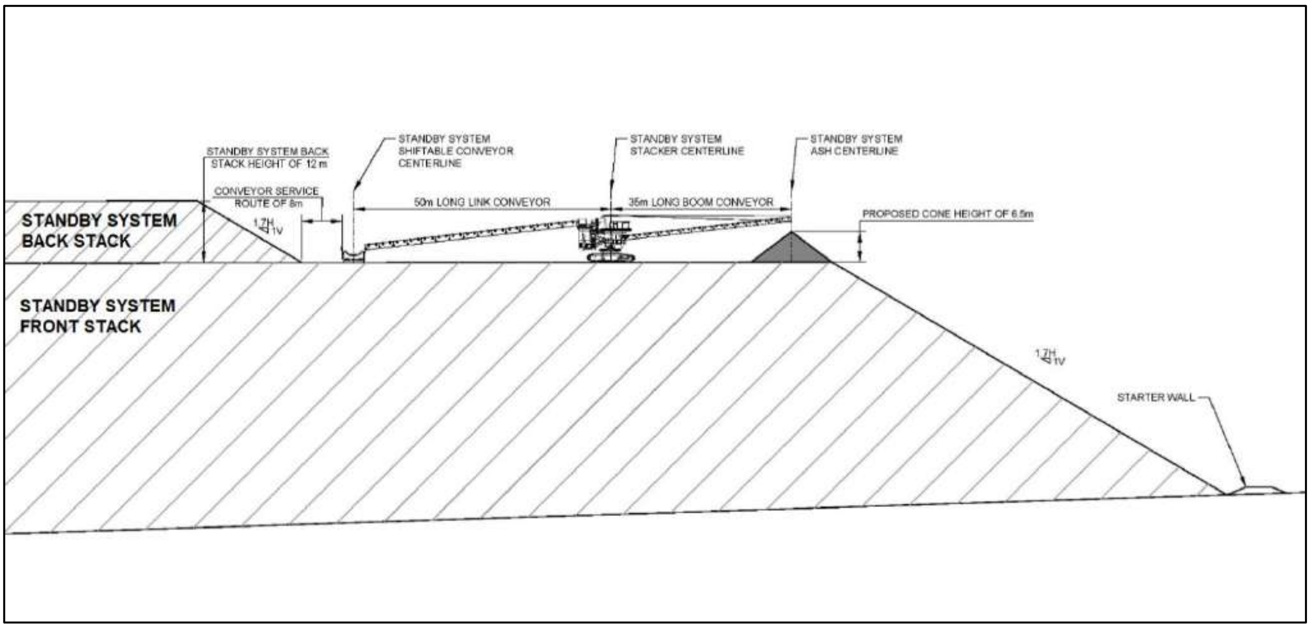


Figure 3: Standby System Frontstack and Backstack Geometry

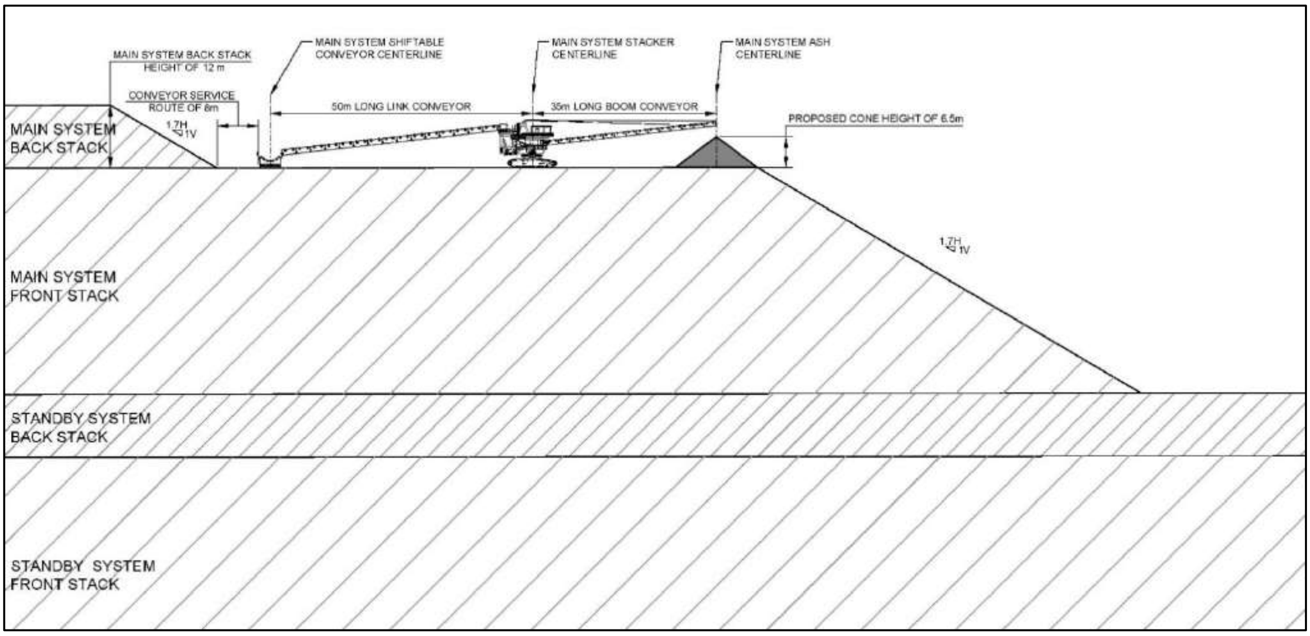


Figure 4: Main System Frontstack and Backstack Geometry

2.4 Ash Dump Handover

In the future ash handling operations may be carried out remote from previously rehabilitated and established areas. It is therefore feasible to hand responsibility for these established areas over to Eskom. To avoid any conflict over these matters a procedure must be set up whereby both Eskom and the ash handling operators agree to the handing over of an area.

It is suggested that a cattle fence be erected to separate areas that have been handed over from the operational areas. The areas in question should be surveyed, measured and inspected prior to their acceptance and full records kept as handover of new areas progresses.

Another approach is to require the ash dump contractor to take over the previously completed areas of the rehabilitated ash dump at the start of his ashing contract and maintain the rehabilitation and earth works facilities for the duration of his contract. Concrete structures would be beyond his expertise and should be maintained separately, however earthworks interfaces to them should be regularly maintained by the ashing contractor, due to the critical nature of these interfaces until they have stabilised. A takeover inspection would be held with Eskom and the contractor at the beginning of his contract where it is the contractor's responsibility to point out all areas not up to standard. Eskom should then place a separate civil works contract to bring these works up to standard before the contractor takes them over. Near completion of his contract, a second takeover inspection is held where it is Eskom's responsibility to point out all areas not up to standard and the contractor is to bring these areas to the same standard of the first takeover upgrading, before the end of his contract.

The benefit of this approach is that the contractor firstly has the necessary personnel and equipment to affect ongoing maintenance upgrading of earth works facilities, as well as being the party spending the most time on the dump, allowing timeous identification of stormwater and erosion related problems, allowing minor repairs to be done before too much consequential damage takes place. This should at the very least include the stabilising rehabilitation areas, if the extent is to be kept to a minimum to reduce the contract risk and costs.

3. ASH DUMP DEVELOPMENT & CONSTRUCTION

All aspects of the design of the ash dump should be based on making maximum use of Stacker machines operational capabilities and reducing the amount of necessary dozing to a minimum. This consists of optimising various geometrical configurations within the constraints of such as:

- Safe edge distance
- Access clearance
- Cone height
- Boom lengths
- Dump height, etc.

If the design is followed, a cost effective and environmentally acceptable ash dump shall be constructed. Adverse circumstances may arise during the life of the dump. These will be managed by consultation between the owners, designers and operators of the ash dump.

Future optimisation should attempt to cover all envisaged eventualities during the life of Kusile Power Station. The designs and drawings have been completed to cover the entire life of the facility. The theory behind the designs and drawings should still be applicable in the ongoing operations but optimisation should be considered should the operator deviate from the growth plan or there is a change in the mechanical equipment.

The advancing ash face for both the bottom and top stacker systems shall be allowed to fall at the angle of repose of the ash.

The ADF is phased in ten main phases (basal coverage growth with eleven phases in total) to allow for deferral of capital across the operating life as well as to prevent degradation of unused pre-deposition infrastructure such the basin liner system and contamination of leachate collection drains.

3.1 Ash Dump Drawing List

Table 4: Drawing List Legend

Abbreviation	Description
REV	Revision
DWG	Drawing
AA	Ashing Archive Drawing
AC	Ashing Construction Drawing
EA	Engineering Archive Drawing
EC	Engineering Construction Drawing
DD	Detailed Design
RIP	Revision In Progress
ASB	As-Built Drawing

Table 5: Phase 1 Drawing List

366-	511841	Locality Plan
366-	511842	Topographical Survey Plan
366-	511843	Site Layout Master Block Plan
366-	511844	60yr General Arrangement Layout
366-	511845	Phase 1 General Arrangement Layout
366-	511846	Attenuation Dam Layout
366-	511847	Phase 1 Fencing Layout
366-	511848	60yr PCD General Arrangement Layout
366-	511849	Phase 1 PCD General Arrangement Layout
366-	511850	60yr CWD General Arrangement Layout
366-	511851	Phase 1 CWD General Arrangement Layout
366-	511852	PCD-1A Layout and Setting Out Data
366-	511853	PCD-1A Sections
366-	511854	PCD-1A Sub-Soil Drainage Layout
366-	511855	PCD-1A Details
366-	511856	PCD-1B Layout and Setting Out Data
366-	511857	PCD-1B Sections
366-	511858	PCD-1B Sub-Soil Drainage Layout
366-	511859	PCD-1B Details
366-	511860	PCD-2 Layout and Setting Out Data
366-	511861	PCD-2 Sections
366-	511862	PCD-2 Sub-Soil Drainage Layout
366-	511863	PCD-2 Details
366-	511864	CWD-1 Layout and Setting Out Data
366-	511865	CWD-1 Sections
366-	511866	CWD-1 Sub-Soil Drainage Layout
366-	511867	CWD-1 Details
366-	511868	CWD-2 Layout and Setting Out Data
366-	511869	CWD-2 Sections
366-	511870	CWD-2 Sub-Soil Drainage Layout
366-	511871	CWD-2 Details
366-	511872	ROAD PCD Layout, Setting Out Data and Sections
366-	511874	Piping and Electrical Reticulation Layout
366-	511875	Road PCD Details
366-	511876	Phase 1 Attenuation Dam Layout

366-	511878	PCD Typical Outlet Details
366-	511879	Silt Trap Type 1 Plan and Sections
366-	511880	Silt Trap Type 4 Plan Layout
366-	511881	Silt Trap Type 4 Sections and Details
366-	511883	River Pipeline Catchments
366-	511884	River Pipeline Layout
366-	511885	Holfonteinspruit Longsection Sheet 1 OF 4
366-	511886	Holfonteinspruit Longsection Sheet 2 OF 4
366-	511887	Holfonteinspruit Longsection Sheet 3 OF 4
366-	511888	Holfonteinspruit Longsection Sheet 4 OF 4
366-	511889	Tributary Longsection Sheet 1 OF 2
366-	511890	Tributary Longsection Sheet 2 OF 2
366-	511891	Distribution Line Diversion Layout
366-	511892	ADF Landfill Liner and drainage details
366-	511893	ADF Landfill Berm Details
366-	511894	ADF Stormwater management cathment details
366-	511895	ADF Stormwater management Plan details
366-	511896	ADF Channels Longitudinal Sections CWD East Temporary
366-	511897	ADF Channels Longitudinal Sections CWD East Temporary
366-	511898	ADF Channels Longitudinal Sections Dirty W East Temporary
366-	511899	ADF Channels Longitudinal Sections Dirty W East Temporary
366-	511900	ADF Channels Longitudinal Sections Dirty W North-East
366-	511901	ADF Channels Longitudinal Sections Dirty W West
366-	511902	ADF Channels Longitudinal Sections Dirty W West
366-	511903	ADF Channels Longitudinal Sections Clean W West
366-	511904	ADF Channels Longitudinal Sections Clean W West
366-	511905	ADF Channels Longitudinal Sections Dirty W North 1 and 2
366-	511906	ADF Channels Longitudinal Sections Dirty W Southwest and Southeast
366-	511907	ADF Channels Longitudinal Sections Dirty W Southwest and Southeast
366-	511908	ADF Channels Longitudinal Sections Dirty W Southeast and Southwest
366-	511909	ADF Channels Longitudinal Sections Temporary Drains SE, Dirty W NW
366-	511910	ADF Channels Longitudinal Sections Temporary Drains SE, NW
366-	511911	PCD and CWD Silt Traps: Structural Reinforcement Details Type 1
366-	511912	PCD and CWD Silt Traps: Structural Reinforcement Details Type 4
366-	511916	Spillway Type 1 for PCDs and CWDs 20m
366-	511917	Spillway Type 2 for PCDs and CWDs 10m
366-	511918	Stream Diversion Pipeline Typical Sections

366-	511919	Attenuation Dam No.1, Layout and Setting out details
366-	511920	Attenuation Dam No.2, Layout and Setting out details
366-	511921	Attenuation Dam No.3, Layout and Setting out details
366-	511922	Attenuation Dam No.4, Layout and Setting out details
366-	511923	Attenuation Dam No.5, Layout and Setting out details
366-	511924	Attenuation Dams By-pass drains details
366-	511925	Attenuation Dam 2 inlet / outlet
366-	511926	Attenuation Dam 3 inlet / outlet
366-	511927	Attenuation Dam 5 inlet / outlet
366-	511928	Attenuation Dams By-pass drains Setting out details
366-	511929	Attenuation Dams 1 and 2 Wall setting out
366-	511930	Attenuation Dams 3, 4 and 5 Wall setting out
366-	511931	Road-Channel Culvert crossing no.1
366-	511932	Road-Channel Culvert crossing no.2
366-	511933	Road-Channel Culvert crossing no.3
366-	511934	Road-Channel Culvert crossing no.4
366-	511935	Base Model Starter Embankment Layout
366-	511936	Basin Excavation Layout
366-	511937	Basin Excavation Setting Out Points
366-	511938	Phase 1 Fill Model Layout
366-	511939	Phase 1 Fill Model Sections
366-	511940	Pre-Deposit Layout
366-	513495	Pre-Deposit Setting Out Points
366-	513496	ADF Leachate Drainage Layout
366-	513497	ADF Sub-soil Drainage Layout
366-	513498	PCD-1A Silt Trap Setting out and details
366-	513499	PCD-1B Silt Trap Setting out and details
366-	513500	PCD-2 West Silt Trap Setting out and details
366-	513501	PCD-2 East Silt Trap Setting out and details
366-	513502	CWD-1 Silt Trap Setting out and details
366-	513503	Channels Setting Out Co-Ordinates
366-	513504	Channel Sections sheet 1
366-	513505	Channel Sections sheet 2
366-	513506	Channels Drop Box West
366-	513507	Channels Drop Box East
366-	513508	ADF Final Fill model
366-	513509	Road-Channel Culvert crossing No. 5

366-	513510	Road-Channel Culvert crossing No. 6
366-	513511	Channel switching detail
366-	513512	Dams complex groundwater cut-off drains
366-	513513	ADF south groundwater cut-off drain
366-	513514	ADF pipeline Junction Boxes Setting Out Details
366-	513515	ADF pipeline Outlet Stilling Basin
366-	513516	Leachate and Sub-Soil Collectors Fitting Schedule
366-	513517	Road PCD Silt Trap 1 Setting Out
366-	513518	Road PCD Dirty water Channels
366-	513522	Stream Diversion Pipeline Junction Box No.4 Plan and Elevations
366-	513523	Stream Diversion Pipeline Junction Box No.4 Reinforcing Plan and Details
366-	513524	Stream Diversion Pipeline Junction Box No.9 Plan and Elevations
366-	513525	Stream Diversion Pipeline Junction Box No.9 Reinforcing Plan and Details
366-	513526	Stream Diversion Pipeline Junction Box No.17 Plan and Elevations
366-	513527	Stream Diversion Pipeline Junction Box No.17 Reinforcing Plan and Details
366-	513528	Stream Diversion Pipeline Junction Box No.19 Plan and Elevations
366-	513529	Stream Diversion Pipeline Junction Box No.19 Reinforcing Plan and Details
366-	513530	Stream Diversion Pipeline Junction Box No.25 Plan and Elevations
366-	513531	Stream Diversion Pipeline Junction Box No.25 Reinforcing Plan and Details
366-	513532	Stream Diversion Pipeline Junction Box No.28 Plan and Elevations
366-	513533	Stream Diversion Pipeline Junction Box No.28 Reinforcing Plan and Details
366-	513534	Stream Diversion Pipeline Junction Box No.34 Plan and Elevations
366-	513535	Stream Diversion Pipeline Junction Box No.34 Reinforcing Plan and Details
366-	513536	Stream Diversion Pipeline Junction Box No.38 Plan and Elevations
366-	513537	Stream Diversion Pipeline Junction Box No.38 Reinforcing Plan and Details
366-	513538	Stream Diversion Pipeline Stilling Basin Reinforcement Plan and Details
366-	513539	Stormwater Channels North Crossing Rebar Layout
366-	513542	Engineered Fill Plan and Sections, Setting Out
366-	513544	Test Pit Positions
366-	513545	ADF Base Sections Sheet 1
366-	513546	ADF Base Sections Sheet 1
366-	513547	Attenuation Dams 2, 3 and 5 Guardrails
366-	513548	Attenuation Dams 2, 3 and 5 Spillway details
366-	513550	Road PCD Tank Layout and Section
366-	513551	Road PCD Reinforcement Layout
366-	513552	Road PCD Bending Schedule
366-	513553	Poad PCD Silt Trap 2 Setting Out

366-	513556	Growth Plan Phase 1 Stage 1 Platform
366-	513557	Growth Plan Phase 1 Stage 1 Section
366-	513558	Growth Plan Phase 2 Stage 1 Platform
366-	513559	Growth Plan Phase 2 Stage 1 Section
366-	513560	Growth Plan Phase 3 Stage 1 Platform
366-	513561	Growth Plan Phase 3 Stage 1 Section
366-	513562	Growth Plan Phase 4 Stage 1 Platform
366-	513563	Growth Plan Phase 4 Stage 1 Section
366-	513564	Growth Plan Phase 5 Stage 1 Platform
366-	513565	Growth Plan Phase 5 Stage 1 Section
366-	513566	Growth Plan Phase 6 Stage 1 Platform
366-	513567	Growth Plan Phase 6 Stage 1 Section
366-	513568	Growth Plan Phase 7 Stage 1 Platform
366-	513569	Growth Plan Phase 7 Stage 1 Section
366-	513570	Growth Plan Phase 8 Stage 1 Platform
366-	513571	Growth Plan Phase 8 Stage 1 Section
366-	513572	Growth Plan Phase 9 Stage 1 Platform
366-	513573	Growth Plan Phase 9 Stage 1 Section
366-	513574	Growth Plan Phase 10 Stage 1 Platform
366-	513575	Growth Plan Phase 10 Stage 1 Section
366-	513576	Growth Plan Phase 11 Stage 1 Platform
366-	513577	Growth Plan Phase 11 Stage 1 Section
366-	513578	General Arrangement Bottom Shiftable Conveyor
366-	513579	General Arrangement Top Shiftable Conveyor
366-	513580	EC1 Long Section
366-	513581	EC2 Long Section
366-	513582	Attenuation Dam 3 Inlet Chamber Reinforcement
366-	513583	Attenuation Dam 3 Outlet Chamber Reinforcement
366-	513584	Attenuation Dam 2 Inlet Chamber Reinforcement
366-	513585	Attenuation Dam 2 Outlet Chamber Reinforcement
366-	513586	Attenuation Dam 5 Inlet Chamber Reinforcement
366-	513587	Attenuation Dam 5 Outlet Chamber Reinforcement
366-	513588	Attenuation Dams 2, 3 and 5 Spillway reinforcement

A. Stacker Ashing Sequence – TBC by Eskom BMH

A 2-cycle ashing operation (see Figure 14) walk an optimised route per shift to place ash which halves the traveling operating and maintenance costs over its lifetime. This is done by also starting with the stacker in front of the conveyor after a shiftable conveyor shift and walking to the end of the system, to start building the Cycle1 frontstack from in-to-out. The cut/fill dozing for the Cycle 1 side slope can be done any time from the Cycle 2 shiftable conveyor shift, which has just been completed, up until the stacker reaches the end of the front stack. If it is done straight after the shift however, the dozer will still be on site and the ash will be less cemented. In addition, the stacker will then not be held up when reaching this point, if additional dozing of ash needs to be stacked.

When reaching the end of the Cycle 1 frontstack, the maximum reach Cycle 1 sideslope stacking is placed from the end of the conveyor (See Figure 9), to form the minimum safe edge distance crest for the next conveyor position head station and stack as much sideslope free ash to minimise future sideslope dozing volumes. This also forms the platform necessary to drive the stacker around the head end to behind the conveyor, with sufficient safe edge distance on the outside of the stacker. From this position behind the conveyor, the remainder of the maximum reach Cycle 1 sideslope ashing is stacked from the end of the conveyor. The final Cycle 1 sideslope is then completed behind the Cycle 1 conveyor, placing additional ash with the stacker where necessary. The Cycle 1 backstack is then stacked from out-to-in and lastly the Cycle 1 radial shift is done with the stacker behind the conveyor at the tail end.

After the Cycle 1 shiftable conveyor shift, the Cycle 2 backstack is then constructed from in-to-out. Similarly, to Cycle 1, the cut/fill dozing for the Cycle 2 sideslope can also be done any time from the Cycle 1 shift, until the stacker reaches the end of the Cycle 2 backstack. If it is done straight after the shift however, the dozer will again still be on site and the sideslope ash will be less cemented. In addition, the stacker will not be held up when reaching this point, if additional dozing ash needs to be stacked.

When the Cycle 2 backstack is completed at the head end, the frontstack starter platform in front of the head station and the maximum reach Cycle 2 sideslope stacking should first be done from behind the conveyor (See Figure 10, position B). The frontstack starter platform is needed for the stacker to stand on later when it walks around the head station to start ashing the frontstack. This will require some additional stacking and dozing to advance the frontstack platform crest sufficiently. This starter platform should be constructed before the Cycle 2 final 1:5 sideslope additional dozing ash is placed, to allow the ash the maximum time to strengthen. The final Cycle 2 sideslope is then constructed behind the conveyor, placing additional ash if necessary. The stacker is then moved around the head station to the front of the conveyor, onto the previously constructed platform (See Figure 10, position A). From this position the maximum sideslope free ashing is again done to the south, to minimise future sideslope dozing and form the minimum safe edge distance crest for the next conveyor position head station. From this point the stacker builds the Cycle 2 frontstack from out-to-in until it reaches the outside edge of the standby system. From here the stacker walks about 1km back down to the shiftable conveyor tail end, to start the Cycle 2 radial shift, with the stacker in front of the conveyor.

The stacker ashing reach general arrangement for the standby and main system can be seen in Figures 5 to 20 below.

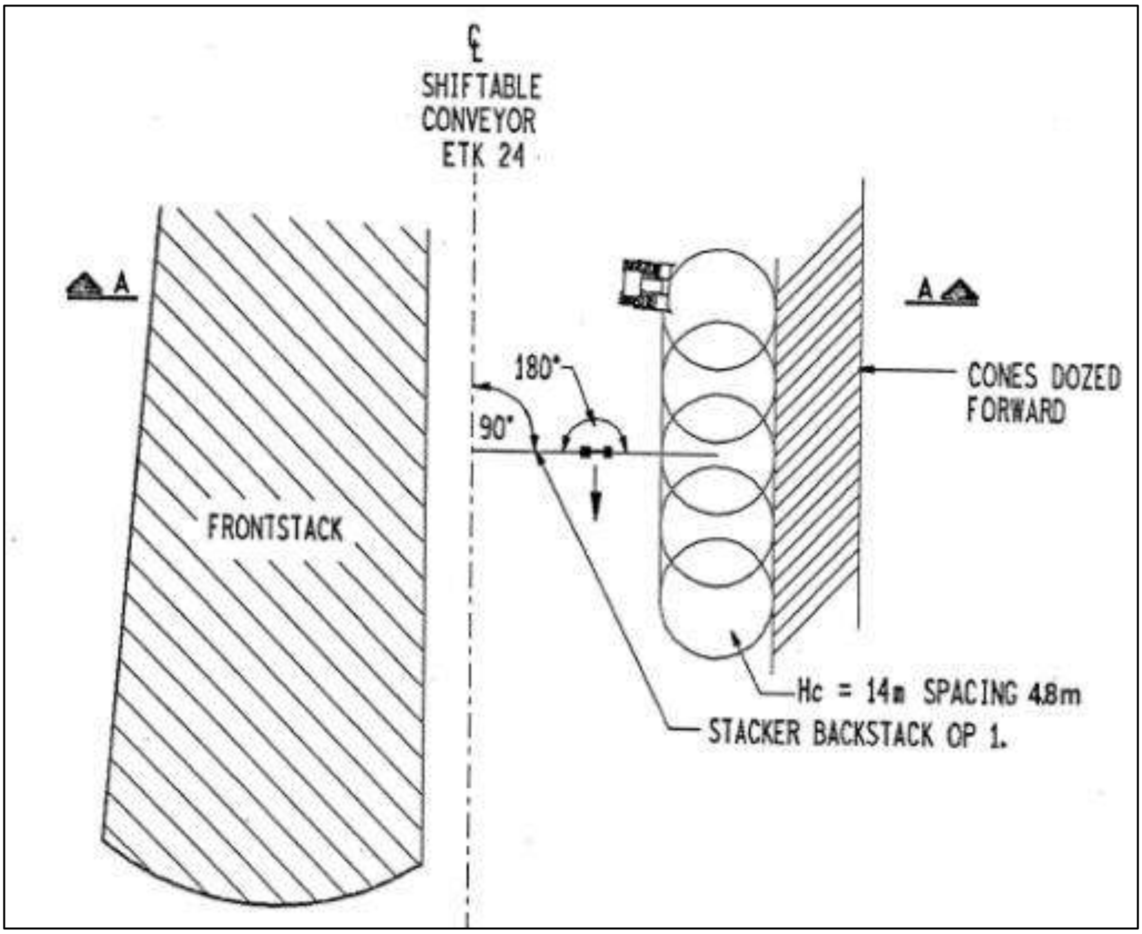


Figure 5: Stacker Backstack Operation 1 – Plan View

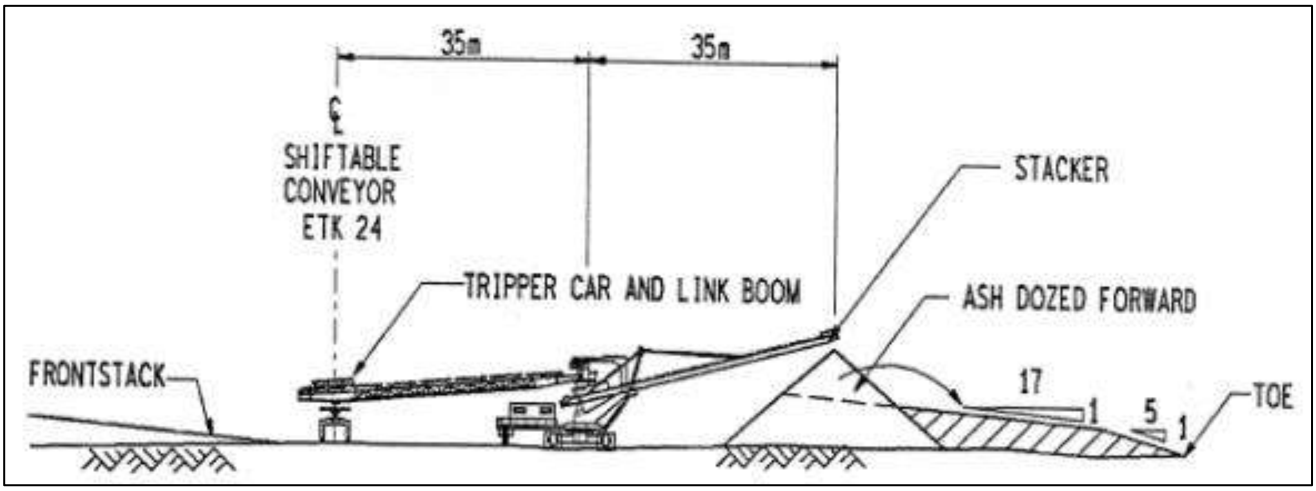


Figure 6: Stacker Backstack Operation 1 – Side View

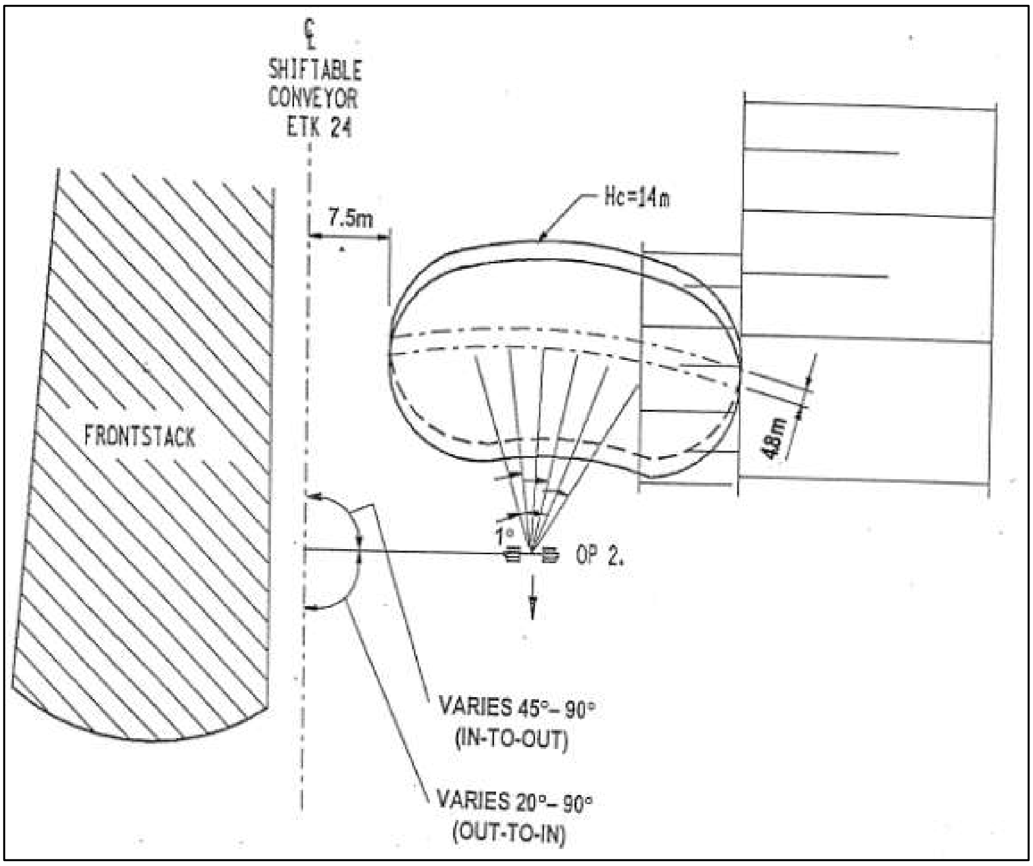


Figure 7: Stacker Backstack Operation 2 (IN-TO-OUT)

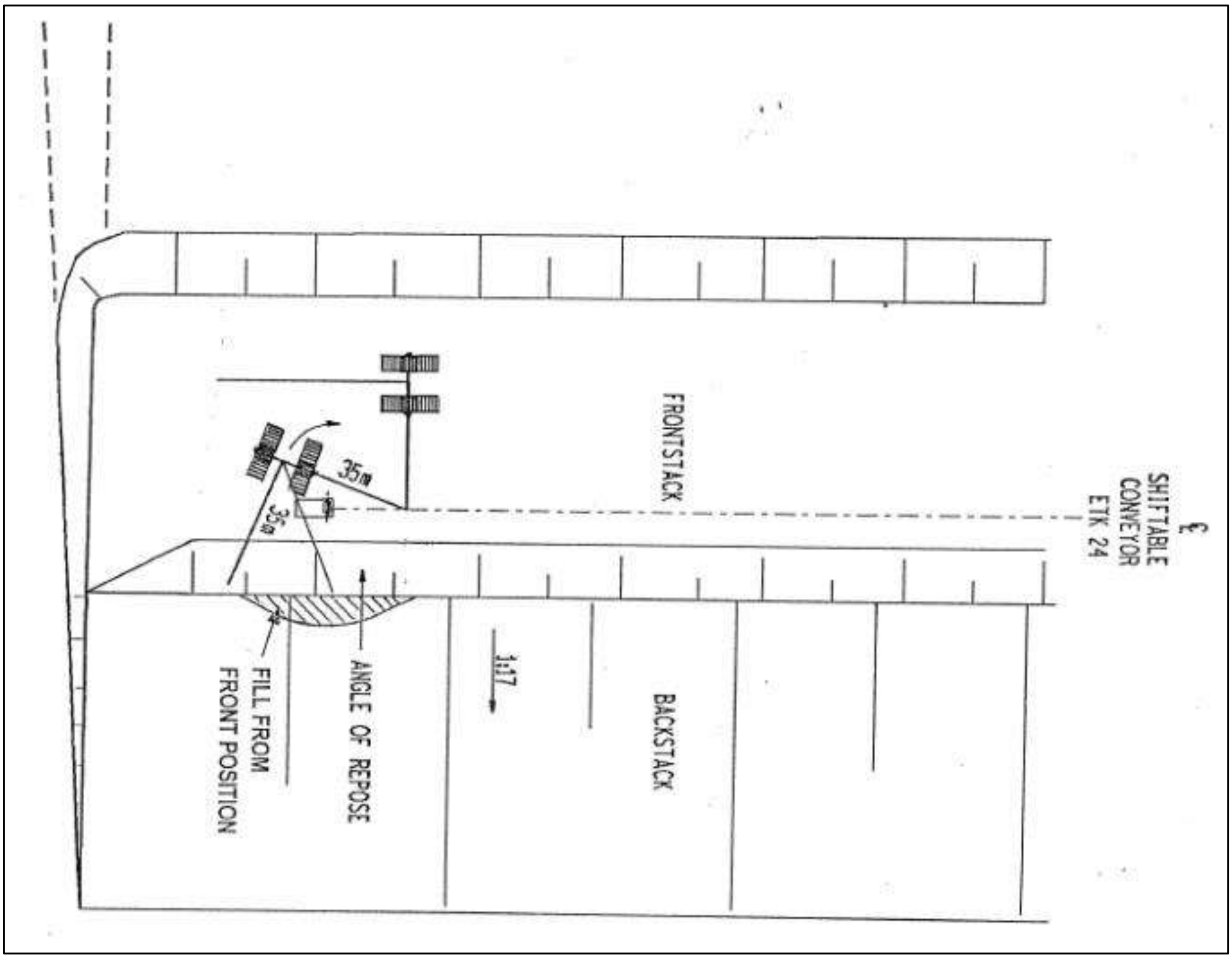


Figure 8: Backstack Filling From in Front of Conveyor

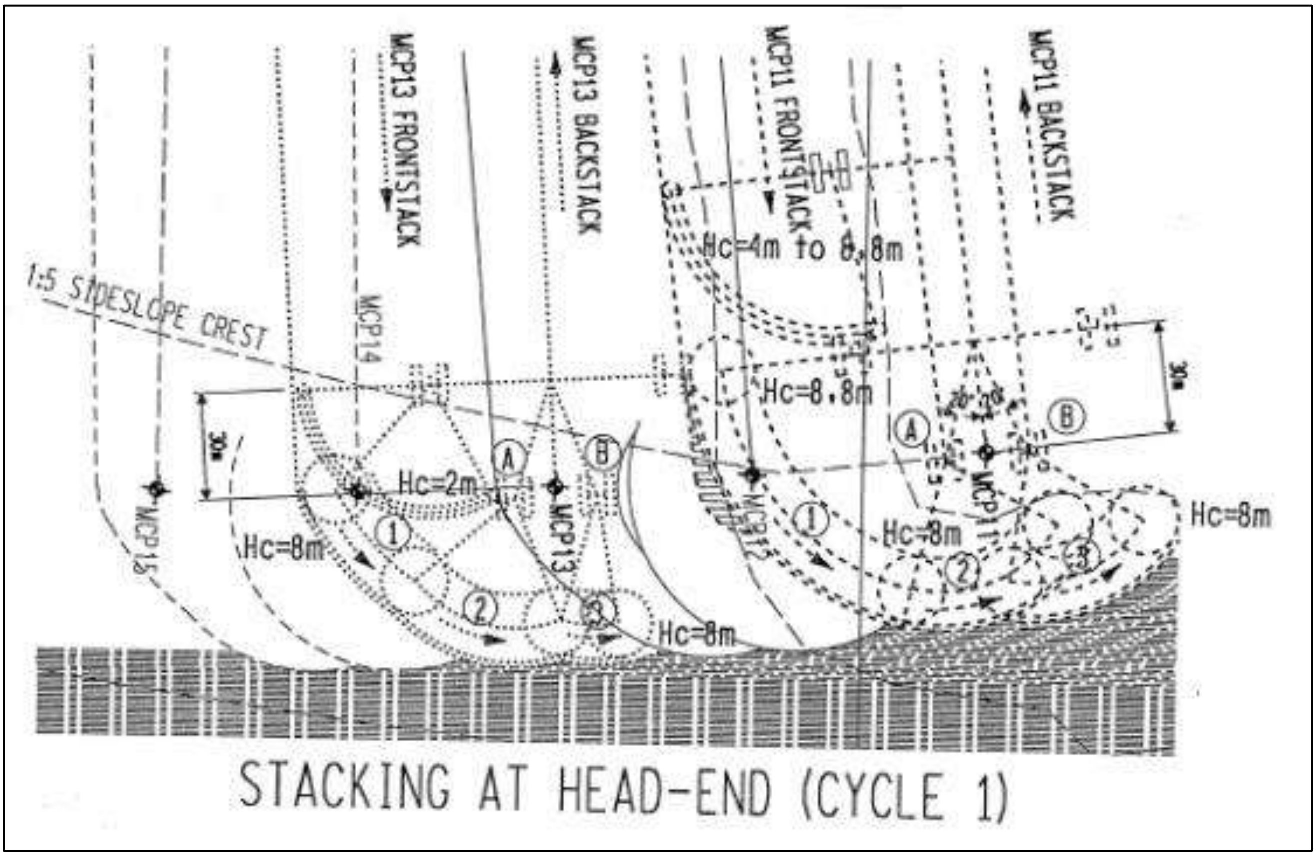


Figure 9: Stacker Ashing Operations At Head End – Cycle 1

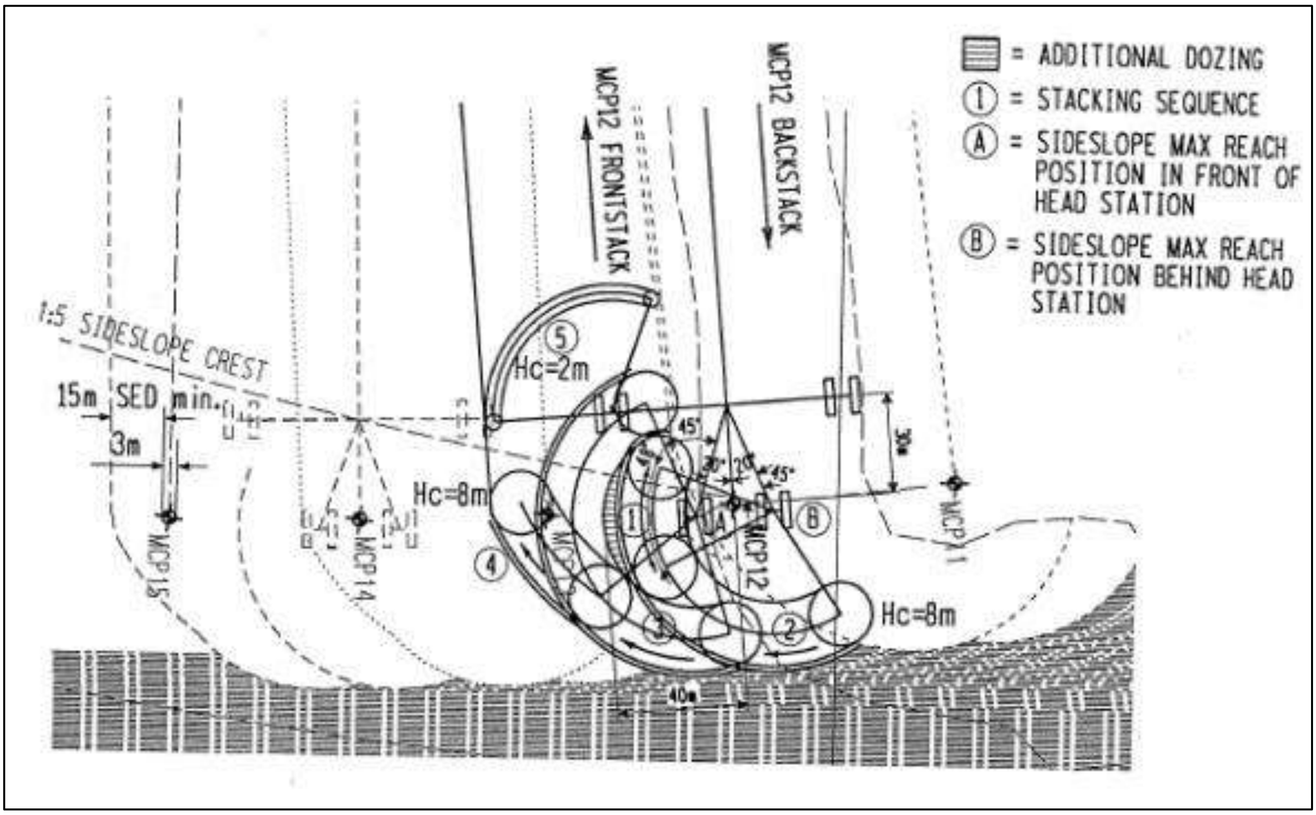


Figure 10: Stacker Ashing Operations At Head End – Cycle 2

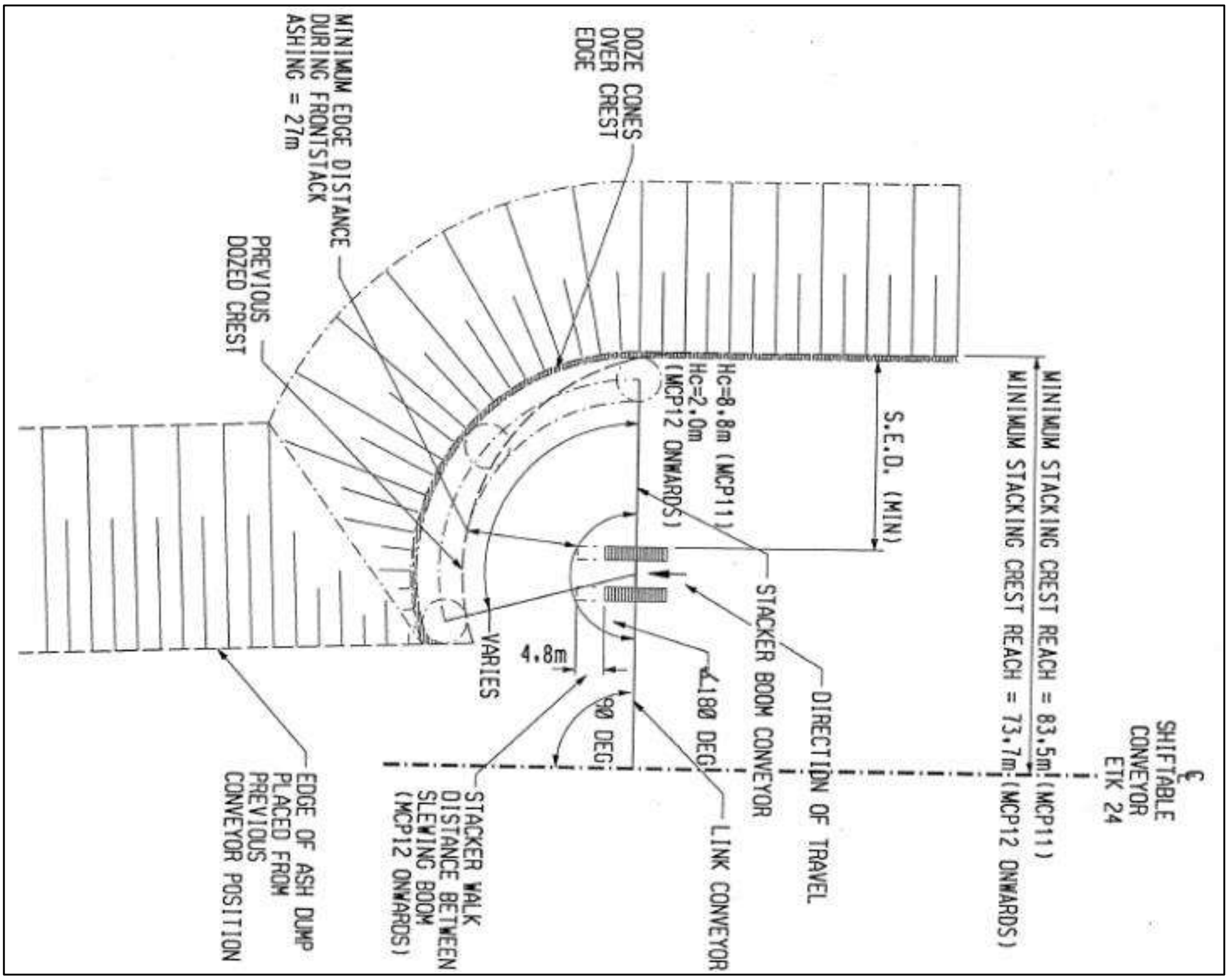


Figure 11: Stacker Frontstack Operation

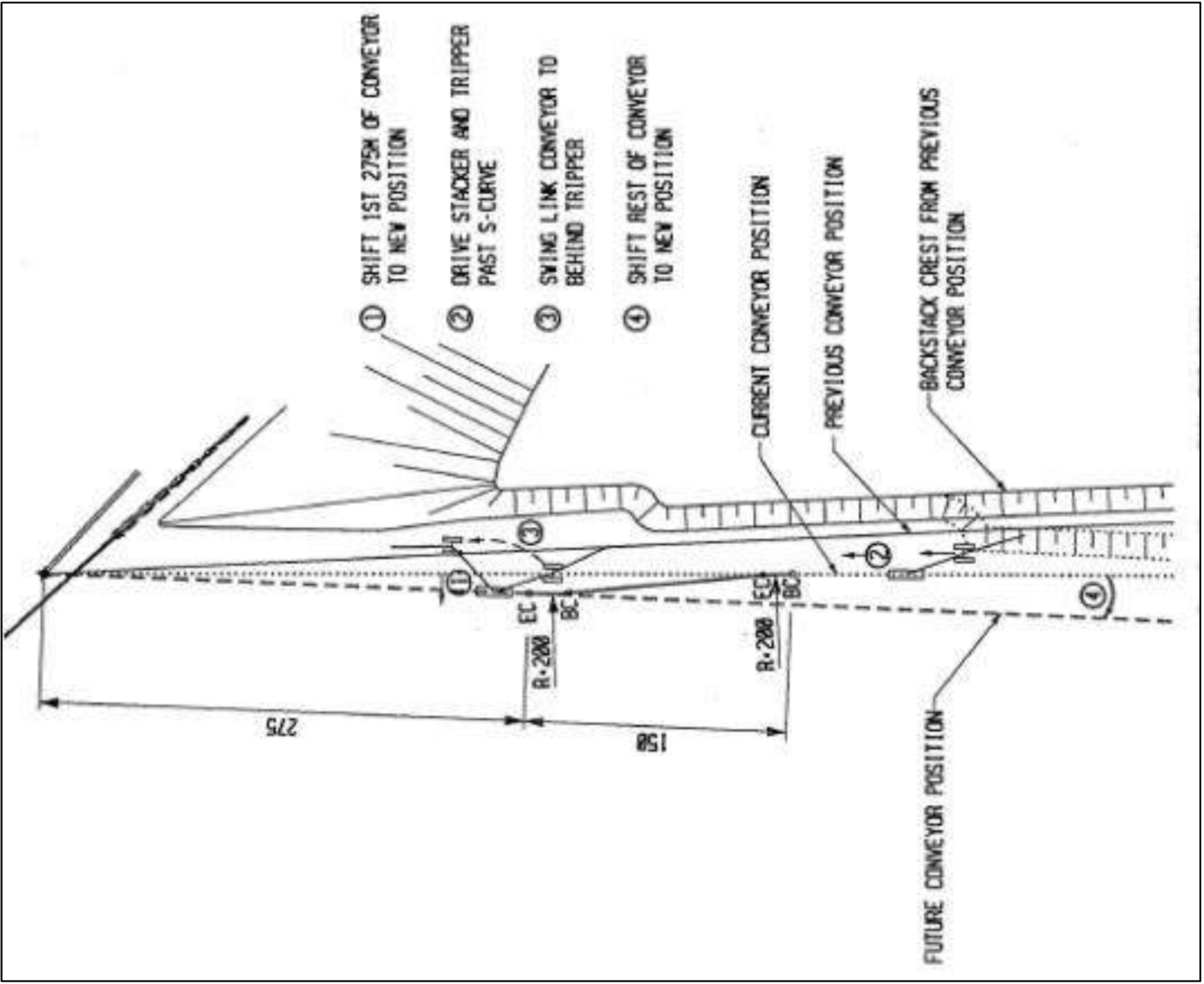


Figure 12: Position of Stacker During Shiftable Conveyor Moves – Cycle 1

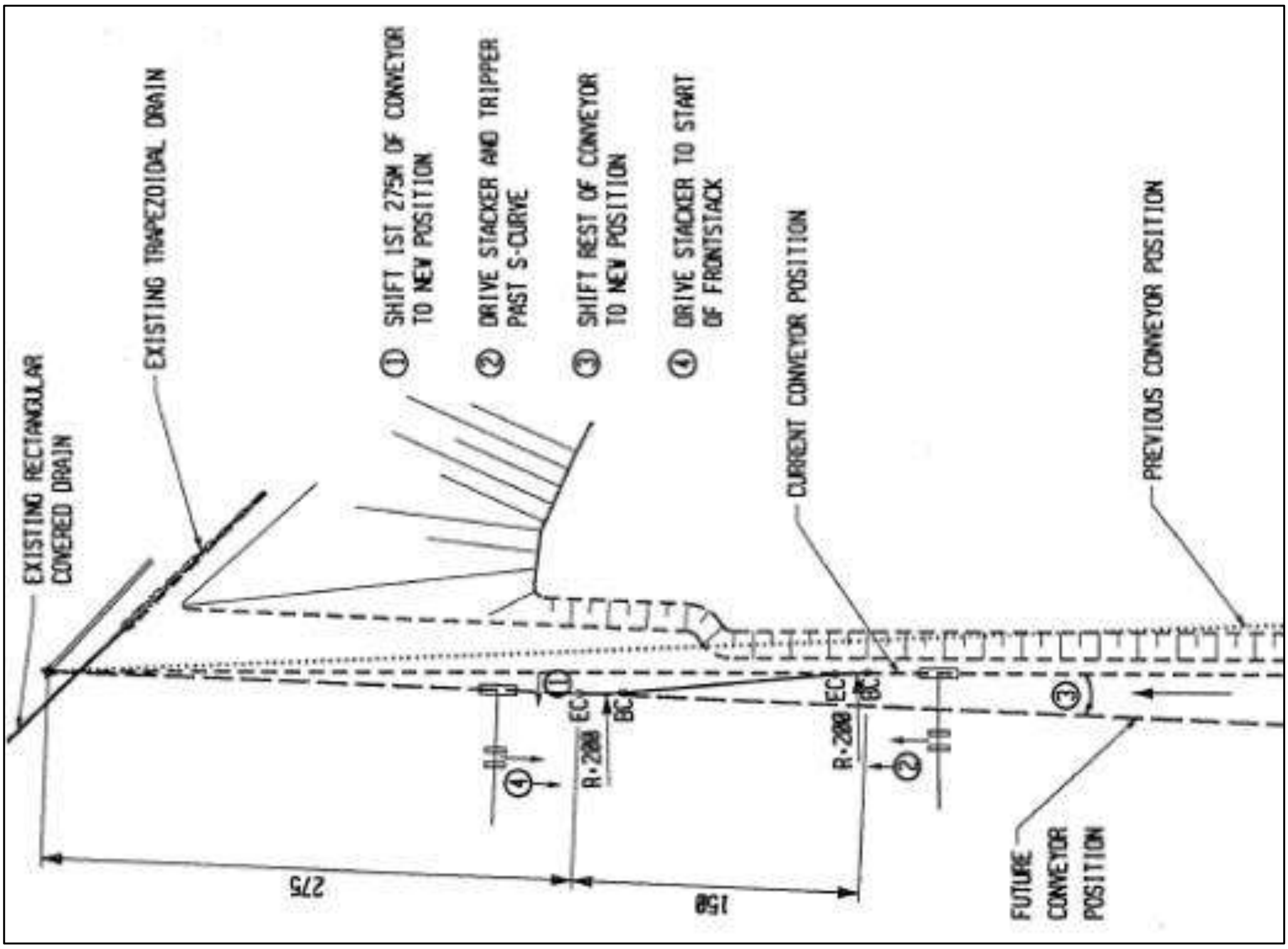
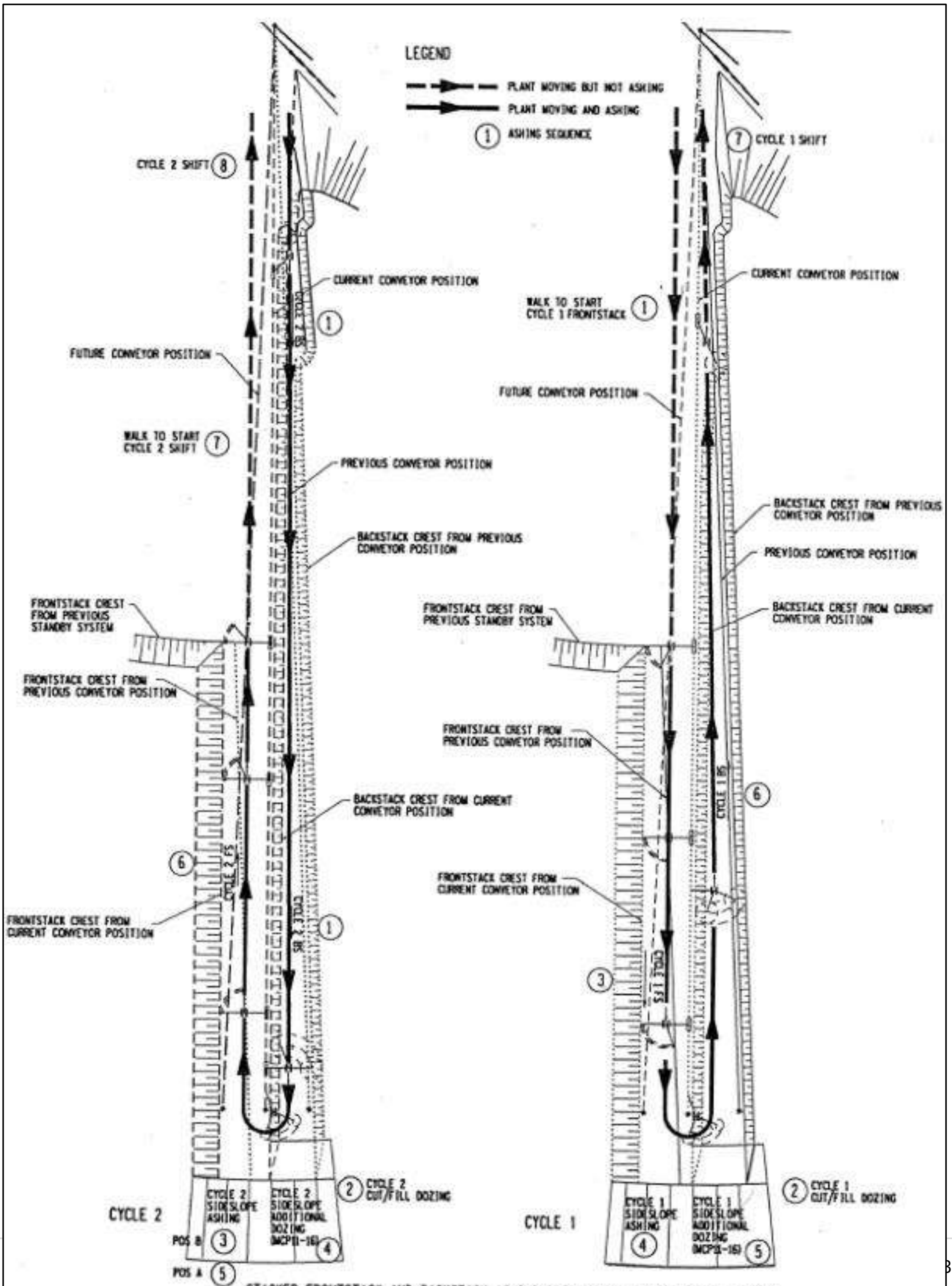


Figure 13: Position of Stacker During Shiftable Conveyor Moves – Cycle 2



STACKER FRONTSTACK AND BACKSTACK ASHING OPERATIONS - TWO CYCLE METHOD

Figure 14: Stacker 2-Cycle Ashing Sequence

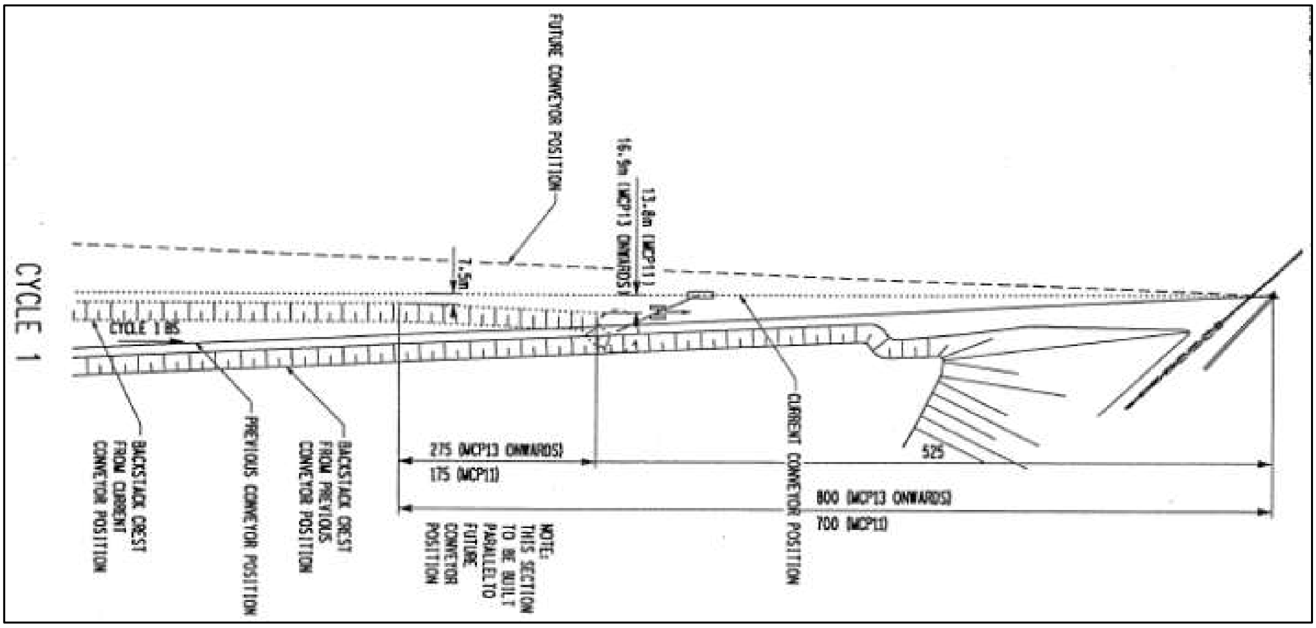


Figure 15: Stacker Backstack Ashing Procedure at Tail End of Conveyor – Cycle 1

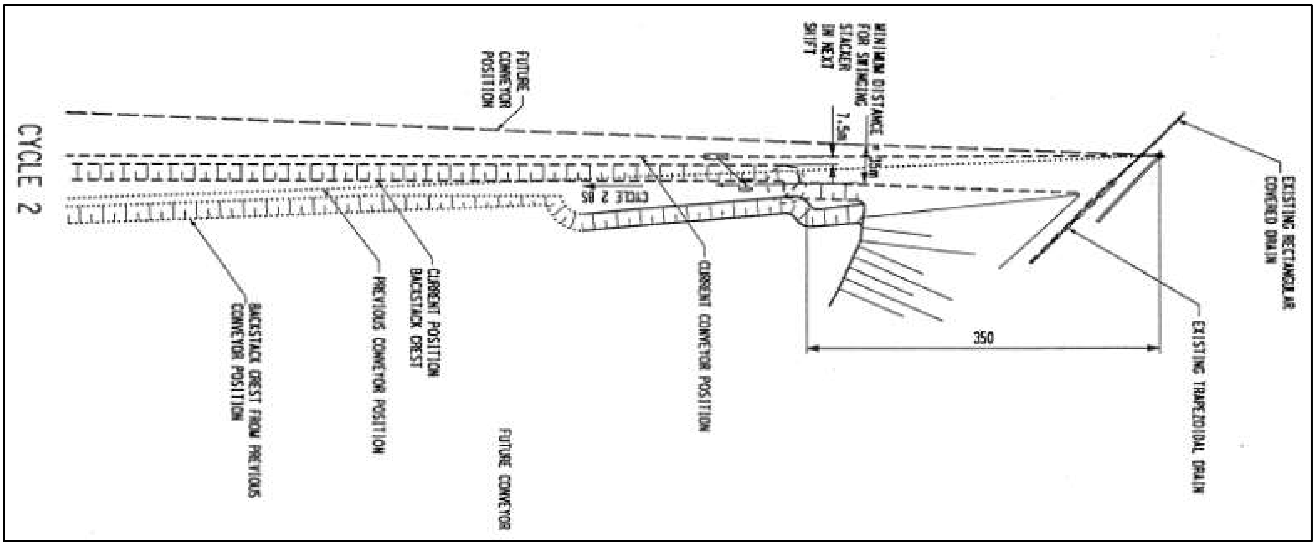


Figure 16: Stacker Backstack Ashing Procedure at Tail End of Conveyor – Cycle 2

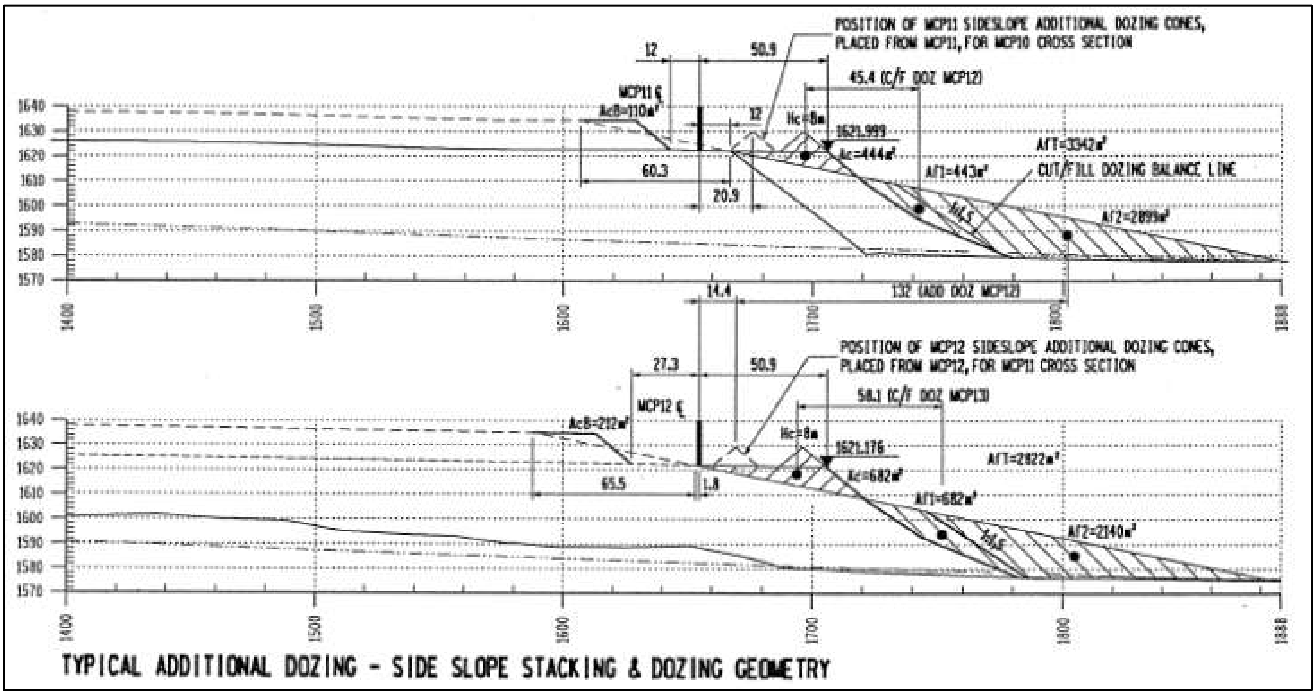


Figure 17: Side Slope Stacking & Dozing Geometry – Additional Dozing

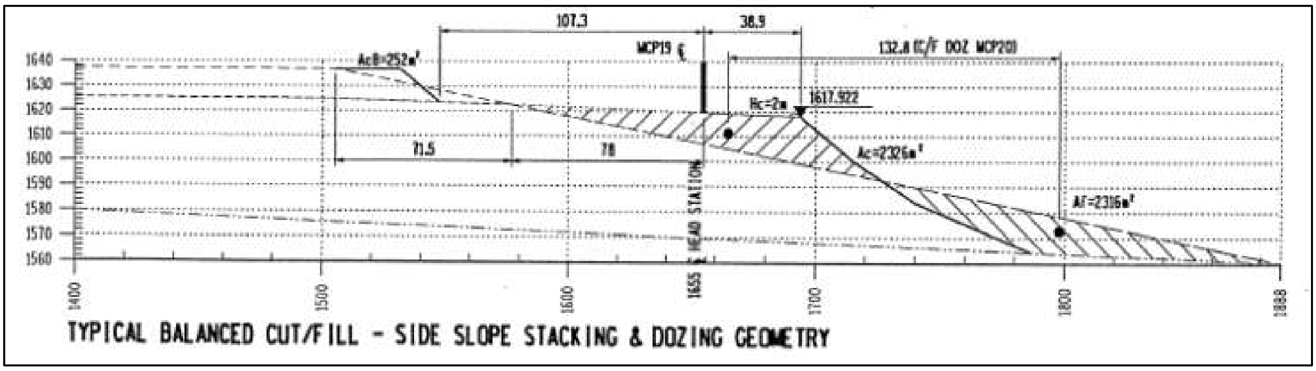


Figure 18: Side Slope Stacking & Dozing Geometry – Balanced Cut to Fill

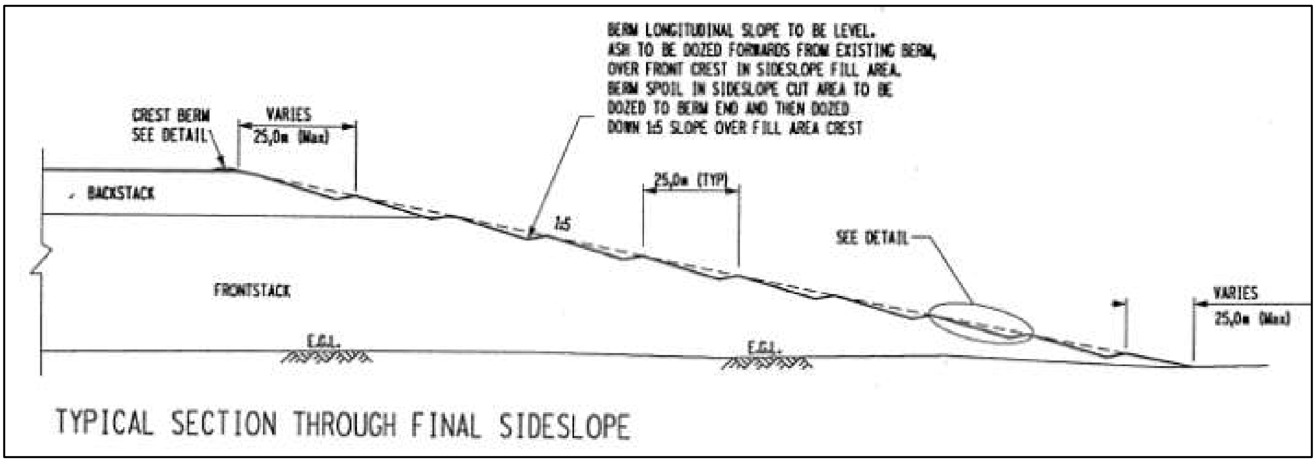


Figure 19: Final Dump Side Slope Geometry

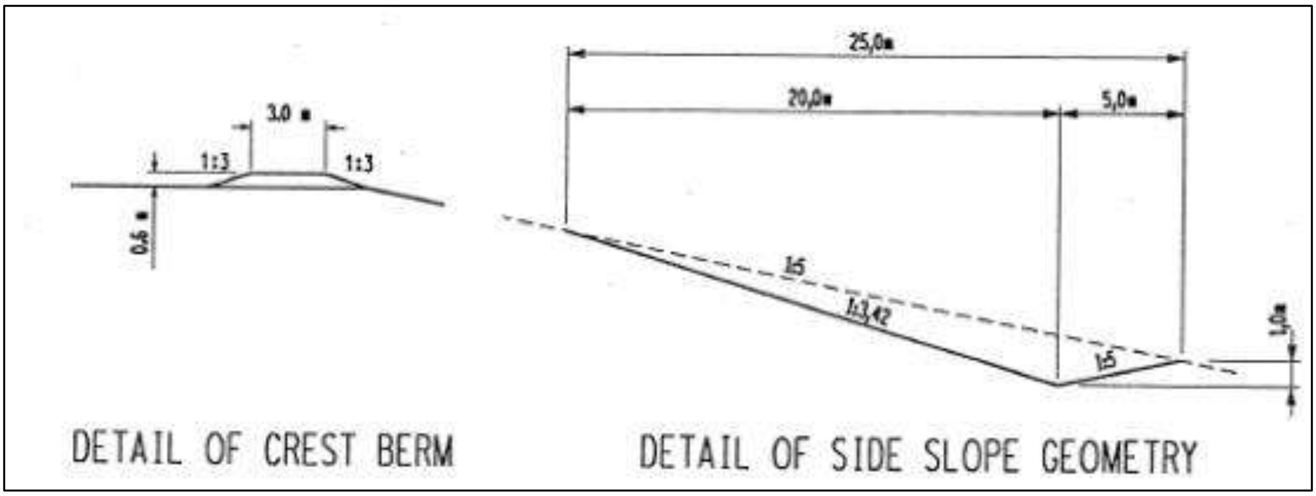


Figure 20: Detailed of Crest Berm & Side Slope Geometry

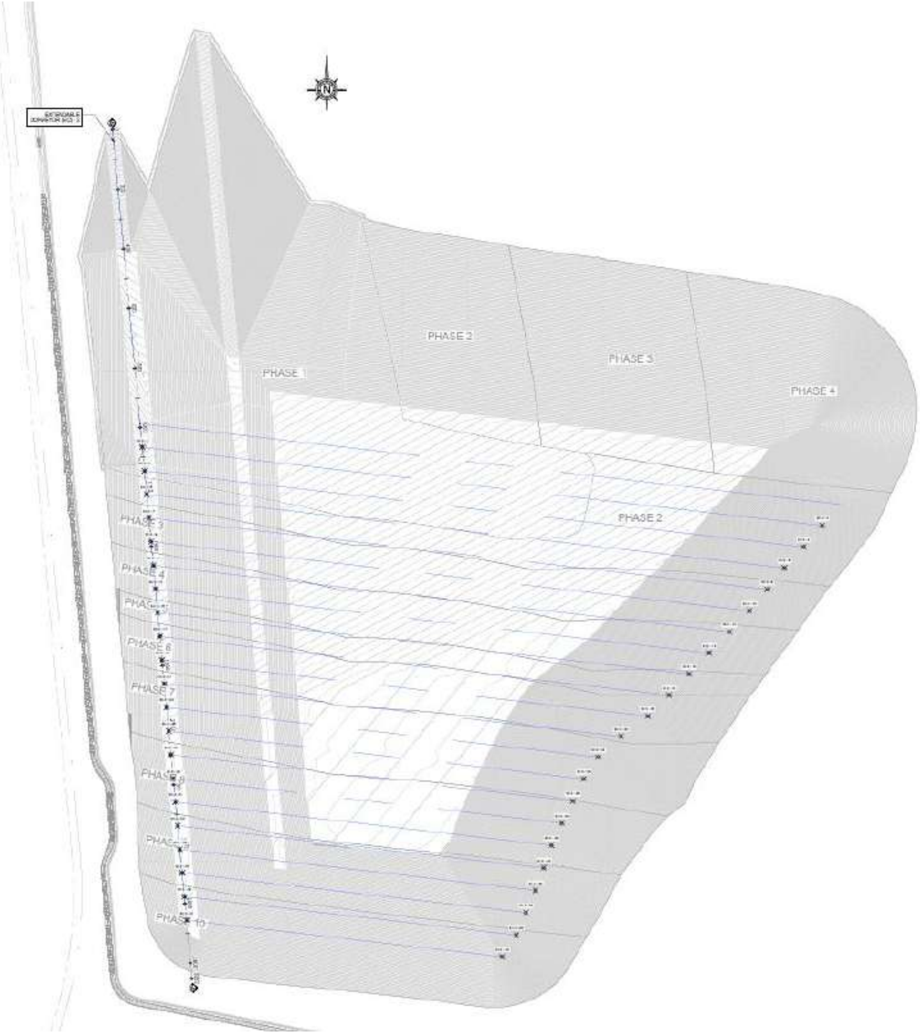


Figure 21: Standby System Shiftable Conveyor Positions

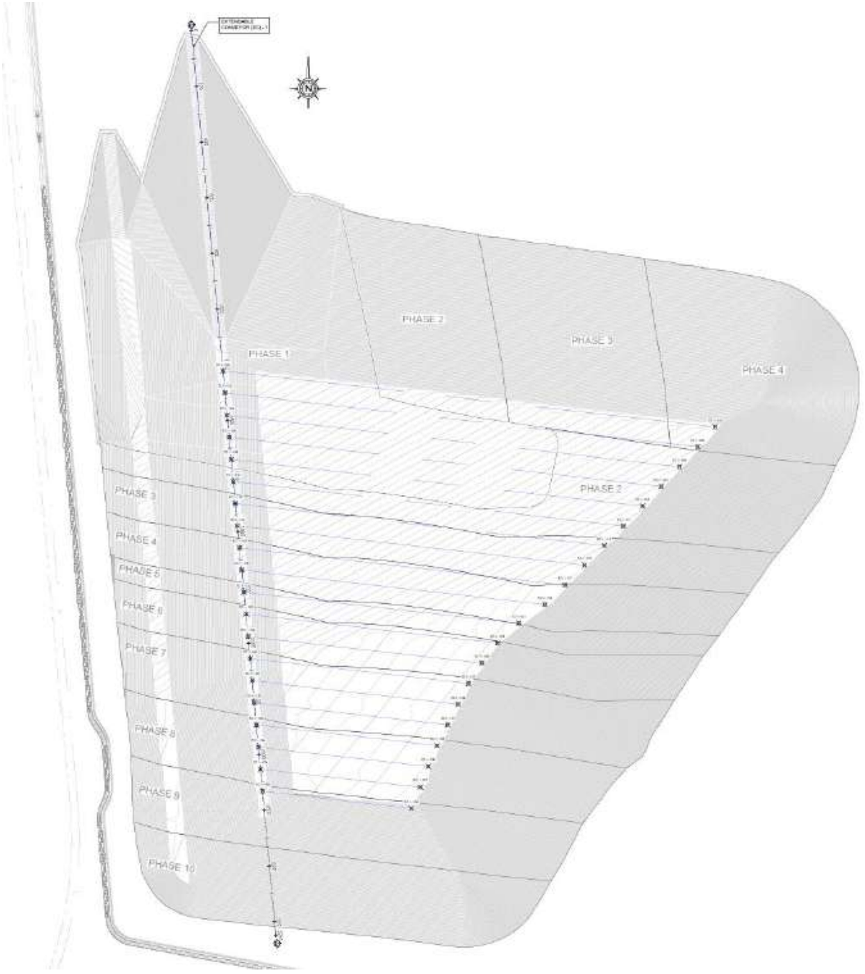


Figure 22: Main System Shiftable Conveyor Positions

B. Frontstack Ashing Operations

The frontstack ashing operations have been optimized to minimise the dozing and only require ashing within the stacker's reach (see Figure 11). Only 2m high ash cones will be used to form the frontstack, reducing the frontstack dozing volume and cost. The smaller cone rows can be placed next to one another, eliminating unnecessary dozing due to overlapping of the cones in front of the stacker (Overlapping to ensure the minimum safe edge distance for the stacker.) In addition, the minimum distance the stacker now needs to approach the newly placed fresh ash advancing frontstack crest in front of the stacker is increased with the smaller cones, reducing the stability risk at the same time as reducing the dozing costs. The stacker now only needs to walk in steps of 4.8m to place the next row of radially stacked 2m high ash cones ahead of the previous row. This allows the back of the new row of cones to be placed against the front of the previous advancing crest at frontstack level, in front of the stacker. The stacker will slew its boom clockwise for one stacking row and then anti-clockwise for the next.

The Cycle 1 in-to-out frontstacking operation shown in Figure 11 is mirrored for the Cycle 2 out-to-in frontstack ashing, allowing the stacker to always ash ahead of itself, forming a platform on which to travel which has ample safe edge distance to the front and side of the stacker. The stacker must always travel with its link conveyor at 90 degrees to the shiftable conveyor, to gain the maximum stacking reach. The stacker's boom is slewed radially from perpendicular to the shiftable conveyor, up to the previous frontstack crest. The 2m ash cones are to be dozed the shortest distance over the advancing edge of the frontstack crest. This operation will form the minimum required stacking reach along the entire length of the frontstack, without any additional ash having to be placed. The shift length has been designed to leave the minimum safe edge distance at the shiftable conveyor head end and it is especially critical that the minimum stacking reach be adhered to in this area, to provide the minimum required safe edge distance for the head end in the next shift. If this is not done, either the shiftable conveyor would have to be shifted shorter, throwing out the subsequent setting out and stacking information, or the shiftable conveyor would be exposed to higher than acceptable risks in the next position.

During ashing the stacker boom conveyor should be lowered to allow only about 0.5m between the top of the 2m cones and the bottom of the conveyor head pulley, to minimise the dust blow. Some equipment will need to be provided to enable the stacker operator to set and monitor the boom vertical angle and 2m stacking cone height within the required 200mm tolerance, as it is not possible to see the top of the cone from the stacker operator's cabin.

The ashing reach for each position of the stacker shiftable conveyors (Top and Bottom) were determined and the setting out coordinates are included in the drawings listed in 366-513556 to 366-513577 and the Design Report (366-511915).

C. Sideslope Ashing Operations

For the bottom stacker there will be a constant decrease in elevation of the front stack as development of the dump progress in a southernly direction. The crest of the shorter final 1:5 slopes will move further and further forward in front the shiftable conveyor head station. The balanced cut/fill stacking point will then also move forward to a point within the maximum stacking reach of the stacker. Construction of the final 1:5 slope will have to be closely monitored and directed on site. Typical stacking and dozing operations and geometry relative to the head station has been provided to facilitate construction of the final side slope (see Figure 9, Figure 10, Figure 17 and Figure 18). A front view of the Main and Standby System final slop geometry can also be seen in Figure 23.

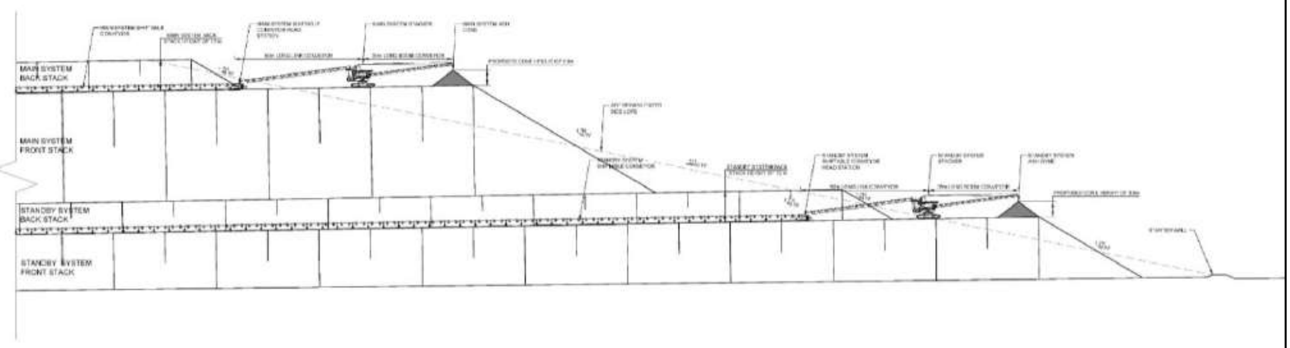


Figure 23: Kusile Cut Back Side Slope Operation

D. Cycle 1 Frontstack Sideslope Ashing Operations

When the stacker reaches the end of the travel for its tripper car at the head end, special sideslope ashing operations need to be done, depending on which direction the stacker is to travel around the head station. The head station travel limit for the tripper car is determined by the conveyor belt geometry and belt skew protection equipment constraints and has been assumed to be 30m back from the head station pulley centerline, for this revision (see Figure 9 & Figure 10). This dimension is not known exactly and should be checked by Kusile engineering when the stacker reaches this point and the drawings updated if necessary.

When reaching the end of the Cycle 1 in-to-out frontstack ashing at this point, the stacker must place as much ash as possible of the sideslope stacking using 2m cones ahead of itself, as it slews its link conveyor around until the stacker reaches the sideslope maximum reach stacking position (see Figure 9 - Position A). During this process, the stacker must place 8m high stacking cones at its maximum reach radius, in order to form the minimum stacking reach and safe edge distance crest requirements for the next conveyor position head station (see Figure 9 – Stacking Sequence A), as well as the platform required to walk the stacker around the head station to Position B behind the head station, tying into the previously placed frontstack crest (see Figure 9, Cycle 1 – Stacking Sequence B). Once the stacker has been moved around to Position B, the remainder of the sideslope maximum reach stacking is done from here, also using 8m high stacking cones, to place the maximum free ash (see Figure 9, Cycle 1 – Stacking Sequence B).

The stacker must then be moved back to around the final 1:5 frontstack sideslope crest line to place additional dozing ash cones onto the 1:5 cut slope (see Figure 17 & Figure 18). This operation will have to be supervised closely by the contractor as it requires 24 hour dozing to continuously remove the stacked ash away from the relatively small stacking area the stacker can reach on the 1:5 slope, due to the safe edge distance and height difference between the shiftable conveyor platform and the cut down 1:5 slope. If any problems arise with the contractor’s plant, the stacker will have to walk to the start of the backstack and place ash in the Operation 1 position, taking care not to place ash in such a position as would prevent it comming out again to complete the frontstack sideslope additional dozing, or to be able to get in behind this ash to build the Cycle 1 backstack from out-to-in.

Once the top stacker system reaches the Cycle 1 balanced cut/fill dozing positions, sufficient ash would exist in the stacked sideslope area, allowing the stacker to start straight away with the full Cycle 1 backstack after moving around the head station to the backstack side, and the final 1:5 backstack sideslope would simply be tied into the final 1:5 frontstack sideslope. If it becomes evident that too little ash was placed at any point to form the complete balanced cut/fill sideslope, additional ash can simply be placed in the next sideslope stacking operation to make up the shortfall. No information is available on the bulking factor of the cut/fill dozed ash and this will have to be monitored on an ongoing basis and the stacking geometry adapted to account for any shortage or surplus of ash.

With the Cycle 1 ashing operation, even though the stacker does not need to go back around the head station again after finishing the final frontstack sideslope, as it will next build the Cycle 1 backstack from out-to-in, the same 32m distance as for Cycle 2 has been chosen for the platform behind the conveyor in the Cycle 1 cut area. This 32m gap has been selected, to give the most flexibility to the contractor to decide when the best time would be to do the cut/fill frontstack sideslope dozing and still allow the stacker to travel around to the backstack. It is however recommended that the cut/fill dozing be done as soon as possible after the shift, to minimise the ripping costs of the cemented ash. The deep auger testing done for the stability review showed that the ash cementation does vary locally, making some of the ash relatively easy to doze away and some requiring cross ripping to loosen it. Once ripped, the ash will not be difficult to push and a pushing difficulty factor doesn't need to be allowed for in the dozing costing. Additional ripping time should simply be allowed for ripping, probably by simply allowing fewer minutes per hour in the dozing efficiency factor.

When dozing the final 1:5 sideslope behind the stacker, the front crest of the final sideslope cut area is to be cut 32m parallel behind the conveyor centerline, with this backwards slope being worked down to a 1:1.5 slope by cutting the ash down in steps and dozing at a successively increasing angle as the cut deepens, to prevent a vertical cut face resulting if the ash is cemented. This is necessary to limit the stability risk to the stacker as it walks along this platform. The final 1:5 slope fill area is to be constructed from this 1:1.5 cut slope.

E. Cycle 2 Frontstack Sideslope Ashing Operations

As mentioned before, the Cycle 2 frontstack sideslope needs to be cut down behind the conveyor as soon as possible after the Cycle 1 shift is completed and the conveyor moved away, to prevent holding the stacker up when it gets to this point and to enable the contractor to cut the ash down as soon as possible, before it cements even harder.

Once the Cycle 2 backstack has been stacked from in-to-out, before placing the additional dozing sideslope ash, the stacker should first carry out the 8m high cone maximum reach ashing operations from Position B behind the conveyor (see Figure 16 – Position B, Stacking Sequences A & B), to prepare the platform for the stacker to walk around and stand on at Position A in front of the conveyor. The area of ash between the existing head end crest and the back of Stacking Sequences A & B can be placed using 2m stacking cones as the stacker walks towards Position B, with the maximum reach being achieved by stacking 8m cones from Position B. Stacking Sequence B will need to be done an additional 2 to 3 times, depending on the frontstack height, to advance the front crest from 36m to 40m from the head station centreline, before moving the stacker around to Position A. This will give sufficient safe edge distance for the stacker at Position A and allow the back of the Stacking Sequence B 8m cones to tie into this crest without forming a valley.

Once the construction of the head end sideslope and frontstack platform is completed from Position B, the stacker must be moved back to the additional dozing ash stacking position (see Figure 17 & Figure 18), to allow it to place the additional dozing ash directly onto the 1:5 sideslope. As with the Cycle 1 sideslope additional dozing operation, the contractor will have to monitor and supervise this operation closely. The relatively small ashing area the stacker can reach directly onto the 1:5 slope can be problematic. If the contractor has any mobile plant problems during this stage, the stacker should slew its boom over the shiftable conveyor and ash over the frontstack crest.

Once sufficient ash has been placed to complete the sideslope, the stacker should be moved around the head station to Position A (see Figure 9 & Figure 10) and place the remaining maximum reach sideslope free ash and the minimum safe edge distance crest for the next conveyor position head station from Position A, using 8m cones (see Figure 9 & Figure 10 – Position A, Stacking Sequences A & B). Stacking Sequence B is done while the stacker slews its link conveyor around towards 90° to the shiftable conveyor, achieving the maximum reach. Once this maximum stacking reach stacking has been done, the 2m cone Cycle 2 frontstacking can begin.

F. **Backstack Ashing Operations**

The backstacking operations will be done using 14m high stacking cones, with a stacker walk distance of 4.8m between cone rows and 0.5m between cones during radial slewing, to allow for final finishing of the backstack to 12m high above the frontstack. This will allow for filling the valleys between the stacking cones and some settlement during final finishing by the dozer. The stacker boom will slew from the previous backstack crest towards the shiftable conveyor, until the backstack front toe reaches 7.5m from the shiftable conveyor centerline, to leave a 6m wide access roadway behind the shiftable conveyor. The stacker then walks 4.8m in the direction of ashing and slews its boom back towards the previous backstack crest while placing the next row of cones (see Figure 5, Figure 6, Figure 7, Figure 8, Figure 15, Figure 16, Figure 17 & Figure 18)

G. **Cycle 1 Backstack Ashing Operations**

The Cycle 1 backstack, built from out-to-in starts with backstack ashing Operation 1, to fill the wedge of ash at the head end between the previous backstack crest and the maximum backstacking reach (see Figure 5 & Figure 6). The Cycle 1 Operation 1 and Operation 2 stacking can be done in 100m working areas to minimise the dust, or Operation 1 completed first if dust is not a problem. The backstack is then built behind the stacker using Operation 2, parallel to the conveyor (see Figure 15 & Figure 16). The backstack needs to be stopped short at this point to enable the stacker to later travel out with its link conveyor at 45 degrees to the shiftable conveyor pointing to the north, when it places the Cycle 2 backstack from in-to-out. However, in order to also eliminate an area of dozing at this point in the next Cycle 2 backstack, due to the stacker not being able to reach back to the Cycle 1 backstack crest, the next 275m of backstack is placed during the Cycle 1 operation, with its toe parallel to the next (Cycle 2) conveyor position to leave sufficient clearance to enable the stacker to travel in-to-out during the Cycle 2 backstack (see Figure 15 & Figure 16)

A 35m gap needs to have been formed in the previous Cycle 2 backstack, parallel to the current Cycle 1 conveyor position from chainage 0m to chainage 350m (see Figure 15 & Figure 16), to enable the Cycle 1 radial shifting operation to be done with the stacker behind the conveyor. This open area allows sufficient space behind the conveyor at the tail end to enable an s-curve to be formed after completing the Cycle 1 backstack and the stacker then driven through the s-curve to the section of conveyor placed on the new shift position (Figure 12 & Figure 13). During this process the stacker link boom is also reversed to point furthest away from the shiftable conveyor head station and tripper car to enable the Cycle 2 backstack to be built from in-to-out after the shift.

H. **Cycle 2 Backstack Ashing Operations**

The Cycle 2 backstack is then placed from in-to-out starting at the northern toe end, and as mentioned above, keeping the minimum of 35m clear between the toe and the next Cycle 1 conveyor position (Figure 15 & Figure 16). From this point the backstack toe is placed parallel to the current Cycle 2 conveyor position, up to the starting position of the backstack, tying into the previously placed backstack front crest. Near the end of the Cycle 2 backstack, the backstack Operation 1 will again first be required to fill in the wedge between the previous Cycle 1 backstack crest and the maximum backstacking reach. The backstack operational crest should be monitored to ensure that too much ash is not placed, as this distance increases to over 100m, as the sideslope moves inwards with the decreasing frontstack height with development of the dump progresses (Figure 17 & Figure 18).

As the stacker completes the Cycle 2 backstack stacking from behind the conveyor at the head end, an area of backstack may not be reachable, due to the stacker's geometric limitations and should be stacked later from in front of the conveyor to prevent unnecessary additional dozing in the next backstack (see Figure 8).

3.1.1 **Mobile Equipment Operations**

Both Stackers should be operated in such a way that the ash is placed as close as possible to the Shiftable conveyor positions and Conveyor ashing reach. However, the use of mobile earthmoving plant (optimum types and sizes to be determined by operating conditions) are required for the following operations:

- Moving of ash to positions outside the reach of the stackers when required.
- Trimming and final profiling of the dump surface and side slopes, construction of the conveyor platforms and stormwater control berms, cleaning of solution trenches and drains and road maintenance.
- Shifting of conveyors, head and tail stations and transformers.
- Compaction of the ash, if deemed necessary, can be carried out by towing a smooth drum roller.

3.1.2 **Stacker Shiftable Conveyor Positions**

The positions of the shiftable conveyor are dictated by the operational capabilities of the Stacker machines. The head station co-ordinates are to be provided on drawings by BMH to enable the accurate setting out of each new conveyor position. These conveyor positions should be adhered to accurately, as any deviation from the design will have a knock-on effect for all future conveyor positions and require a revision to the drawings to determine the new construction setting out and level information. This is required to allow the contractor accurately to construct the dump and to enable Eskom to do signoffs on the dump's "as build" geometry on a regular basis to ensure construction according to the design.

3.1.3 Final Levels & Side Slope Geometry

To maintain the capacity of the ash dump it is important that it is built to the design geometry and levels. The top and bottom stackers frontstack, backstack and sideslope construction geometry and levels are provided on DWG 366-513579 and 366-513578. Both backstacks being built a constant 12m layer on top of each frontstack, except where dozed down to form the sideslopes, and shall be achieved within the following tolerances:

Frontstack	± 500mm (Position)
	± 200mm (Level)
Backstack	± 500mm (Position)
	± 200mm (Level)

The steepest gradient on the frontstack top surface of the dump shall be 1 in 17. The maximum height of the frontstack crest shall be as provided in the construction drawings and the minimum height of the backstack crest shall be 12m above frontstack level, except where dozed down to form the final 1:5 average sideslope.

All final ash dump side slopes shall have an average slope of 1 in 5 (see Figure 19). Erosion protection measures will be constructed on the side slopes (refer to Section 4). The erosion protection berms on the sideslopes are to be formed in the ash, by cutting their profile out of the 1:5 dozed sideslope, pushing the cut material forward from the existing berms and over the sideslope front crest in the fill area. In the cut area of the slope, the berm cut material is to be dozed to the end of the berm and pushed down the 1:5 slope to the start of the fill slope and pushed over the crest. The last few meters of the berms may thus remain unfinished until the next sideslope, due to this process.

The ashing contractor is to construct additional inustructure to limit the erosion of the dump as follows:

A. Backstack final crest berm (see Figure 20) to prevent stormwater runoff volumes from flowing uncontrolled down the sideslopes (revised crest berm as seen in Figure 24).

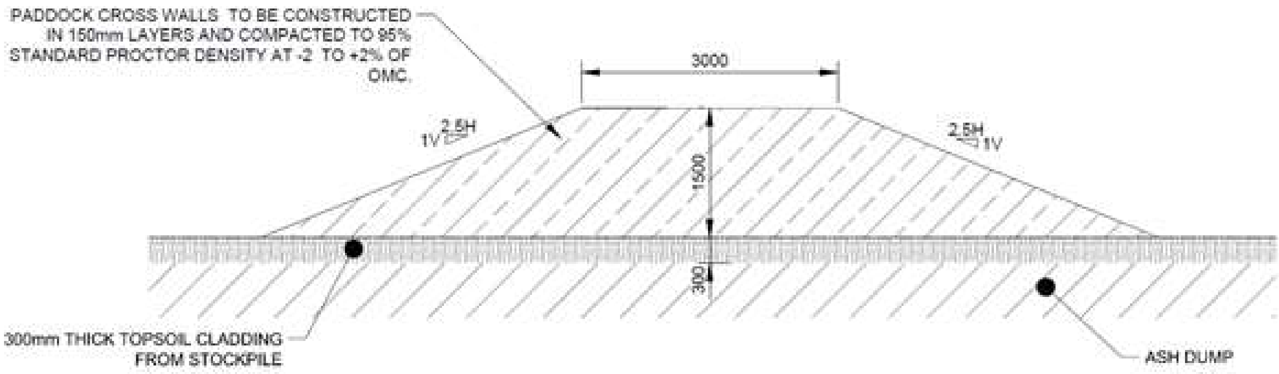


Figure 24: Revised Crest Berm Geometry

B. Takedown chutes shall be provided to lead this backstack top surface water off the dump in a controlled manner, to prevent the formation of large erosion gullies and ash pollution spillages (see Figure 25).

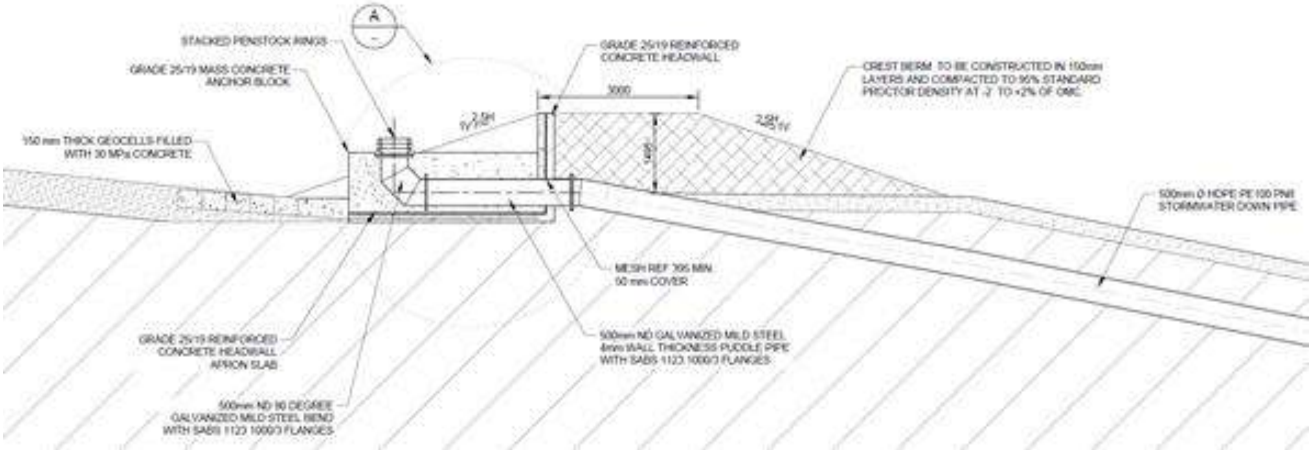


Figure 25: Crest Inlet Structure

C. Contour berm inlets to allow storm water collecting on the final shaped side slopes to drain in a controlled manner, to prevent the formation of large erosion gullies and ash pollution spillages (see Figure 26).

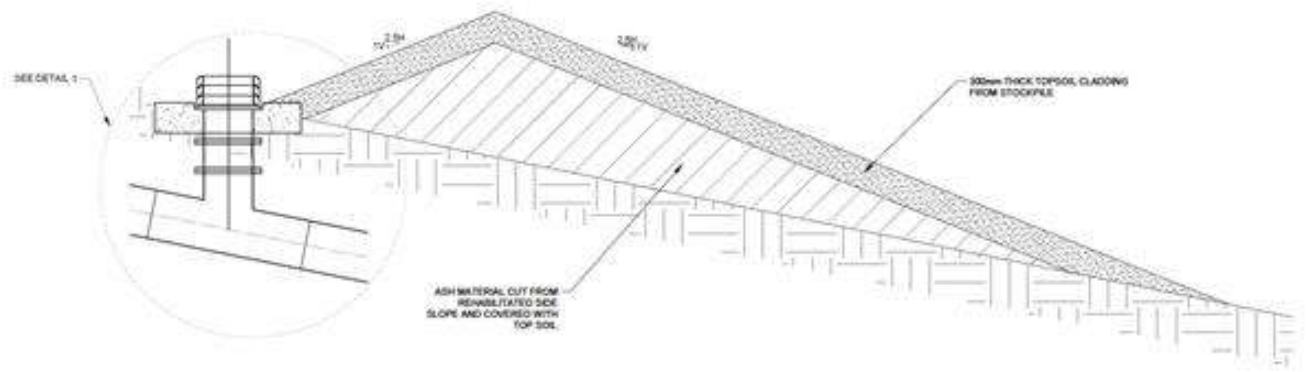


Figure 26: Contour Berm Inlet

For the detailed design of the ADF (Ref: 366-146714) a stormwater management system was proposed to enable rehabilitation of the areas where the ADF has reached its full extent. Refer to drawing to Section 3.1 for the final Post Deposition general arrangement of each phase of the ADF which shows the Post Deposition works required.

3.1.4 **Method Of Deposition**

Eskom BMH to Complete

3.1.5 **Method Of Inloading**

Eskom BMH to Complete

3.2 Conveyor Shift Procedures

The action of moving the shiftable conveyors to a new position is known as a shift and can be a time consuming and complex operation. It must be well planned by the ash dump operators as extra plant and personnel are required on site during the operation.

Before the shift can take place, it must be ensured that:

- The dump is sufficiently advanced, so the new conveyor will have the necessary safe edge distance required
- The alternative ashing system is in good working order and has been recently maintained to alleviate the need for emergency ashing and has sufficient capacity available for the expected shift duration, allowing for unforeseen problems.
- The new conveyor platform has been constructed to the correct line and level and within the tolerances stated in this manual.
- Any erosion protection berms/cut-off drains between the old and new platforms have been levelled.

3.2.1 **Method of Moving the Shiftable Conveyors**

This method applies to both the stacker shiftable conveyors. To carry this out a bulldozer fitted with a jib hook plus a Demag rail shifting head is necessary.

A. Unload all ash from the conveyor and tripper which is to be moved.
B. Clear any ash deposited between conveyor rail sleepers and at head and drive mechanism
C. Prepare area between existing and new conveyor position.
D. Mark the desired new conveyor position according to the head station setting out coordinates given on the drawings.
E. Locate the bulldozer (with fitted rail shifting head attachment) at the tail end of the conveyor.
F. Release the belt tension by slackening the electric winch at the head end.
G. Cut the conveyor belt at the head end and loosen all electrical and C&I cabling.
H. With the tripper car at about chainage 450m, form a 150m long, 200m horizontal radius S-curve in the conveyor near the tail end, locating the first 275m of conveyor from the tail end on its new position.

I. Move the tripper through the S-curve towards the tail pulley of the conveyor to a pre-determined position. The distance selected should at least adhere to the specifications listed in Table 6 below.

Table 6: Conveyor & Tripper Cars Characteristics

Description	Top Stacker	Bottom Stacker
Length of Tripper	58m	58m
Tolerance for positioning of tripper (plus only)	12m	12m
Distance required for two curves to form rail loop	40m	40m
Safety clearance between parked tripper and bulldozer	20m	20m
Total	130m	130m

J. Remove anchor plates and install at new position.

K. Attach a sling to the brackets on the conveyor head end and drag the head pulley portion to its new position.

L. Adjust the tension of the two springs of the rail shifting head by means of a torque wrench to 107Nm.

M. Lift the rail shifting head by attaching the shackle on it to the jib of the bulldozer. Position the head over the rail and lower. As soon as the rail takes the mass, the springs will open the clamp. This is sufficient for the wheel type clamp to be inserted in position. The bar link is now coupled to the rail shifting head which can be lifted. The clamp will close on lifting.

N. The driver can proceed if the rail shifting head is properly secured.

- 1st pass of dozer: lift modules 200mm vertical.
- 2nd pass of dozer: lift the rail 150-200mm and drag modules no more than 300mm each pass, in the direction of the shift.

O. The bulldozer then travels towards the tail pulley until the side shifting of the conveyor becomes difficult.It will be found that the head end of the conveyor has a tendency to creep back. It will be necessary to re-align the head end of the conveyor from time to time.

P. Resplice the conveyor belt and reconnect all cabling to the head station.

4. POLLUTION CONTROL DAMS AND CLEAN WATER DAMS

The 60Yr ADF has been designed to comply with GN 704 requirements which specify on the separation of clean and dirty water on the site.

4.1 Description of Pollution Control Dams (DAM Complex 1-3)

The dams are located as shown in **Figure 27** below. The extent of the dam footprints is limited by the stream diversion to the north, the ADF to the south and the conveyor lines to the west. The natural contours to the east present a limitation to the fall of the drainage lines.

Figure 27: Pollution Control Dams Layout

The Pollution Control Dams (PCDs) consist of PCD's 1A and 1B, which are located between the two conveyor lines EC1 and EC2. PCD 2 that is located east of the conveyor line EC1 and adjacent to

CWD 2; while further east is the group of PCDs numbered from 3 to 6. North of the Klipfonteinspruit is the Road PCD which will pump directly to the silt trap inlet channel of PCD 2.

PCD 3 to 6 are interlinked to allow for a cascading effect from PCD 6 toward PCD 3 with all PCD's being able to gravity drain to a concrete channel leading to PCD 2. Each of the individual compartments has its own outlet pipeline to allow for emptying during maintenance or under emergency situations. Under normal operating conditions the outlets from PCD 3 to 6 should remain open to ensure storage capacity is available in the event of heavy rainfall events and overtopping. Once PCD 2 has reached its capacity, the outlet valves from PCD 3 to 6 should be closed allowing those to collect the active run-off. PCD 3 to 6 can then be drained to PCD 2 at a later stage under controlled conditions.

The Clean Water Dams (CWDs) consist of CWD 1 which is located southwest of PCD's 1A and 1B and CWD 2 which is located adjacent to the PCD2.

The dams are grouped as follows:

Dam Complex 1:	PCD 1A, PCD 1B & CWD 1
Dam Complex 2:	PCD 2 & CWD 2
Dam Complex 3:	PCD 3, PCD 4, PCD 5, and PCD 6.
Dam Complex 4:	Road PCD

Two pump stations have been incorporated to allow for the abstraction of water from the PCDs and CWDs. The pumping is intended for dust suppression, irrigation and transferring of water from one dam to another (see Section 4.2.1). The first pump station (Pump Station 1) is located west of PCD 1A and the second (Pump Station 2) is located east of PCD 2.

Each dam has been designed with dedicated concrete outlet structures and abstraction pipelines as detailed on their respective layout and sectional drawings. The outlet structures and abstraction pipelines have been arranged as follows:

- PCD 1A – Two outlet pipelines are located 1.0m above the highest base floor level leading to Pump Station No. 1. The two lines are for the dust suppression system and the transfer system.
- PCD 1B – Two outlet pipelines are located 1.0m above the highest base floor level leading to Pump Station No. 1. The two lines are for the dust suppression system and the transfer system
- CWD 1 –Two outlet pipelines are located 1.0m above the highest base floor level leading to Pump Station No. 1. The two lines are for the irrigation system required for rehabilitation.
- PCD 2 – Two outlet pipelines are located 1.0m above the highest base floor level leading to Pump Station No. 2. The two lines are for the dust suppression system and the dirty water transfer system.
- CWD 2 – Two outlet pipelines are located 1.0m above the highest base floor level leading to Pump Station No. 2. The two lines are for the irrigation system required for rehabilitation.
- PCD 3 & 4 – Outlets for both dams are located at the inner corners of the dams' compartments (1.0m above the highest base floor level). Abstraction pipelines from both dams terminate in a channel on the southern side of the dams. The channel discharges flow to the inlet channel of PCD 2. The abstraction pipelines are equipped with isolation valves in a valve chamber to enable controlled withdrawal from the dams. Withdrawal of water from the dams via the pipelines shall be restricted to dam drawdown scenario for maintenance purposes and for supplying PCD 2 with water for dust suppression if the level in PCD 2 is low but there is sufficient water in either of the dams.
- PCD 5 & 6 – A similar arrangement as for PCD 3 & 4 has been designed for the outlet structure and abstraction pipelines. A single valve chamber houses the isolation valves on the pipelines. The pipelines terminate in the same open channel that collects water from PCD 3& 4 and terminates at the inlet channel of PCD 2.
- Road PCD – With extraction occurring by means of a submersible slurry pump feeding a pipeline to PCD 2 silt trap inlet channel.

4.2 Pollution Control Dams (DAM Complex 1-3)

4.2.1 Process Description and Control Philosophy

The ADF and pollution control dams will be developed in two construction phases in line with the progression of the ADF development as follows:

- Phase 1:
  - This phase will consist of all pertinent infrastructure for the start-up operations of the ADF. The infrastructure will include the initial ash dump development footprint, Dam Complex 1 (PCD 1A, PCD 1B & CWD 1), Dam Complex 2 (PCD 2 & CWD 2) and Dam Complex 4 (Road PCD).
- Phase 2:
  - This phase will consist of the construction of the remaining ADF footprint in 5-year lined area intervals. PCDs 3 to 6 (Dam Complex 3) form part of Phase 2 of the design.

A simplified water flow is presented in Figure 28. This figure illustrates the water flows between different water management infrastructure, which is to serve the ADF.

The dams on the western side of the ADF (Dam Complex 1) will cater for the western and central catchment, while the dams on the northern side of the ADF (Dam Complex 2 & 3) will cater for the eastern and northern catchment. There is allowance for strategic spillways at the dams but spilling is to be avoided as far as reasonably possible. This is a requirement to comply with Regulation 704 of the Water Act and there is also the risk that any significant spillage could lead to flooding of the pump station units and conveyor lines. Any spillage needs to be reported through the appropriate channels within the stipulated timeframe of the environmental department of Eskom.

Runoff from storm events within the dirty water catchment of the ADF and the seepage inflows from the ADF is intended to flow, via lined channels, that converge and then flow through a silt trap. The silt trap is intended to function as a stilling basin to allow settling out of solids before the flow continues further, entering one of the PCDs. Runoff from the rehabilitated area of the ADF (top-soiled and grassed) that has been monitored and confirmed clean will report to the permanent Clean Water Dam 1 & 2 or Dirty to Clean PCD 1B via the clean water channels or Dirty to Clean Channels respectively.

The upstream dam, PCD 6, will spill into the downstream dam, PCD 5, which will in turn spill into the downstream dam, PCD 4, which will in turn spill into the downstream dam, PCD 3 which will have an emergency spillway that flows into the Klipfonteinspruit river area. PCD 2 has been sized to spill only once in 50 years as per Government Notice (GN) 704. Concrete silt traps will be installed at the entrance of the PCD 2, 4 & 6.

There is a provision to transfer water from PCD 1 and PCD 2 to the ADDD and later on to PCD 3 and PCD 6. Pumps located in Pump Station No. 1 will transfer water from PCD 1 to the ADDD and later on PCD 3 and PCD 6 and pumps located in Pump Station No. 2 will transfer water from PCD 2 to the ADDD and later on PCD 3 and PCD 6. The transfer provision prevents spillages from PCD 1 and PCD 2 when there is still capacity in dams PCD 3 to 6. Refer to Section 4.2.3 for the transfer control settings.

Make-up water will be provided for the dams when the levels are low and they cannot provide sufficient volumes to supply the demand. Refer to Section 4.2.3 for control settings.

The following should be noted for the make-up systems:

- HRD make-up water will only be supplied to PCD’s 1A, 1B and 2. This is dirty water that will be used for the dust suppression system.
- Clean raw water make up can be supplied from the Kendal – Kusile raw water pipeline to all CWD’s and all PCD’s.

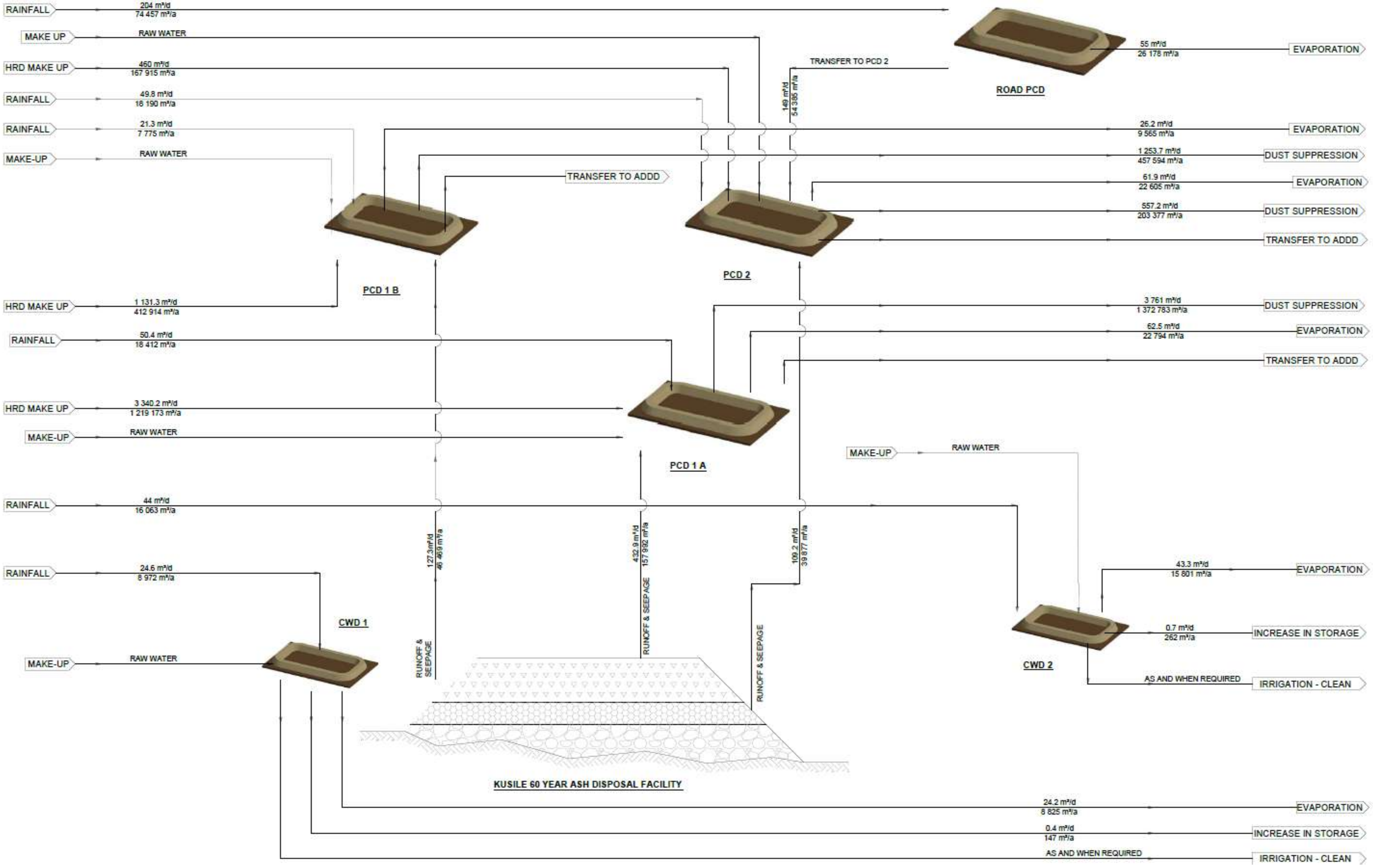


Figure 28: Process Flow for PCD's and CWD's

4.2.2 **Dam Capacities**

The compartments have been sized based on the total volume requirements provided in the integrated water balance model with a calculated required combined operating volume of at least 425,000 m^3 for the PCD group and 70,000 m^3 for the CWD group. The dams are designed with a combined volume (to Full Supply Level) of at least 477,518 m^3 for the PCD group (decreasing to 434,643 m^3 once PCD1B becomes a CWD) and 67,472 m^3 for the CWD group (increasing to 110,347 m^3 once PCD 1B becomes a CWD) (refer to Table 7 for breakdown of Dam Volumes and Levels).

The reserve volume for each group is to function as a contingent and is intended to limit spilling from the dams, to be less than once in 50 years and therefore meet the conditions set out in Government Regulation No. 704. The two main components highlighted from the regulation, stipulates that clean water catchments must be separated from dirty water catchments as far as practically possible, and no dirty water system may spill into the adjacent clean water system more than once in a fifty-year period. The design approach also allows for a limited reserve component to lower the risk of flooding of the other infrastructure.

The design volumes and dam geometric information for each dam in Phase 1 is shown in Table 7.

Table 7: Pollution Control Dam Volumes and Levels

	PCD 1A Compartment		PCD 1 B **Compartment**		PCD 2 **Compartment**		Road PCD **Compartment**		CWD 1 **Compartment**		CWD 2 **Compartment**	
	1	**2**	**1**	**2**	**1**	**2**	**1**	**2**	**1**	**2**	**1**	**2**
Crest Level (m)	1 458.50	1 458.50	1 450.40	1 450.40	1 461.00	1 461.00	1 445.00	1 445.00	1 455.90	1 455.90	1 467.70	1 467.70
Internal Spillway Level (between compartment) (m)	1 457.50	1 457.50	1 449.40	1 449.40	1 460.00	1 460.00	Variable		1 454.90	1 454.90	1 466.70	1 466.70
External Spillway Level (m)	1 457.70	1 457.70	1 449.60	1 449.60	1 460.20	1 460.20	1 444.20	1 444.20	1 455.10	1 455.10	1 466.90	1 466.90
Internal Base Level (m)	1 452.14	1 452.14	1 445.12	1 445.12	1 452.85	1 452.85	1 438.58	1 438.58	1 451.36	1 451.36	1 460.87	1 460.87
Depth (m)	6.36	6.36	5.28	5.28	8.15	8.15	6.42	6.42	4.54	4.54	6.83	6.83
Capacity (m^3)	102285		42875		113471		24000		18555		48917	
Freeboard (mm)	800	800	800	800	800	800	800	800	800	800	800	800
Side slope	1:3	1:3	1:3	1:3	1:3	1:3	Vertical		1:3	1:3	1:3	1:3
Outlet Pipe Diameter (mm)	600	600	600	600	600	600	TBC		600	600	600	600
Invert Pipe Level (m)	1 454.50	1 454.50	1 447.29	1 447.29	1 455.00	1 455.00	TBC		1 452.90	1 452.90	1 462.70	1 462.70

4.2.3 Control Settings

Table 8 presents the recommended control levels for the dams. If it is required to adjust these levels, new setting levels must be simulated in the Water Balance Model before implementation. The exercise will give a picture of the expected performance of the system.

Table 8: Dam Control Levels

LEVEL SETTING	PCD 1A and PCD 1B	PCD 2	PCD 3	PCD 4	PCD 5	PCD 6	Road PCD	CWD 1	CWD 2
High Level / Transfer Setting (%)	50%	50%	None	None	None	None	5%	None	None
Recommended Operating Level (%)	25%	25%	No control	No control	No control	No control	0%	50% - Control not required	50% - Control not required
Stop for Make-Up Water (%)	15%	None	None	None	None	None	None	15%	15%
Low Level (%)	5%	5%	No control	No control	No control	No control	0%	5%	5%

Note: All levels indicated in the table are based on a zero-reference level of the top of concrete of the outlet sumps in the dams. The volume of water that is below the outlet sump level is considered as dead volume in the dams.

Road PCD should be pumped out to PCD 2 as soon as levels exceed 5%. At no point should water be allowed to settle and remain in the Road PCD for extended periods of time (resulting in silting up of extraction chamber).

The low-level setting represents a level where make-up water will be required for the dam. The setting has been standardized to 5% for all the dams (see note below Table 8).

The supply for make-up water will be stopped at level of 15%. The level has been standardized based on requirements for PCD 1A. It is set that each filling of PCD 1A should represent 10% of the total volume of the dam. The volume represents approximately one week of dust suppression requirements from PCD 1A if there is no rainfall within the time period.

The recommended operating level has been set to 25% for the PCDs and 50% for the CWDs. 25% setting for the PCDs leaves sufficient buffer in the dams whilst still providing sufficient water for the intended use. The CWDs should be operated at a normal setting of 50%. There is no risk on the dams in regard to environmental contraventions due to spillages.

High level setting for PCDs 1A, 1B and 2 is 50%. Once the water level reaches this setting the transfer system will be initiated where water will be pumped from the respective PCD 1A/1B or PCD 2 to PCD 6.

4.2.4 **Dams with a safety risk**

Most of the dams will be storing volumes greater than 50,000 m^3 and some have outer containment walls with maximum vertical heights that is greater or equal to 5 m. The dams will also contain potentially polluted water. These dams are therefore considered to be dams with a safety risk according to the Water Act and the classification is guided by the Dam Safety Regulation R139 of 2012 of South Africa.

The new dams will be licensed with the Department of Water Affairs in terms of Dam Safety legislation. The classification process indicates that the dams will be classified as Small Dams with a Significant Hazard Potential with a possibility of a Great Rating for Economic Loss. The dams will have to be operated, monitored and inspected in compliance with the applicable legislation and applicable conditions as reflected in the dam safety permit.

The dams will need to be inspected for function and integrity at least once a quarter, but a monthly inspection is recommended and after a heavy storm event. The inspections are to be done by personnel that is appropriately trained to identify and record any abnormalities or variations. An appropriate monitoring register and inspection procedure must be developed for these inspections.

A high-level dam safety evaluation is recommended to be done on an annual basis with the presence of an independent professional present. The intend of the high-level evaluation is to review the monthly records and allow for a visual inspection of the dam conditions and integrity. The category II dams are also likely to be required a formal dam safety evaluation that generally needs to be done every 5 years or as stipulated in the formal communication from the dam safety office.

4.2.5 Silt Traps

Reinforced concrete silt traps will remove silt from the dirty and clean water streams before discharging into the pollution dams. Major PCD’s (PCD 2, PCD’s 6 & 5 and PCD’s 4 & 3) will consist of a double silt trap to allow for cleaning and maintenance while one remains operational. PCD’s with single silt traps generally receive flow from leachate collection systems and therefore will receive very little silt, therefore justifying the single silt trap proposal. The silt traps have been sized, in accordance with the particle settling theory that is based on Stoke’s law, with the assumption of a 1 in 10-year flood discharge during 24-hour flood event. Storm event flows bigger than this will run through the silt trap.

The silt trap chambers can be accessed by large plant (FEL) to remove silt and dump to the drying pad area. It is important that silt traps are emptied regularly to avoid excessive build-up of silt within the chambers and subsequent overflow of silt to the PCD’s. It is easier to de-silt a silt trap than it is to de-silt an entire dam.

In order to isolate the silt traps, a cherry picker or TLB will be required to raise or lower the sluice isolation gates at the outlet of the silt trap.

4.2.6 Sub-Soil Dewatering and Monitoring

All dams have been designed with an intricate sub-soil drainage system beneath the base of the dams. This is a requirement to prevent the uplift of the lining system within the dams as a result of ground water upward pressures.

The sub-soil drains have been designed to drain to a de-watering pumping manhole as a distance away from the dams themselves. Fitted with a sump (for monitoring) and a float switch submersible pump, the collected sub-soil seepage will be collected and pumped via a return line to the dam from which the sub-soil seepage was collected. Monitoring / testing of the seepage should be done should the seepage be discharged to the natural environment.

4.2.7 **Maintenance requirements**

It is important to have an appropriate maintenance programme to allow for timeous action and allow for the optimal function and integrity of the dam structures.

Actions to include but not limited to:

- Operating and servicing all valves and structures at regular intervals as per manufacturers specifications. If the valves are not operated at regular intervals, there is a risk that they may not be able to operate during an emergency which can have catastrophic consequences.
- The cleaning out of the compartments and traps from silt deposits at intervals. Deposition of silt reduces the available capacity within the compartments and increases the risk of frequency of spillage and flooding of downstream structures.
- Recording, appropriate reporting and remedial works of subsidence, movement, depressions, or cracks in and on the walls.
- Eradication of shrubs or trees and ants, approved filling of burrows and holes.
- Protection and repair of the exposed HDPE liner.

It is advised that cleaning operations be carried during low flow (winter season) conditions.

5. DUST SUPPRESSION, STORMWATER MANAGEMENT & EROSION CONTROL

5.1 General Description of Dust Suppression Systems

The dry ashing system has the potential to create severe dust blow problems, detrimentally affecting the efficiency of the operating staff and equipment and creating visual and airborne pollution of the surrounding area. Readily identifiable effects due to dust blow conditions are:

- Reduced visibility and hence reduced safety on the dump and environs.
- Unpleasant working conditions leading to potential health problems.
- Detrimental effects on mobile plant and ash stacking equipment
- Soiling of vegetation and adjacent areas.
- Detrimental visual impact.

Every effort must be made to control the dust from the ash dump to prevent any of the above problems from occurring. The following methods, used on their own or combined, can be used for dust control on the ash dump:

- Smooth drum roller compaction.
- Water via mobile sprinkler machines or water bowsers.
- Water via sprinkler system.
- Soil cover.

5.1.1 **Smooth Drum Roller Compaction**

Nominal compaction will increase the possibility of a crust being formed due to pozzolanic action. A smooth drum roller towed by a dozer or a self-propelled roller is envisaged for this operation.

It should be noted that this method can only be used on the advancing face where it is dozed to a 1 in 3 slope.

5.1.2 **Water**

Water applied via a sprinkler system, mobile sprinkler machine or using a water bowser may prove effective in controlling dust. It should be applied at a rate such that a hard surface crust, resistant to dust blow is achieved.

5.1.3 **Soil Cover**

A thin layer of soil cover spread over the ash surface (~50mm) may prove successful in controlling dust blow problems and limiting the amount of water required for dust suppression.

Similar measures to those stated above must also be taken to prevent dust blow from access roads, haul roads, borrow areas, and other bare soil areas from where dust may be generated.

5.2 Dust Suppression Sprinkler System

The water demand for dust suppressing with the sprinkler system was computed by working out spray depths for both summer and winter months. As calculated within the Water Balance Report (366-513521) and based on the Kusile LPS Basic Design, Table 9 and Table 10 present the computations for the spray depths for summer and winter months respectively.

Table 9: Winter Dust Suppression Spray Depths

Dust suppression cycle (Days)	**5**
A/As	20%
Es	2.23
Daily spray duration (hrs/d)	10
Ea (mm)	4
Ess (mm)	3
Rainfall (mm)	0
Rate of Application (mm/24h day)	32.6
Spray Depth (mm/spray duration)	13.6

Table 10: Summer Dust Suppression Spray Depths

Dust suppression cycle (Days)	**5**
A/As	20%
Es	4.37
Daily spray duration (hrs/d)	10
Ea (mm)	8
Ess (mm)	6
Rainfall (mm)	0
Rate of Application (mm/24h day)	64.35
Spray Depth (mm/spray duration)	26.8

Where;

- Dust suppression cycle is the defined duration for dust suppressing on an area
- A/As is the percentage of area that is sprayed on a daily basis in relation to the overall area
- Es is the evaporation rate of water from the waste surface that has not been sprayed on a particular day
- Daily spray duration is the amount of hours to suppressed in a day
- Ea is the evaporation rate of water into the air above the water surface over the area that is being sprayed
- Ess is the evaporation rate from the saturated surface that is being sprayed
- R is the rainfall on a particular day
- Rate of application is the required spray depth for a complete 24hour period
- Spray depth is the actual spray requirement for the given spray duration.

The actual volumetric dust suppression requirements are worked out by multiplying the spray depths to the effective dust suppression areas. Tab 11 of the Microsoft Excel Water Balance Model presents the daily time step analysis of the Dust Suppression Requirements.

5.2.1 Dust Suppression Pump Design

As per Eskom. (2020). *Low pressure services Basic Design Report for Kusile Power Station 60 Year Ash Disposal Facility*.

5.3 Top-soiling & Rehabilitation

Top-soiling and rehabilitation of completed portions of the ash dump is carried out to ensure the following:

- Water running from the dump's rehabilitated surface is clean and uncontaminated by the ash
- The site should be suitable for other land uses following the dump's completion. The area should therefore be accessible and capable of sustaining future plant growth
- The completed ash dump must have as small a negative visual impact on its environment as possible. Final grading of side slopes and crests should be such that the ground profiles are curved, smooth and flowing rather than sharp changes in line and gradient. Storm water management should however not be compromised in the process

In accordance with the above, the ash dump must be rehabilitated progressively. Areas where final shaping and levelling of the ash has been completed must be top-soiled and rehabilitated immediately.

Care shall be taken to recover the topsoil necessary for use in top-soiling operations from positions in front of the advancing ash face before it is covered by ash. Areas and depths of suitable topsoil are indicated in the material balance (Refer to Appendix I). Once stripped the topsoil shall be used as soon as possible for rehabilitation purposes. Topsoil and dust suppression borrow areas shall be shaped so that storm water can drain naturally and is not ponded in them. Alternatively, a drainage trench must be provided to lead the water to the nearest dirty water infrastructure.

If it is necessary to stockpile the topsoil it shall be done in such a way that double handling of the material is prevented, and haulage distances are minimised. Stockpile heaps should not exceed 3.0m in height, and any necessary steps to prevent their erosion must be taken.

Top-soiling and rehabilitation shall be completed during the post deposition phase of construction of each phase. It must be noted that the material balance may change depending on the volume of material excavated during each phase and the material balance should be adjusted to fit the as built construction plan.

5.4 Storm Water Control & Erosion Protection System

Storm water runoff from the rehabilitated ADF will not necessarily be considered as dirty because the ADF would be capped with a layer of topsoil and vegetated. This water may potentially be considered as clean but this will have to be monitored and confirmed before being routed to the clean water dam. Therefore, provision has been made for monitoring the water quality at the Energy Dissipation Structures at the toe of the ADF.

Monitoring of the water quality will be done by obtaining “grab samples” from Energy Dissipation Structures which will be positioned at the toe of the dump just upstream of the storm water channels. The structures will be easily accessible to enable simple sampling procedures. The sampled water shall be assessed against water quality standards as stipulated in Section 10.7. Runoff that originates from open ash areas and recently rehabilitated areas that have not been monitored to ascertain if they’re clean shall be directed to the PCDs via the Dirty Water Channels.

Should it be confirmed that the runoff water is “clean”, it will be diverted to report to the clean water management system by constructing the proposed channel switch over from dirty to clean for the internal channel which will report to the CWD’s and PCD 1B which will become CWD 1B once phase 1 is fully rehabilitated. As each phase is rehabilitated, so the channel switch over point will extend to just beyond the tested and confirmed “clean” rehabilitated area.

The abovementioned rehabilitation philosophy ensures that all runoff reports to the PCDs before it is released to the CWD or directly to the environment. The approach was adopted to minimize the risk of contaminating CWDs.

5.4.1 **ADF Side Slopes**

Storm water runoff on the 1 in 5 side slopes will be diverted to berm overflow structures spaced at 25m horizontal intervals along the ADF perimeter berm. At the bottom of the dump, the runoff will be captured by a 2.5m high earth berm that is located beyond the toe of the dump and just before the storm water channels. The berm will capture the flow allowing it to seep into the leachate collection layer with any excess being directed to the berm overflow structures where it will flow (without erosion) to an appropriate channel based on the configuration as defined in Section 5.4 for the channel selection.

5.4.2 **Runoff from The Advancing Face**

Construction of the ADF shall be in phases as to defer capital cost expenditure and allow the management of contaminated runoff water in small areas. The runoff on a newly developed area shall accumulate against the advancing ash face from where it shall either be pumped to the dirty water channels on the west or east of the ADF or it shall be allowed to seep into the leachate collection drains.

Details of the drains and pumping configuration are provided in the design drawings. Development of each new phase (Phase 1 to 11) must be done sufficiently ahead of the advancing ash face to ensure that the required infrastructure is in place capable of containing polluted runoff water.

Under no circumstances shall water be allowed to pond at, or in the immediate vicinity of the toe of the advancing face of the dump for an excessive time as this could reduce the factor of safety and cause the dump to become unstable. Water must be removed using the pumping system if seepage into the leachate collection drains fail to drain the area within 2 days.

Pumping of the water shall be achieved by a mobile pumping unit established during the wet season.

Excessive run-off that is captured within the concrete channel drop boxes would need to be pumped into the ADF cell, over the perimeter berm to be captured within the leachate collection system by means of a mobile pumping unit established during the wet season.

5.4.3 Containment Area

It shall be ensured that there is at all times a containment area available for use at the advancing face. The next area to be used must be constructed well before the area in use is covered in ash.

5.5 Storm water Control In Working Areas

5.5.1 Conveyor Platform

Berms shall be constructed in order to prevent concentrations of runoff and subsequent erosion both at the conveyor platform and in the area over which the conveyor is moved.

6. ROADS

The facility has been designed with a set of permanent roads and temporary roads. The permanent roads comprise of the conveyor service roads which are located both sides of all the conveyors along their entire length. Other permanent roads are as follows:

- E-Discharge
- Operations and Maintenance Facilities
- Electrical Substation Building;
- Interlink between the Top Extendible and Bottom Extendible Conveyor service roads
- Stilling Basins;
- Pollution Control Dams;
- Pump Stations
- ADF ring road.

The roads on the perimeter of the dump will be constructed in phases as the dump progresses. At the end of each phase, temporary roads will be constructed to link the eastern and western sides of the facility. Since temporary roads are constructed in the ashing area for the next phase of the ash body development, it is necessary that new temporary roads should be constructed before any pre-deposition work starts for the next phase.

7. ATTENUATION DAMS AND STREAM DIVERSION PIPELINE

The facility has been designed with attenuation dams located along the existing stream routing beneath the ADF body. These attenuation dams have been designed to direct stormwater run-off into inlet chambers and into the stream diversion pipelines beneath the ADF base. These pipelines range in diameter from 2.0m to 2.5m.

7.1 Inspections

Regular inspections should be carried out on the attenuation dams, inlet / outlet chambers, and the inside of the stream diversion pipeline.

Attenuation Dams

The dams will need to be inspected for function and integrity at least once a quarter, but a monthly inspection is recommended and after a heavy storm event. The inspections are to be done by personnel that is appropriately trained to identify and record any abnormalities or variations. An appropriate monitoring register and inspection procedure must be developed for these inspections.

A high-level dam safety evaluation is recommended to be done on an annual basis with the presence of an independent professional present. The intend of the high-level evaluation is to review the monthly records and allow for a visual inspection of the dam conditions and integrity. The category II dams are also likely to require a formal dam safety evaluation that generally needs to be done every 5 years or as stipulated in the formal communication from the dam safety office.

The attenuation dam structures should be inspected in accordance with the DWS Dam Safety Evaluation and Inspection Report (DW149E).

Pipeline

Forming a critical function in the ADF, the pipeline will require regular inspections throughout the ADF operating life. Isolation of the pipeline (by means of lowering the flap gates on the inlet chamber) and accessing the pipeline (either through the outlet stilling basin or through the outlet chamber access point on the downstream side of the attenuation dams) will need to be done bi-annually. Careful inspection and documentation of the pipelines internal condition should be captured with focus made on the soffit of the pipeline, identifying cracking or signs of stress.

7.2 Maintenance

Regular maintenance of the dams (grass cutting, erosion protection / rehabilitation), pipeline (de-silting) and the chambers (confirming flap gates are still operational and no signs of damage to chambers) is required and should be carried out as and when required throughout the lifespan of the facility.

8. ONGOING CIVIL WORKS

8.1 Anchor Blocks to Conveyors

TBC by Eskom - Anchor blocks are required to stabilise the end points of the shiftable and extendible conveyors. Two types of block can be used:

- Larger block with a pivot is used at the tail station
- Smaller non-pivotable block was used at the head station.

The block at the head station should be an anchor plate. Three anchor plates shall be kept at the dump site. One plate anchors the bottom stacker conveyor, another anchors the top stacker conveyor and the third is kept in readiness for the next conveyor move. When a move is planned the spare anchor plate is buried in the correct position so that when the conveyor is moved it can be anchored immediately. The unused plate is then dug up, cleaned, painted and stored ready to be used for the next conveyor move.

8.2 Silt Traps & Pollution Control Dams (Dam Complex 1-4)

The prime objective of the pollution control dams is:

- to prevent spillage of polluted water into the natural environment, by containing water from the ADF;
- to have sufficient storage capacity for stormwater run-off generated from the impacted areas, from large storms.
- to minimize the need for make-up water for ash disposal at the station by having sufficient water in the PCDs.

8.2.1 **Desilting of the Silt Traps and PCDs**

The level of the silt will gradually rise with each subsequent storm and therefore the capacity of the ponds and silt traps will be decreased. There is allowance for strategic spillways at the dams but over spilling is to be avoided as any significant spillage could lead to flooding of the pump station units and conveyor lines.

The catchment and seepage inflows will flow via lined channels through a silt trap which will allow any suspended solids to settle out before entering into the primary dams basin. This will ensure that the majority of the sedimentation occurs within the silt traps and limit the siltation (and thus cleaning) within the dam basins.

The dam level is controlled by pumping ash water back to the ADF for dust suppression. The Contractor shall at all times liaise closely with the operating staff from Kusile power station to ensure that the water balance in the station, the stability requirements of the ADF and Eskom’s zero effluent discharge philosophy are all adhered to. The Contractor should assist as far as possible with the level control of the PCDs by letting more water off the ADF when the level in the PCD dams drops below 500mm or by retaining more water on the facility when the level exceeds the design top water level before freeboard. The silt level should not exceed 30% of the dam capacity, any indication of sediments building up in the pollution control dams will necessitate the cleaning of the silt traps. The safety and the stability of the ADF will always take preference to any level control issues.

Silt removal should ideally happen during dry seasons, or more frequently if occasion demands, the silt must be removed from the silt traps and dams and the original capacities restored. Access to the dam compartments is provided by means of access ramps with a slope of 1V:8H constructed in each dam compartment and the silt traps have access side slopes of 1V:7H. The base of the dams have a ballast layer/protection layer comprising of mesh ref 395 reinforcing, 300mm thick concrete slab with sufficient strength to withstand the loading of a FEL to undertake silt removal during the dry season. Care must be taken during the cleaning activity to ensure that no undue damage occurs to the protection layer. The operator should also be instructed not to impact the side slopes of the dam/silt trap with the bucket of the machine during the cleaning operation. The recovered silt should be returned to the ash dump in an area where ashing activities are being carried out.

The Contractor must take monthly measurements of the silt levels close to the silt trap overflow position. A more global silt level survey is to be performed every six months at a minimum of four positions per silt trap and dam. The Contractor is to develop their own system to ensure that silt levels are being measured at the same approximate positions every time.

In order to isolate the silt traps, a cherry picker or TLB will be required to raise or lower the sluice isolation gates at the outlet of the silt trap.

8.2.2 **Grass Cutting**

The outer slopes of the earth dam walls will be top-soiled and hydroseeded. Once established, the grass must be cut monthly during the rainy season and any trees or shrubs, which take root on the dam walls must be removed promptly.

8.2.3 **Ongoing Monitoring and Maintenance**

The Contractor must undertake the following checks and present in a format that is acceptable to the Project Manager the status of the following items on a monthly basis:

- Water storage capacity
  - The current water level in each dam and the estimated volume stored (as calculated from dam storage curves).
  - Are all the pumps operational?

  - Average silt levels in the dam (annually).
  - Is adequate storage capacity available for the 1 in 50 year 24 hour event?

- Dam walls
  - Are there any wet spots on the downstream slope?
  - Are there any sign of erosion on the internal or external slopes?
  - Are there any cracks along the crest?
  - Are there any signs of settlement or movement.
  - Are there any shrubs or trees growing on the wall?

- Downstream pollution
  - Has any contaminated water spilled into the environment?
  - If so, was the Project Manager notified immediately thereafter of the volume and the reason for the spill?
  - Has the spillway been impacted in anyway due to the spill?
  - Has the APP been notified of the spill?

- Silt Traps
  - Silt levels in the silt traps
  - Is there adequate retention storage capacity available to satisfy the desilting requirements. (The silt level should not exceed 30% of the dam capacity).
  - Are there any wet spots on the downstream slope?
  - Are the overflows in a good and functional condition?
  - Are there any sign of erosion on the internal or external slopes?
  - Are there any cracks along the crest?
  - Are there any shrubs or trees growing on the wall?

Typical checklists can be found in Appendix K of this document.

8.3 Cleaning Ash Trf Houses & Overland Conveyor Ash Containment Slab

Eskom to Confirm - The cleaning of transfer houses and the overland conveyor ash containment slab is dealt with in detail in Appendix E

8.4 Topsoil Stripping & Replacement

This is an ongoing operation and shall be carried out in accordance with section 8.3 and 8.4.

8.5 Erosion Control

Various measures to combat erosion have been discussed in section 4.3. These measures must be adhered to and any eroded areas of the dump must be rectified as soon as possible to prevent major problems occurring.

8.6 Access Roads & Ramps

Access roads and ramps shall be constructed and extended as and when necessary. This work must be planned well in advance since the roads and ramps must be in place and useable by the time they are required.

8.7 Advancing Ash Faces

The advancing ash face will be susceptible to erosion caused by water from excessive dust suppression operations and by storm water. This situation must be monitored closely and any gullies which may be formed should be filled using ash as soon as possible to avoid any negative effects on the stability of the advancing ash face.

The ash dump should be constructed such that storm water runoff will flow evenly across the surface of the dump and the advancing face. A gully would only be formed where water is allowed to concentrate. ie. A low point in the dump's surface. It is therefore inadequate just to fill the gullies with ash. The area surrounding the gullies must be regraded and built up to the correct levels to prevent recurrence of the problem.

8.8 Offices & Workshops

The offices and workshops made available to the ash dump operator must be maintained in good order and kept clean at all times. The regulations laid down in the MOS Act of 1983 must be adhered to at all times.

9. ENVIRONMENTAL PROCEDURES

Refer to the Environmental Management Programme (EMP) report which is added as an Appendix to this document.

10. ASH REHABILITATION PROCEDURE

10.1 Materials

10.1.1 Chemicals

All chemicals (agricultural remedies) shall be selected, stored, issued and applied in terms of the relevant legislation applicable to agricultural remedies viz. Act 26 of 1947 and Act 15 of 1973.

10.1.2 Herbicides Quality

Herbicides shall comply, as relevant, with the requirements of CKS 362, 415, 428, 430, 514, 537, 546, 550, 572 and 578.

10.1.3 Organic Fertilizers

Compost:

- Compost shall be well matured, friable organic matter and free from weed seed, soil or foreign materials. Spent mushroom compost complying with these requirements shall be acceptable.

Manure:

- Manure shall be well matured kraal manure free from soil, weed seed or other foreign material.

Bone Meal:

- Bone meal shall be finely ground with a minimum content of 4% Nitrogen and 20% Phosphate.

Peatmoss

- Peatmoss shall be commercial peat.

10.1.4 **Seed Mixtures**

Only fresh seed shall be used, of the type specified in clause 8.6.2. Purity of any seed specified shall be not less than 70% with a minimum germination capacity of 60%.

Grass seed mixtures shall be as specified and mixed on site.

10.1.5 **Veld Grass Mulch**

Indigenous veld grass from surrounding areas can be cut and used as a mulching material. Care shall be taken not to damage these areas except for removing the top growth by hand or by a suitable mechanical means.

In general, veld grass should be harvested during the dormant season after seed has set.

10.1.6 **Plant Sizes**

Trees and Shrubs shall comply with the following minimum standards:

- Tree height;
    - 1500 mm - 2000 mm (Excluding container height)
- Shrub height:
    - 1000 mm - 1500 mm (Excluding container height)

Minimum container sizes to be:

- Trees:
    - 20 liters

- Shrubs
  - 4 liters

10.1.7 **Plants**

Plants shall at all times be healthy, shapely and well established. Containerized plants shall not be rootbound. Plants shall grow well and be free from scars or damage, insect pests, diseases or parasites.

Each plant shall be handled, packed and transported in the accepted industry manner for that species or variety and all the necessary precautions shall be taken to ensure that the plants arrive at the site in a proper condition for successful growth.

During delivery to the site, plants shall be adequately protected from damage by sun, wind or other causes.

10.1.8 **Tree Stakes**

Tree stakes should be treated poles (round droppers) complying with SABS 457, 35 mm minimum diameter and 2 400 mm long. These shall be used for both single and multiple staking. Creosoted timber shall not be used.

10.1.9 **Tree Ties**

Tree ties for fixing trees to stakes shall be of plastic, rubber or other similar material which supports the tree in a substantial manner. Ties shall be designed to minimize abrasion and to allow for sufficient space around the tree trunk to permit growth.

10.2 Plant (Equipment)

10.2.1 **General**

Machinery which is to operate over grassed areas should be fitted with suitable low pressure tyres to limit the depth of tracking and amount of compaction of the soil.

10.2.2 **Seeding**

Where seeding is done by mechanical means eg hydro-seeding, the capabilities of the machine used should be proven.

10.3 Stripping Of Topsoil

The estimated volume of material required in order to complete the Pre & Post-Deposition works has been determined based on the construction sequence and available geotechnical data for the site in order to determine the stripping depth for each phase. It must be noted that the material balance may change depending on the volume of material excavated during each phase and the balance should be adjusted to fit the as built construction plan.

Cladding layer shall be 300mm thick and enough material should be stripped to allow continuous rehabilitation of the ADF until the next Pre-Deposition phase. This shall limit open ash areas which generates dust pollution and dirty water. If enough material is not made available for rehabilitation until commencement of the next Pre-Deposition phase the open ash areas will increase which increases the potential for dust pollution and dirty water generation and this can be easily avoided.

10.4 Topsoil Spreading & Site Preparation

When an area becomes available, rehabilitation shall commence as soon as possible.

Prior to topsoil spreading or sodding taking place, the area to be rehabilitated should have smooth and flowing contours without any slacks or hollows where water may accumulate, unless specifically so required.

Side slopes shall have contour berms constructed parallel to the contours to fore-stall the development of gullies caused by stormwater run-off.

Before the placement of sods commences the area should be scarified to a depth of at least 30 mm to give a roughened surface. The sods must be tightly butted against each other and laid in rows with staggered joints. Sodding to be started from the bottom of the slope, and sods are to be firmed down after placement. This is the preferred option which will limit erosion and should eliminate the need for seeding.

Topsoil shall be spread to a thickness of 300 mm over all areas to be rehabilitated, except those areas where veldgrass sods are to be used. (Refer to clause 8.6.3) After spreading, rip or plough the area to a depth of 400 mm minimum (The upper ash layer must be mixed with the lower topsoil layer). Fertilizing, seed sowing and planting should commence before the loosened soil is naturally recompacted again.

10.5 Fertilizing

Testing

- Representative soil samples shall be taken to establish the fertilizer requirements. These tests must be done at an established soil testing laboratory. The results of the tests will determine the proposed fertilizer program.

Application

- Apply fertilizers according to the accepted fertilizer program.
- Fertilizer shall be applied only during dry weather and shall be mixed into the soil within 12 hours of application. Soluble inorganic fertilizer shall be applied not more than two weeks before planting and shall only be applied to growing plants when the leaves are dry.

Note that due to the presence of ash no agricultural or dolomitic lime should be necessary, even though the top-soil to be used may require pH correction.

10.6 Planting Procedure

10.6.1 **Containerized Plants**

The time of planting trees and shrubs shall be dependant upon current land uses surrounding the ash dump. Planting positions and species shall be indicated by the Civil and Building Division's landscape architect.

Plants will be indigenous and chosen from the list below:

- Trees:

  - Acacia caffra
  - Acacia karroo
  - Acacia robusta
  - Acacia sieberana "Woodii"
  - Buddleia saligna
  - Celtis africana
  - Combretum erythrophyllum
  - Cussonia paniculata
  - Dias cotinifolia
  - Dombeya rotundifolia
  - Erythrina lysistemon
  - Euclea crispa
  - Halleria lucida
  - Kiggelaria africana
  - Olea europaea subsp. africana
  - Rhus dentata
  - Rhus lancea
  - Rhus leptodictya
  - Virgilia oroboides
  - Ziziphus mucronata
- Shrubs
  - Bauhinia tomentosa
  - Buddleia auriculata
  - Buddleia salvifolia
  - Diospyros lycoides
  - Grewia occidentalis
  - Mundulea sericea
  - Nuxia congesta
  - Rhamnus prinoides
  - Rhus pyroides
  - Rothmania capensis

Planting of trees and shrubs on the ash dump shall be done in the following manner in the planting positions indicated by the landscape architect. Plant the trees and shrubs in clumps of 3-8 plants ensuring a coverage of at least 25 plants/ha or as indicated by a landscape architect. Use the combination of plant species as shown below and plant in a random fashion as to imitate natural clumps.

Table 11: Proposed Combination of Plant Species

Description	Unit	Value
Trees		
Acacia karroo	Quantity/ha	7
Olea europaea subsp. africana	Quantity/ha	3
Rhus lancea	Quantity/ha	4
Ziziphus mucronata	Quantity/ha	3
Shrubs		
Diospyros lycoides	Quantity/ha	4
Rhus pyroides	Quantity/ha	4
	Total	25

Planting holes shall be excavated square in plan. The edges of the holes shall be roughened to ensure future root penetration into the ash. Round holes are not to be accepted.

Planting hole dimensions shall be as follows:

- 1 000 mm x 1 000 mm wide x 1000 mm deep, with the provision that holes shall be large enough to provide a clearance of at least 150 mm around the root ball when planted.
- The plant holes are to be back-filled with approved topsoil and additional fertilizers.
- Apply to each hole, in addition to the normal fertilizing program:

  - 200 dm3 compost
  - 100 gms 2:3:2 fertilizer
  - 350 gms bone meal

- The added fertilizer must be incorporated in the backfill soil in the vicinity of the plant's root ball.
- Plants brought out of storage for planting shall be well watered before removal, shall be brought out at a rate commensurate with the planting program, protected from drying out and shall be planted on the same day.
- Polythene or other non perishable containers shall be carefully removed immediately before planting with as little disturbance as possible to the root ball, and any damaged roots carefully pruned. Balled and burlapped plants shall be placed in their wrapped ball, the burlap being folded back and removed just before final backfilling of the plant hole.
- Plants shall be planted in moist but not waterlogged conditions. Loose roots shall be arranged naturally and not be matted and soil shall be worked in amongst them (especially in the case of ex-open-ground material). The hole shall be finally backfilled and lightly foot compacted and a round depression equal in diameter to the width of the hole and approximately 75mm deep, constructed around each plant.

Staking:

- Trees shall be supported and protected by means of a stake, or stake system of the materials.
- Placing of stakes, backfilling, compacting and tree planting shall be done in one operation.

Tying:

- Each tree shall be firmly secured to the stake or system of staking at, two points, at least, to prevent excessive movement. Tying shall be in accordance with the accepted method used in the local climatic conditions.

10.6.2 **Seeding**

Seeding shall include the sowing of veld grass seeds on the ash dump area as well as any other areas that are to be rehabilitated.

Tree and shrub seeds can also be incorporated with the veld grass seed mixture. Seed shall be suitably pre-treated.

The seed mixture can be as shown Table 12. The aim is to establish a veld grass type similar to that of the surrounding natural veld grass areas.

Table 12: Proposed Grass Seeding

Description	Unit	Value
Chloris gyana (Rhodes)	Quantity/ha	10
Cynodon dactylon ("Kweek")	Quantity/ha	10
Digitaria eriantha ("Smutsvinger")	Quantity/ha	7
Eragrotis curvula	Quantity/ha	5
Eragrotis teff ("Oulandsgras")	Quantity/ha	3
Total	Quantity/ha	35

Time of sowing:

- All seeds (grass mixtures, shrub and tree seeds) should be sown during the period **15 September to 15 November**. This should ensure sufficient establishment time for the seedlings.

Seeding Procedure:

- Seed sowing must commence directly after the soil preparation and before the loosened soil is compacted again. The area to be sown shall be uniformly damp, but not water-logged, to at least 100 mm prior to seeding.

- Seeding can be done by fluid drilling, hydro-seeding or other acceptable means. Where hydroseeding is proposed the mixture shall include cellulose pulp or mulch at the rate of 25kg/kilolitre of water used.

10.6.3 **Sodding**

When veld grass sods are available it should be used in preference to seeding to establish a grass cover. Prior to the placement of the sods, the area should be scarified to a depth of at least 30 mm to give a roughened surface. The sods must be tightly butted against each other and laid in rows with staggered joints. Sodding to be started from the bottom of the slope, and sods are to be firmed down after placement.

Fill spaces with topsoil to ensure minimum thickness of 300 mm over the ash by spreading topsoil as necessary. This is the preferred option which will limit erosion, and which should eliminate the need for seeding.

10.6.4 **Topsoiling Only**

Where it is considered that the spread topsoil contains sufficient grass and seeds and climatic and other conditions are favourable, no additional seeding may be necessary. This may be the case for rehabilitation of disturbed areas surrounding the ash dump.

10.7 Irrigation

Water to be used for irrigation should not exceed the specified water quality standards as described in Table 13. Water that is sampled in the Energy Dissipation Structures should adhere to these standards before it is released to the Clean Water Dams or directly released to the environment.

Table 13: Water Polluted

Description	Unit	Value
Water conductivity	US/cm	2250
Sodium Absorption Ratio (* SAR)	Mg/litre	18
Boron concentration	Mg/litre	0.75

Description	Unit	Value
Other trace elements Concentrations	N/A	Refer to Table 14

$$* \quad SAR = \frac{Na\,[\,1 + (\,8.4 - Phc)]}{(\,0.5\,(\,Ca + Mg))*0.5}$$

Na, Ca and Mg in mg/liter and pHc is calculated as shown below:

- pHc = (pK'2 - pK'c) + p(Ca+Mg) + p(Alk)

  - Where:

    - pK'2 - pK'c = Ca+Mg+Na in Mg/liter
    - p(Ca+Mg) = Ca+Mg in Mg/liter
    - p(Alk) = CO3+HCO3 in Mg/liter
    - Substitute the calculated sums of concentrations values with the values from Table 15 for the final calculations of the pHc value.
    - Trees & Shrubs (Nursery Stock)

Table 14: Recommended Maximum Concentration Value of Trace Elements in Irrigation Water

Element (Symbol)	Unit	Continues Use on Any Soil Type	Up to 30 year Continues Use on Fine Soil Types of pH 6.0 to 8.5
Al	Mg/litre	5,0	20,0
As	Mg/litre	0,1	2,0
Be	Mg/litre	0,1	0,5
B	Mg/litre	0,75	3,0-10,0
Cd	Mg/litre	0,01	0,05
Cr	Mg/litre	0,1	1,0
Co	Mg/litre	0,05	5,0
Cu	Mg/litre	0,2	5,0
F	Mg/litre	1,0	15,0
Fe	Mg/litre	5,0	20,0
Pb	Mg/litre	5,0	10,0
Li	Mg/litre	2,5	2,5
Mn	Mg/litre	0,2	10,0
Mo	Mg/litre	0,01	0,05
Ni	Mg/litre	0,2	2,0
Se	Mg/litre	0,02	0,02
V	Mg/litre	0,1	1,0
Zn	Mg/litre	2,0	10,0

Table according to Van der Nest, 1991

Table 15: Calculation of the pHc

Sum of Concentration	Unit	PK'2 – PK'c	p(Ca + Mg)	p(Alk)
0,05	Mg/litre	2,0	4,6	4,3
0,10	Mg/litre	2,0	4,3	4,0
0,15	Mg/litre	2,0	4,1	3,8
0,20	Mg/litre	2,0	4,0	3,7
0,25	Mg/litre	2,0	3,9	3,6
0,30	Mg/litre	2,0	3,8	3,5
0,40	Mg/litre	2,0	3,7	3,4
0,50	Mg/litre	2,1	3,6	3,3
0,75	Mg/litre	2,1	3,4	3,1
1,00	Mg/litre	2,1	3,3	3,0
1,25	Mg/litre	2,1	3,2	2,9
1,5	Mg/litre	2,1	3,1	2,8
2,0	Mg/litre	2,2	3,0	2,7
2,5	Mg/litre	2,2	2,9	2,6
3,0	Mg/litre	2,2	2,8	2,5
4,0	Mg/litre	2,2	2,7	2,4
5,0	Mg/litre	2,2	2,6	2,3
6,0	Mg/litre	2,2	2,5	2,2
8,0	Mg/litre	2,3	2,4	2,1
10,0	Mg/litre	2,3	2,3	2,0
12,5	Mg/litre	2,3	2,2	1,9
15,0	Mg/litre	2,3	2,1	1,8
20,0	Mg/litre	2,4	2,0	1,7
30,0	Mg/litre	2,4	1,3	1,5
50,0	Mg/litre	2,5	1,6	1,3

Sum of Concentration	Unit	PK'2 – PK'c	p(Ca + Mg)	p(Alk)
80,0	Mg/litre	2,5	1,4	1,1

Table according to Van der Nest, 1991

10.7.1 Grass Areas

To achieve successful grass establishment all newly seeded areas should be irrigated at a rate of at least 25 mm/week for the first 3 to 4 months until successful cover has been obtained. An efficient irrigation system is necessary at all times to supplement natural rainfall, during the first 4 months. Water logging is to be avoided, especially during the first 8 weeks.

Although the intention is to establish a grass cover which will not need additional irrigation following establishment, additional irrigation may be required in periods of drought to prevent rehabilitated grass areas from becoming patchy.

10.7.2 Trees & Shrubs (Nursery Stock)

Plants shall be irrigated manually or using an irrigation system within 2 hours of planting, each plant receiving at least 50 liters.

Thereafter, plants shall be watered at least weekly during the first growing season or until fully established.

10.8 After Planting Care

Newly seeded/planted areas should be protected against undue traffic and/or other disturbances throughout the establishment period but especially during the first two growing seasons so as to achieve settlement of plants and acceptable grass cover.

10.9 Maintenance

Rehabilitated areas should be adequately maintained. Maintenance shall include:

- Continual repair of damage caused by erosion or any other cause.
- Replacement of dead or damaged plants with the same species as originally planted
- Upkeep of acceptable grass cover with reseeding as necessary
- Mowing of grass on a once yearly basis during Autumn to ensure a sufficient diversity of grass species
- Additional tree/shrub planting as may be required.
- Follow up fertilizing based upon soil tests taken at least once every year.

11. MAINTENANCE PROCEDURES

Regular maintenance must be carried out throughout the ash dump to maintain full and cost-effective use of the facility.

11.1.1 **Checklists**

Various aspects of the ash dump facility must be checked on a regular basis to ascertain whether any remedial works are necessary. Such checking must be marked on a checklist so that variances can easily be spotted. Appendix H shows an example of a checklist. Any additional items may be added to the checklist.

11.1.2 **Maintenance Requirements**

It is of no use carrying out and making records of these checks if immediate remedial action is not taken to rectify the problems. The maintenance cost over the life of the ash dump could be very high and therefore any remedial work should be well planned and cost effective. The advantages of a cheap, short life span remedy should be weighed against the advantages of a more expensive but durable remedy.

12. LEGAL AND SAFETY ASPECTS

An ash dump can be an unsafe place and unauthorised persons should not be allowed onto it. Preventing public access will reduce much of the legal risk associated with the ash dump.

Safety on the ash dump should be constantly addressed and any dangerous working situations should be avoided. Personnel should be kept informed as to safe working practices and all regulations pertaining to the ash disposal site must be adhered to.

Compliance with the above measures should ensure that a good safety record is maintained at the ash disposal facility.

12.1.1 Statutory Requirements, Regulations & Standards

The following statutes have relevance to the ash disposal facility and shall be adhered to at all times:

- The Mines and Works Act (Act 27 of 1956)
- The Water Act (Act 54 of 1956)
- The Environment Conservation Act (Act 100 of 1982)
- The Conservation of Agricultural Resources Act (Act 43 1983)
- The Agricultural Pests Act (Act 36 of 1983)
- The Health Act (Act 63 of 1977)
- National Environmental Management: Waste Act (Act 59 of 2008)
- National Environmental Management: Waste Amendment Act (Act 29 of 2014)

Any amendments or additions to the above must be adhered to from the time they are promulgated.

In addition, all Eskom regulations shall be adhered to.

The applicable SABS 1200 standards shall apply to civil works carried out at the ash disposal facility.

13. SURVEY & SETTING OUT DETAILS

13.1.1 **Control Beacons & Bench Marks**

Details and drawings of the positions of control beacons and bench marks that will be necessary to build the dump will be supplied to the Contractor from available survey as per Section 13.1.2.

13.1.2 **Topographical Surveys**

A LiDAR topographical survey was undertaken by Southern Mapping Geospatial (SMG) in May 2013 as part of the Concept Design phase of the project for the EIA submission.

The survey was carried out using an aircraft mounted LiDAR system that scanned the ground and the data was used to produce a digital terrain model (DTM). The following deliverables were produced:

- CADdesign files in Microstation DGN & DWG formats showing:
  - Ortho-photo tiles layout
  - LiDAR point block layout
  - Contours at 0.5m, 1m and 5m intervals
  - The project area surveyed with boundaries
- Ortho-rectified aerial images with a 15cm pixel resolution
- Composite images of the total area survey with a 0.5m pixel resolutions.
- Google Earth Overlay in kmz format.
- Thinned ground and non-ground LiDAR points in ASCII format.
- Survey reports.

All survey data is based on the **Hartebeesthoek94 WG29 coordinate system** which is currently the official geodetic datum for South Africa.

14. ASH DUMP MANAGEMENT

Kusile Power Station Generation will manage the operation of the ash dump, contracting out the construction as and when required. It is however envisaged that, in order to ensure that construction is carried out according to the designs and to provide an ongoing civil technical advisory service, regular progress meetings, chaired by Kusile Generation, will be held including personnel from the Civil and Building Division and the ash dump operators. This will allow the designers reasonable access to the construction operations and problems and allow professional engineering design responsibility for the design to be borne with an acceptable level of risk. The station must ensure that the necessary funds are made available for regular ash dump inspections and coordination meetings, as well as investigations and remedial work as and when necessary, to ensure both the short term safety and availability of the ashing facility, as well as its long term health.

It is envisaged that regular monthly ash dump inspections and coordination meetings between the parties will be required to allow the professional civil designer to remain in touch with the ongoing construction process and problems. Weekly inspections by the Kusile engineering civil person will be required to ensure that critical situations do not go undetected or get worse in the relatively long interval between monthly inspections by the professional responsible person. Feedback by telephone or email would suffice to alert the responsible person as to the development of critical situations needing special attention. The Kusile Operating ashing contract personnel who would be present on the dump a few times a week must also do cursory visual inspections, reporting to the Kendal civil engineering person if they pick up any problems needing further engineering investigation or rectifying. Similarly, the contractor’s ash dump construction supervisor and personnel who would be present on the dump on a daily basis should be instructed to report any problems of a serious nature which they cannot address themselves.

15. REFERENCES

- Design Review - Technical Report, Report No. 22020-T01-CE-RPT-001 Rev B, by EPCM, 12 April 2022.
- Design Review - Technical Report, Report No. 22020-T01-CE-RPT-001 Rev B, by EPCM, 12 April 2022.

- Application for Water Use Licence for the Kusile 60 Year Ash Disposal Facility at Kusile Power Station, Mpumalanga, Report no. 12712-46-Rep-001 Kusile IWULA-Rev0, by Zitholele Consulting (Pty) Ltd, 6 November 2015.
- Request for Approval of Design Drawings for Eskom SOC Kusile Power Station 60 Year Ash Disposal Facility in Terms of Section 50 of the National Environmental Management Act: Waste Act, 2008 – Mpumalanga Province, Ref No. 12/9/11/L193/6 by DFFE 9 June 2021.
- Waste Classification Report no. JW030/13/D121-Rev3 by Jones and Wagener Consultants in January 2014,
- Application for Water Use Licence for the Kusile 60 Year Ash Disposal Facility at Kusile Power Station, Mpumalanga, Report no. 12712-46-Rep-001 Kusile IWULA-Rev0, by Zitholele Consulting (Pty) Ltd, 6 November 2015.
- Van der Nest, L J, Desember 1991, *Besproeiingswaterstandaarde vir veldgrasaanplantings op kragstasie Asstapel,* Navorsingsinstituut vir Hervestigingsekologie, PU vir CHO,
- Hodgson, D I, November 1987, *Report on the installation of a water quality monitoring system at the Kendal Power Station,* Eskom
- Kreuiter, A, May 1991, *Report on the dust suppression on the Dry Ash Dump and trials using AECI Chemicals and Soil Spreading methods,* Matimba Power Station, Report No G 043 A, Eskom
- Ramdayal, N & Owens-Collins, S, Dec 2016, *Kusile Ash Disposal Facility Geotechnical Investigation Interpretive Report*, Kusile Power Station, Report Number: 1651909-307604-1, Golder
- Kaponda T. D, *Detail Design Report for Kusile Power Station 60 Year Ash Disposal Facility,* Kusile Power Station, Report No 366-146714, Eskom
- Hodgson, F S and Grobbelaar, R, October 1996, Pozzolanic Characteristics and Rain Water Infiltration at the Kendal Fly Ash Dump, Eskom Technology Group, Report No TRR/S96/142, Institute for Ground Water Studies
- Kreuiter, A and Falla, J F, *Stability Review of Ash Dump,* Kendal Power Station, Report No 23CBKE013A, GED Civil & Building division, In Progress

- Govender, D S, 21 January 1999, Dry Ash Disposal Facility – 1998 Stormwater Management Review, Kendal Power Station, Report No 23CBKE020A, GED Civil & Building division

APPENDIX A: ASH DUMP DEVELOPMENT CONSTRUCTION DRAWINGS

APPENDIX B: WATER QUALITY MONITORING POINTS

APPENDIX C: BOREHOLE MONITORING RECORDS

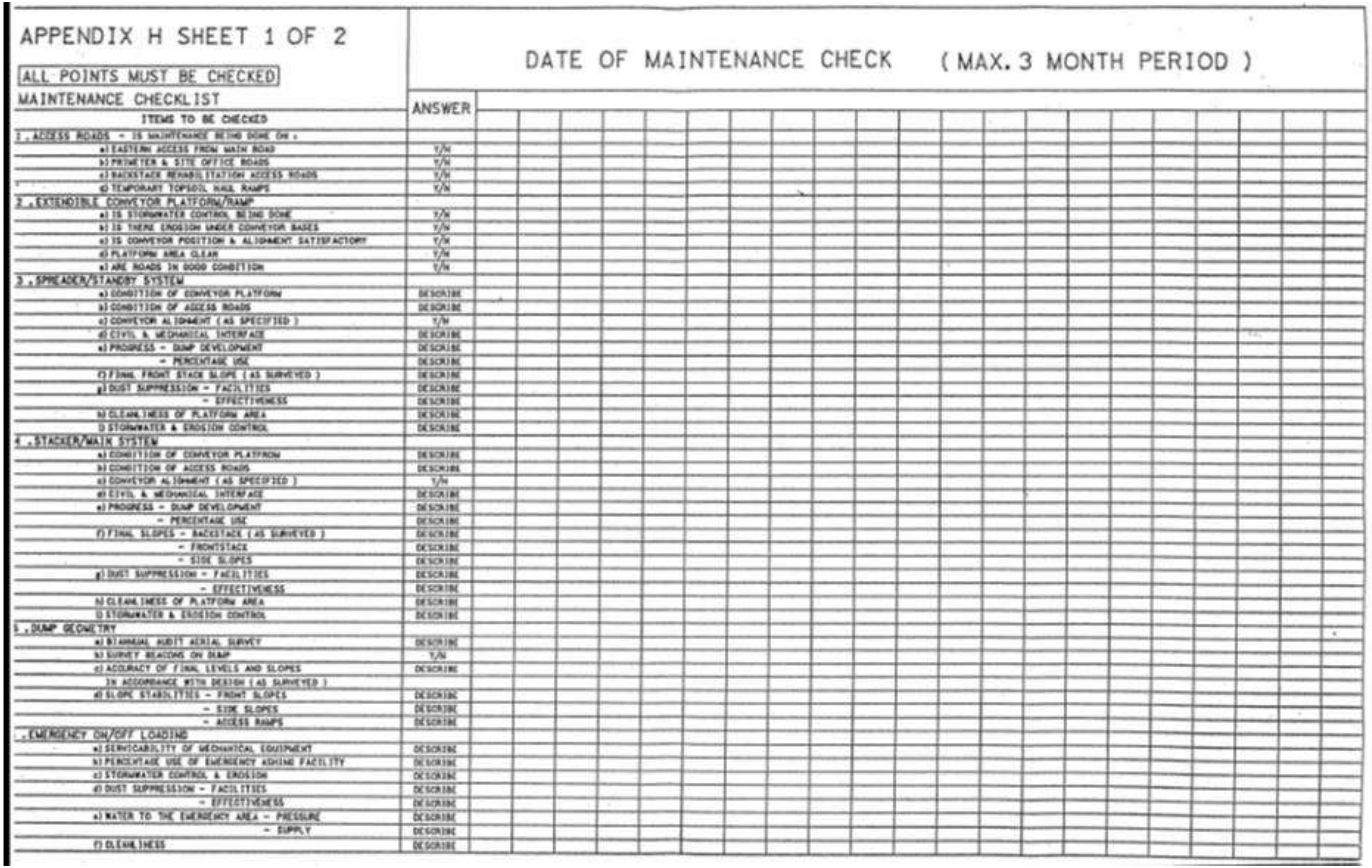
APPENDIX D: ASH DUMP IRRIGATION AND DUST SUPPRESSION LAYOUTS

APPENDIX E: CLEANING ASH TRANSFER HOUSES AND OVERLAND CONVEYOR ASH CONTAINMENT SLAB

APPENDIX F: SURVEY RECORDS

APPENDIX G: RECORD OF ANNUAL ASH STRENGTH TESTING

APPENDIX H: MAINTENANCE CHECKLIST

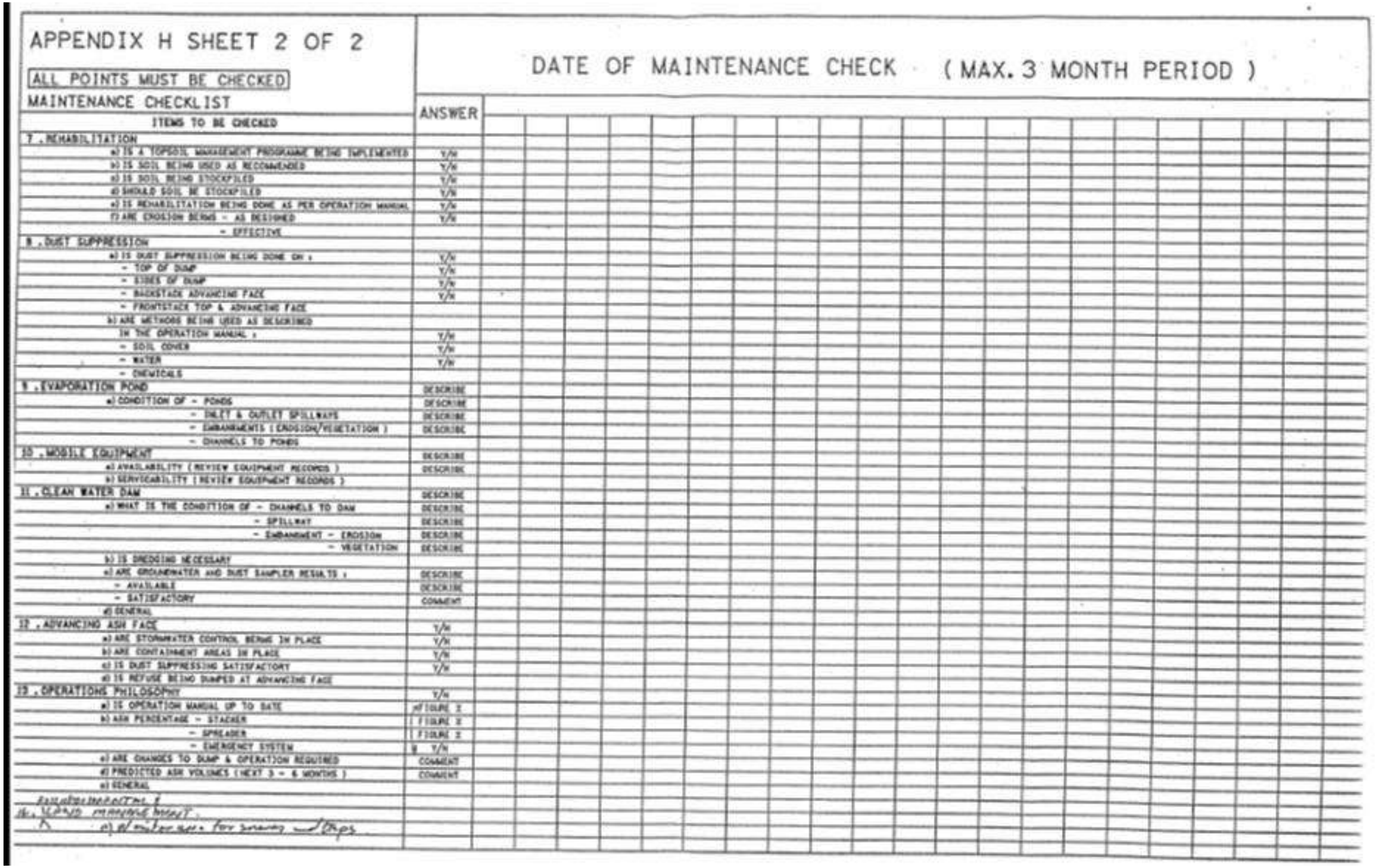


APPENDIX H SHEET 1 OF 2

ALL POINTS MUST BE CHECKED

MAINTENANCE CHECKLIST

ITEMS TO BE CHECKED	ANSWER	DATE OF MAINTENANCE CHECK (MAX. 3 MONTH PERIOD)																				
1 . ACCESS ROADS - IS MAINTENANCE BEING DONE ON :																						
a) EASTERN ACCESS FROM MAIN ROAD	Y/N																					
b) PRIMETER & SITE OFFICE ROADS	Y/N																					
c) BACKSTACK REHABILITATION ACCESS ROADS	Y/N																					
d) TEMPORARY TOPSOIL HAUL RAMPS	Y/N																					
2 . EXTENDIBLE CONVEYOR PLATFORM/RAMP																						
a) IS STORMWATER CONTROL BEING DONE	Y/N																					
b) IS THERE EROSION UNDER CONVEYOR BASES	Y/N																					
c) IS CONVEYOR POSITION & ALIGNMENT SATISFACTORY	Y/N																					
d) PLATFORM AREA CLEAR	Y/N																					
e) ARE ROADS IN GOOD CONDITION	Y/N																					
3 . SPREADER/STANDBY SYSTEM																						
a) CONDITION OF CONVEYOR PLATFORM	DESCRIBE																					
b) CONDITION OF ACCESS ROADS	DESCRIBE																					
c) CONVEYOR ALIGNMENT (AS SPECIFIED)	Y/N																					
d) CIVIL & MECHANICAL INTERFACE	DESCRIBE																					
e) PROGRESS - DUMP DEVELOPMENT	DESCRIBE																					
- PERCENTAGE USE	DESCRIBE																					
f) FINAL FRONT STACK SLOPE (AS SURVEYED)	DESCRIBE																					
g) DUST SUPPRESSION - FACILITIES	DESCRIBE																					
- EFFECTIVENESS	DESCRIBE																					
h) CLEANLINESS OF PLATFORM AREA	DESCRIBE																					
i) STORMWATER & EROSION CONTROL	DESCRIBE																					
4 . STACKER/MAIN SYSTEM																						
a) CONDITION OF CONVEYOR PLATFROM	DESCRIBE																					
b) CONDITION OF ACCESS ROADS	DESCRIBE																					
c) CONVEYOR ALIGNMENT (AS SPECIFIED)	Y/N																					
d) CIVIL & MECHANICAL INTERFACE	DESCRIBE																					
e) PROGRESS - DUMP DEVELOPMENT	DESCRIBE																					
- PERCENTAGE USE	DESCRIBE																					
f) FINAL SLOPES - BACKSTACK (AS SURVEYED)	DESCRIBE																					
- FRONTSTACK	DESCRIBE																					
- SIDE SLOPES	DESCRIBE																					
g) DUST SUPPRESSION - FACILITIES	DESCRIBE																					
- EFFECTIVENESS	DESCRIBE																					
h) CLEANLINESS OF PLATFORM AREA	DESCRIBE																					
i) STORMWATER & EROSION CONTROL	DESCRIBE																					
5 . DUMP GEOMETRY																						
a) BIANNUAL AUDIT AERIAL SURVEY	DESCRIBE																					
b) SURVEY BEACONS ON DUMP	Y/N																					
c) ACCURACY OF FINAL LEVELS AND SLOPES	DESCRIBE																					
IN ACCORDANCE WITH DESIGN (AS SURVEYED)																						
d) SLOPE STABILITIES - FRONT SLOPES	DESCRIBE																					
- SIDE SLOPES	DESCRIBE																					
- ACCESS RAMPS	DESCRIBE																					
. EMERGENCY ON/OFF LOADING																						
a) SERVICABILITY OF MECHANICAL EQUIPMENT	DESCRIBE																					
b) PERCENTAGE USE OF EMERGENCY ASHING FACILITY	DESCRIBE																					
c) STORMWATER CONTROL & EROSION	DESCRIBE																					
d) DUST SUPPRESSION - FACILITIES	DESCRIBE																					
- EFFECTIVENESS	DESCRIBE																					
e) WATER TO THE EMERGENCY AREA - PRESSURE	DESCRIBE																					
- SUPPLY	DESCRIBE																					
f) CLEANLINESS	DESCRIBE																					



APPENDIX H SHEET 2 OF 2

ALL POINTS MUST BE CHECKED

MAINTENANCE CHECKLIST

ITEMS TO BE CHECKED	ANSWER	DATE OF MAINTENANCE CHECK (MAX. 3 MONTH PERIOD)																				
7 . REHABILITATION																						
a) IS A TOPSOIL MANAGEMENT PROGRAMME BEING IMPLEMENTED	Y/N																					
b) IS SOIL BEING USED AS RECOMMENDED	Y/N																					
c) IS SOIL BEING STOCKPILED	Y/N																					
d) SHOULD SOIL BE STOCKPILED	Y/N																					
e) IS REHABILITATION BEING DONE AS PER OPERATION MANUAL	Y/N																					
f) ARE EROSION BERMS - AS DESIGNED	Y/N																					
- EFFECTIVE																						
8 . DUST SUPPRESSION																						
a) IS DUST SUPPRESSION BEING DONE ON :	Y/N																					
- TOP OF DUMP	Y/N																					
- SIDES OF DUMP	Y/N																					
- BACKSTACK ADVANCING FACE	Y/N																					
- FRONTSTACK TOP & ADVANCING FACE																						
b) ARE METHODS BEING USED AS DESCRIBED																						
IN THE OPERATION MANUAL :	Y/N																					
- SOIL COVER	Y/N																					
- WATER	Y/N																					
- CHEMICALS																						
9 . EVAPORATION POND	DESCRIBE																					
a) CONDITION OF - PONDS	DESCRIBE																					
- INLET & OUTLET SPILLWAYS	DESCRIBE																					
- EMBANKMENTS (EROSION/VEGETATION)	DESCRIBE																					
- CHANNELS TO PONDS																						
10 . MOBILE EQUIPMENT	DESCRIBE																					
a) AVAILABILITY (REVIEW EQUIPMENT RECORDS)	DESCRIBE																					
b) SERVICEABILITY (REVIEW EQUIPMENT RECORDS)																						
11 . CLEAN WATER DAM	DESCRIBE																					
a) WHAT IS THE CONDITION OF - CHANNELS TO DAM	DESCRIBE																					
- SPILLWAY	DESCRIBE																					
- EMBANKMENT - EROSION	DESCRIBE																					
- VEGETATION	DESCRIBE																					
b) IS DREDGING NECESSARY																						
c) ARE GROUNDWATER AND DUST SAMPLER RESULTS :	DESCRIBE																					
- AVAILABLE	DESCRIBE																					
- SATISFACTORY	COMMENT																					
d) GENERAL																						
12 . ADVANCING ASH FACE	Y/N																					
a) ARE STORMWATER CONTROL BERMS IN PLACE	Y/N																					
b) ARE CONTAINMENT AREAS IN PLACE	Y/N																					
c) IS DUST SUPPRESSING SATISFACTORY	Y/N																					
d) IS REFUSE BEING DUMPED AT ADVANCING FACE																						
13 . OPERATIONS PHILOSOPHY	Y/N																					
a) IS OPERATION MANUAL UP TO DATE	% FIGURE																					
b) ASH PERCENTAGE - STACKER	% FIGURE																					
- SPREADER	% FIGURE																					
- EMERGENCY SYSTEM	Y/N																					
c) ARE CHANGES TO DUMP & OPERATION REQUIRED	COMMENT																					
d) PREDICTED ASH VOLUMES (NEXT 3 - 6 MONTHS)	COMMENT																					
e) GENERAL																						
Environmental &																						
14. [illegible] management.																						
a) [illegible] for [illegible] steps																						

APPENDIX I: MATERIAL BALANCE BASED ON DETAILED DESIGN

APPENDIX J: DAM STORAGE CURVES

APPENDIX K: DAM CHECKLISTS

TYPICAL POLLUTION CONTROL DAM INSPECTION CHECKLIST & NOTES

THREE MONTHLY ROUTINE INSPECTION REPORT FOR

DAM NAME: Kusile Power Station's Pollution Control Dam 1A, 1B,2,3,4,5,6, Road (circle the appropriate dam being inspected)

DATE: ………………………………………………

TABLE 1: PARTICULARS OF THE DAM

Category classification	
Maximum wall height (m)	
Full supply level (m)	
Storage capacity at FSL (m^3)	
Crest length of wall (m)	
Spillway width (m)	
Foundation description	
Description of dam wall	

1. DAM WALL

1.1 Rainfall data (when last rained and how much):

……………………………………………………………………………………….………………………………………
…

1.2 Water level in the dam at time of inspection:

…….……
…

1.3 Dam Walls	Wall / crest	Upstream Slope	Downstream Slope	Downstream Area
Hollow or low points?				
Settlement/subsidence?				
Bulges/wall movements?				
Undercutting?				
Cracks?				
Holes (ants, animals)?				
Trees or shrubs?				
Grass in good condition?				
Condition of liner?				
Any erosion?				
Wet patches/Seepage?				
Amount of seepage water?				
Seepage water dirty/turbid?				
Additional Items				

2. CONCRETE STRUCTURES

2.1 Is the concrete in good condition?

……………………………………………………………………..…………………………………………………………
…

2.2 Is there serious erosion or undercutting at the concrete structure?

……………………………………………………………………………………………………..………………………………
…

2.3 Any serious cracks?

…………………………………………………………………………………..……………………………………………
…

2.4 Any wall movement or relative movement at joints or cracks?

……
…

2.5 Any signs of material particles moving through cracks?

……
…

SPILLWAY STRUCTURES

2.6 Is there serious erosion in spillway or spillway channel?

……
…

2.7 Is the spillway channel blocked by reeds, shrubs, trees or anything else?

……
…

2.8 Is there any undercutting of concrete structures?

……
…

2.9 Are concrete structures in good condition?

……
…

3. OUTLET WORKS

3.1 Are there any leakages in or outside the outlets works?

....

3.2 Are pipes, valves, all equipment and paintwork in a good condition?

....

3.3 Are all valves in smooth working condition?

....

4. SUB-SOIL DRAIN SYSTEM

4.1 Are there any leakages or seepage visible at the manhole(s)?

....

4.2 Is water flowing into the manhole free of solids?

....

4.3 Is the water flowing freely and reporting to the sump?

....

4.4 Is the pump sump pump, pipes, valves, all equipment and paintwork in a good condition?

……
…

5. RECOMMENDATIONS

5.1 Which of the above-mentioned points or any other should urgently be rectified?

……
……
……
……
……
……
………………

NAME OF INSPECTOR: ……………………………………

SIGNATURE OF INSPECTOR: ……………………………….DATE: …………………

NAME OF OWNER: ………………………………………..

SIGNATURE OF DAM OWNER: ……………………………..DATE: …………………

TYPICAL CLEAN WATER DAM INSPECTION CHECKLIST & NOTES

THREE MONTHLY ROUTINE INSPECTION REPORT FOR

DAM NAME: Kusile Power Station's Clean Water Dam (CWD) 1,2
(circle the appropriate dam being inspected)

DATE: ....

TABLE 1: PARTICULARS OF THE DAM

Category classification	
Maximum wall height (m)	
Full supply level (m)	
Storage capacity at FSL (m^3)	
Crest length of wall (m)	
Spillway width (m)	
Foundation description	
Description of dam wall	

1. DAM WALL

1.1 Rainfall data (when last rained and how much):

....
...

1.2 Water level in the dam at time of inspection:

……….......
...

1.3 Dam Walls	Wall / crest	Upstream Slope	Downstream Slope	Downstream Area
Hollow or low points?				
Settlement/subsidence?				
Bulges/wall movements?				
Undercutting?				
Cracks?				
Holes (ants, animals)?				
Trees or shrubs?				
Grass in good condition?				
Condition of liner?				
Any erosion?				
Wet patches/Seepage?				
Amount of seepage water?				
Seepage water dirty/turbid?				
Additional Items				

2. CONCRETE STRUCTURES

2.1 Is the concrete in good condition?

………………………………………………………………………..………………………………………………………… …

2.2 Is there serious erosion or undercutting at the concrete structure?

………………………………………………………………………………………………………..…………………………… …

2.3 Any serious cracks?

……………………………………………………………………………………..………………………………………………… …

2.4 Any wall movement or relative movement at joints or cracks?

....
...

2.5 Any signs of material particles moving through cracks?

....
...

SPILLWAY STRUCTURES

2.6 Is there serious erosion in spillway or spillway channel?

....
...

2.7 Is the spillway channel blocked by reeds, shrubs, trees or anything else?

....
...

2.8 Is there any undercutting of concrete structures?

....
...

2.9 Are concrete structures in good condition?

………
…

3. OUTLET WORKS

3.1 Are there any leakages in or outside the outlets works?

………
…

3.2 Are pipes, valves, all equipment and paintwork in a good condition?

………
…

3.3 Are all valves in smooth working condition?

………
…

4. SUB-SOIL DRAIN SYSTEM

4.1 Are there any leakages or seepage visible at the manhole(s)?

………
…

4.2 Is water flowing into the manhole free of solids?

……..

4.3 Is the water flowing freely and reporting to the sump?

……
…

4.4 Is the pump sump pump, pipes, valves, all equipment and paintwork in a good condition?

……
…

5. RECOMMENDATIONS

5.1 Which of the above-mentioned points or any other should urgently be rectified?

……
……
……
……
……
……
………………

NAME OF INSPECTOR: ……………………………………

SIGNATURE OF INSPECTOR: ………………………………….DATE: …………………

NAME OF OWNER: …………………………………………..

SIGNATURE OF DAM OWNER: ……………………………..DATE: …………………

Inspection Notes

Dam Wall

The slopes of the dam must be inspected for any signs of seepage, cracks, movement, erosion, ants nests and animal burrows. Plant growth on the embankment must be kept short. No trees or shrubs must be allowed on the embankment.

The extent of wet patches on the downstream side of the dam must be marked with pegs to enable changes to be observed. The position relative to the crest and the distance along the crest must be recorded. Seepage water must be checked for the presence of soil particles.

The position, length and width of significant cracks on the embankment must be marked and recorded. Transverse or diagonal cracks running from upstream to downstream or any rapid changing cracks must always be viewed with suspicion. The appearance, location and extent of any movement, erosion and caving must be observed and recorded. If survey beacons have been disturbed it must be noted in the report.

Spillway

The spillway and the reno mattress down chute erosion protection must be inspected. Erosion of the spillway channel and undercutting of concrete or gabion structures must be recorded and repaired.

Liner

Visible liner on the side slopes to be visually inspected for tears, holes, damage etc.

Manholes

The underdrains under the dam feed into manholes. The flow in these pipes is to be checked visually monthly as well as the volumes, from pump data or flow meters. If excessive quantities of water are recorded a water quality sample should be taken to check if the liner is leaking excessively.

Downstream area

A strip with a width equal to twice the height of the dam, measured from the toe of the embankment, must be inspected. The presence of seepage water, marshy conditions, pools, cracks or any displacement must be observed and recorded and the position noted. Trees or shrubs within a 20m wide strip downstream of the toe of the wall must be removed.

Pump station

The pump station on the dam must be inspected at weekly intervals. The pumps, motors and other electrical equipment must be inspected according to the recommendations of the manufacturer. Flowmeters must also be inspected to ensure that they are in a serviceable condition. A log book must be kept to record inspections.

APPENDIX L: ENVIRONMENTAL MANAGEMENT PROGRAMME FOR THE KUSILE POWER STATION 60 YEAR ASH DISPOSAL FACILITY

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Contractor document no.:	366-513593
Document type:	Technical Report
Contractor Name:	EPCM
Revision no.:	0.2
Prepared by	KP Matulovich
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Unit/s no.:	Balance of Plant
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Rev	Date	Document Status	EPCM Reviewed	EPCM Approved	Client Review/ Approval
0	31 January 2024	Issued for Construction	Samuel du Rand Megan Hardwick Van Wyk Botha	Samuel du Rand Signature	Signature
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CONTENTS

List of Figures

List of Tables

Abbreviations

ADF	Ash Disposal Facility
AWBM	Australian Water Balance Model
CWD	Clean Water Dam
DWS	Department of Water & Sanitation
EPCM	EPCM Consultants SA
FSL	Full Supply Level
HRD	Holding Recycling Dam
LPS	Low Pressure Service
MAE	Mean Annual Evaporation
MAP	Mean Annual Precipitation
NWA	National Water Act
PCD	Pollution Control Dam
PS	Power Station
TWL	Top Water Level
WB	Water Balance
WMA	Water Management Area

1. TERMS OF REFERENCE

EPCM Consultants SA (Pty) Ltd has been appointed by Eskom Holdings (SOC) Ltd to review existing design, develop Detailed Design for Phase 1 and consolidate all required information for the 60-year Ash Disposal Facility Design Report, to support the amended Water Use Licence Application.

2. BACKGROUND INFORMATION

2.1 Project Background

Kusile is a coal fired power station, owned and operated by Eskom Holdings SOC (Ltd). Construction of the power station began in 2008, and the first power production started in 2017. Currently 4 units of the planned 6 units have been constructed and have been connected to the national grid. Coal is burned in large boilers, and the heat generated in the burning of the coal drives the electricity generation process. The residual material from the coal-burning is ash.

The power station will utilise dry ashing facilities for the disposal of its ash. Studies have indicated that the current ash/gypsum co-disposal facility, which received Environmental Authorisation with the power station in March 2008, is inadequate to accommodate the expected volume of ash to be generated during the life of the station. It’s carrying capacity would be limited to less than 10 years if both gypsum and ash are disposed on the facility.

An additional facility is therefore required to accommodate ash disposal for the 60-year design life of the station. Once the first phase of the ADF has been completed, the two waste streams of ash and gypsum will be permanently separated. Thus, the current co-disposal facility will be utilised exclusively for the storage of gypsum, for the 60-year design life of the station.

Eskom Holdings SOC Ltd. (Eskom) appointed EPCM to provide professional services for Detail Design of the 60 Year Ash Disposal Facility (ADF) at the Kusile Power Station: Under the current EPCM contract, and described in this report, is the detailed design for the construction of Phase 1.

The new ADF facility has been designed in compliance with the facility's Integrated Environmental Authorisation issued by the Department of Environmental Affairs (DEA) and the Water Use Licence requirements, to support an amended Water Use Licence Application (WULA) that would be issued by the Department of Water and Sanitation (DWS). The construction works and operations of the facility are to follow suit and also comply to the licence requirements.

- The scope of new infrastructure that will be required includes the following:
- Conveyor system for the transportation of ash from the power station to the new ADF.
- Barrier containment / liner system for the ADF.
- Stormwater, contaminated water, and clean water management systems.
- Site services.
- Office facilities for a contractor to operate and maintain the facility.
- Support systems for dust suppression, irrigation, electrical, control and instrumentation.
- Pump Houses and pipe reticulations.

The ADF is to be developed in twelve phases, with the extension of the ADF at approximately five-year intervals. Eskom's objective is to construct and commission the proposed ADF so that ash produced by the power station over its design life of 60 years can be disposed in a safe and responsible manner.

An authorisation for the Ash Disposal Facility (ADF) was issued in 2015. In order to commence with the development, Eskom Holdings SOC Limited applied for a Water Use Licence for the applicable water uses, in terms of the National Water Act, 36 of 1998, as amended.

The water balance model was done to determine if the newly proposed dirty water containment facilities were adequately sized to comprehend all dirty water inflows (dirty runoff from the ADF and conveyor corridor portion) without any of the facilities spilling more than once in 50 years. This is to ensure compliance with regulation GN704 from the national water act (NWA).

The Kusile PS will make use of “dry stacking” to dispose the ash produced at said PS. Dry stacking of tailings also known as filtered tailings are generally transported by conveyor or truck, where it is then deposited, spread, and compacted to form an unsaturated tailings deposit. Even though this method of disposal is referred to as “dry stacking”, this term is incorrect as the tailings is not completely dry but rather it is partially saturated (Davies & Rice, 2001).

2.2 Locality

The Kusile PS is located approximately 25km south-east of the town Bronkhorstspruit in the Mpumalanga Province and the newly proposed 60-year ADF is located approximately 2km south-west of the Kusile PS boundary and directly east of the R686 road. Kusile’s newly proposed ADF is located approximately 3km east of the Gauteng – Mpumalanga boundary, and falls within the Mpumalanga province, the locality of the newly proposed ADF is shown in **Figure 1**.

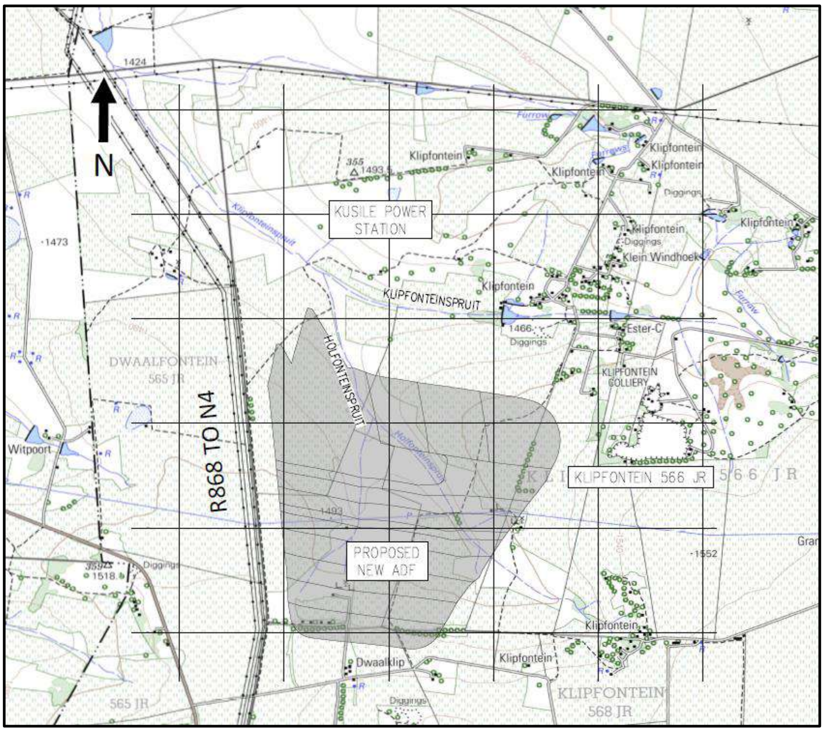


Figure 1: Locality Map

2.3 Quaternary Catchment and Drainage of Site

The Kusile ADF falls within the Olifants Water Management area. The disposal of ash will take place in the B20F quaternary catchment. Water naturally drains north-west in the direction of the Klipfonteinspruit on site, which is fed by the perennial river Holfonteinspruit; and it is expected that the Klipfonteinspruit river system will be impacted by the construction of the ADF.

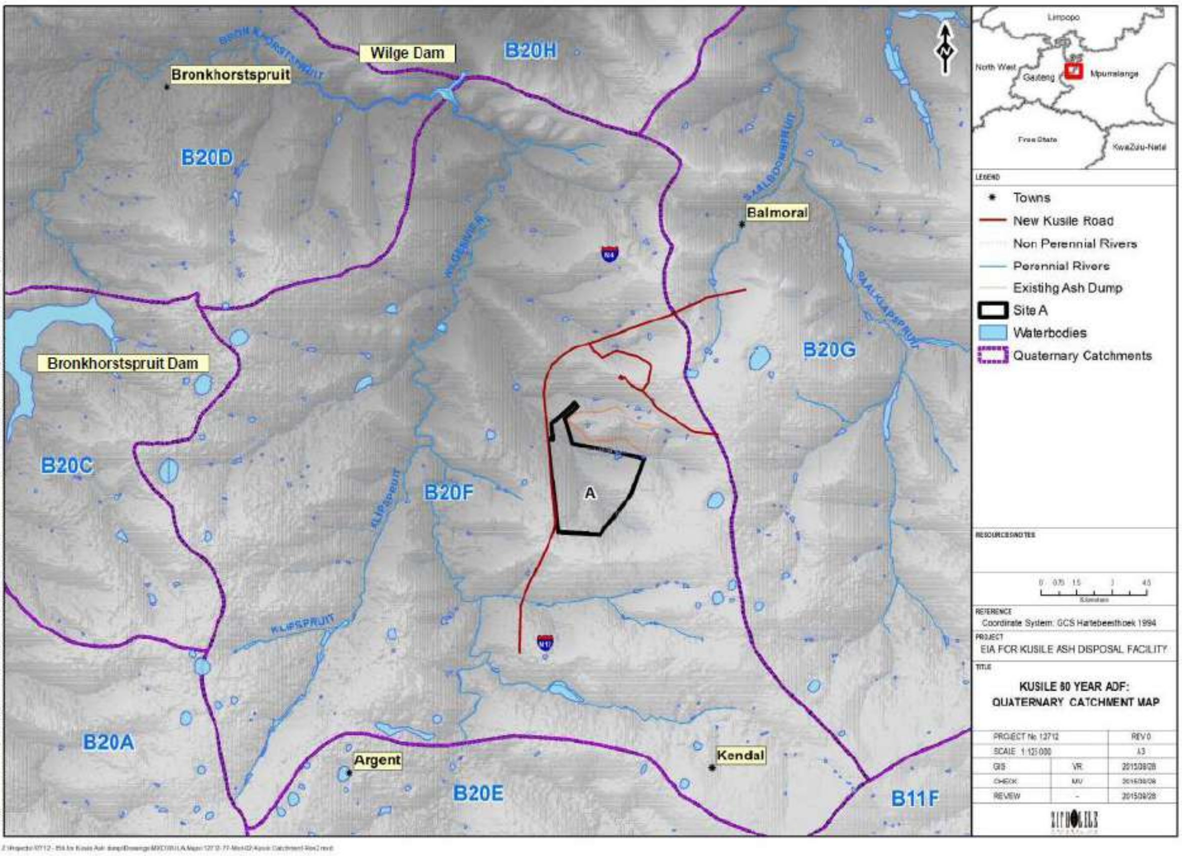


Figure 2: Quaternary Catchment B20F (Zitholele, 2014)

3. CLIMATIC CONDITIONS

3.1 Rainfall and Evaporation

Kusile ADF operations fall within the summer rainfall region of South Africa, being a warm temperature, with cold dry winters and hot summers. The summer rainfall is sporadic, with frequent thunderstorms, associated with high-intensity rainfall events.

The Wilgerivier (SAR) 0514618_W weather station is the closest station to the study area that has reliable (95%) historic daily rainfall data, with records ranging from 1903 – 2000. The Mean Annual Precipitation (MAP) is reported as 697 mm per annum. Daily rainfall depths were extracted using the daily rainfall data extraction utility developed by Richard Kunz, from the Institute for Commercial Forestry Research (ICFR), in conjunction with the School of Bioresources Engineering and Environmental Hydrology (BEEH) at the University of KwaZulu-Natal, Pietermaritzburg, South Africa. This utility will assist the user in extracting observed and infilled daily rainfall values from a database that was developed by Steven Lynch during a Water Research Commission (WRC) funded research project (K5/1156) awarded to BEEH. The project, titled “The development of a raster database of annual, monthly and daily rainfall for southern Africa”, was completed in March 2003.

Department of Water and Sanitation (DWS) station B2E001 is the closest station to the study area that has reliable historic evaporation data. Relatively high levels of evaporation occur in the area. The maximum evaporation rate occurs in December, with a mean rate of 5.65 mm per day. Evaporation is greater than rainfall for all months of the year resulting in a marked moisture deficit in the region. The average rainfall and average evaporation figures are shown in Table 1.

Table 1: Summary of Rainfall and S-Pan evaporation data

Month	Mean Precipitation (mm)	Mean Evaporation (mm)
January	132.7	166.1
February	84.4	143.5
March	95.3	135.4
April	34.5	105
May	14.6	85.3
June	9.4	67.4
July	2.7	74.6
August	6.9	102.4
September	12.4	139.5
October	76.0	163.1
November	110.3	160.3
December	116.0	174.9
Mean Annual Total	695.2	1523.8

4. WATER BALANCE

The formulation of the water balance model will be done to determine if the dirty water management systems are sized and operated adequately to contain the dirty water runoff from the ADF and conveyor corridor without any of the dirty water containment facilities spilling more than once in 50 years.

4.1 Model Overview

The model is a continuous stochastic model formulated in the GoldSim simulation software package. GoldSim water balance models utilize a Monte Carlo simulation engine to assess the current operational functionality of the dirty water containment facilities.

The water balance model will include / incorporate the following focus areas:

- Ash Disposal Facility (ADF);
- Rehabilitated Areas.
- Dust Suppression on the ADF;
- Irrigation on Rehabilitated Areas;
- Pollution Control Dams (PCDs); and
- Clean Water Dams (CWDs).

The dirty water runoff from the ADF is conveyed to the newly constructed Pollution Control Dams (PCDs) by means of lined concrete channels. Over the 43-year period, the dirty areas of the ADF will be rehabilitated and clean water runoff generated from these rehabilitated areas will be in turn, conveyed to the newly constructed Clean Water Dams (CWDs).

The dirty water collected within the PCDs will be used for dust suppression and clean water stored in the CWDs will be used for irrigation. Dust suppression is primarily implemented on the ADF footprint itself and irrigation will be sprayed over the rehabilitated areas.

The main components of the water balance model will include:

- Rainfall generator model – Stochastic rainfall generation based on historic local rainfall patterns.

- Catchment model – Runoff and recharge simulation of the various dirty water areas (ADF area, etc);
- Pond model – The storage and transfer of water from the containment facilities (PCDs and CWDs); and
- Allocator Function Element – Give an indication of the expected top up (make-up) water required for dust suppression and irrigation purposes.
-

4.2 Process Flow Diagram

The PFDs for Phase 1, Phase 2, Phase 3, and Phase 4 – 11 are found in **APPENDIX A**.

4.2.1 Phase 1

Phase 1 will take place over the first seven years of operations and six water containment facilities will need to be constructed before operations commence. Four of the containment facilities will be PCDs and the remaining two will be the CWDs. Dirty runoff from the ADF will be conveyed via concrete channels to PCD 1 A, PCD 1 B, PCD 2 and Road PCD. PCD 1 A, PCD 1 B, and PCD 2 will supply water for dust suppression purposes. Additional make-up water will need to be provided to these PCDs to supply additional water for the dust suppression demand during the dry spells. The two CWDs will receive water through rainfall directly onto the footprint of each of the dams respectively. This water build up will either accumulate or evaporate over time. Additional make-up is provided during dry spells.

4.2.2 Phase 2

Progressing to phase 2, two more dirty water containment facilities will need to be constructed PCD 3 Complex (PCD 3 and PCD 4). Dirty runoff from the ADF will be conveyed to PCD 1 A, PCD 1 B, PCD 2, Road PCD and PCD 3 Complex (PCD 3 and PCD 4 that is newly constructed) via concrete channels. PCD 1 and PCD 2 will supply water for dust suppression purposes. Additional make-up water will need to be provided from the HRD or clean raw water make-up lines connected to the PCDs to supply additional water for the dust suppression demand during the dry spells. PCD 3 Complex will transfer top up water to PCD 2 and any excess water from PCD 3 Complex will overflow into PCD 2. The CWDs will receive water through rainfall and make-up water, and this water will be used for irrigation purposes. The expected rehabilitation of the ADF will commence in this phase and therefore the requirement for water for irrigation arises.

4.2.3 Phase 3

Phase 3 will take place from year 13 – 17 of the simulation. In this phase, PCD 1 B will be transitioned into a clean water dam (CWD 1 B). Dirty runoff from the ADF will be conveyed to PCD 1 A, PCD 2, Road PCD and PCD 3 Complex (PCD 3 and PCD 4) via concrete lined channels. PCD 1 and PCD 2 will supply water for dust suppression purposes and will require make-up water (from the HRD or clean raw water make-up lines connected to the PCDs) during the dry rainfall spells. PCD 3 Complex will supply top up water to PCD 2 (for dust suppression) and any excess water from PCD 3 Complex will overflow into PCD 2. The three CWDs (CWD 1, CWD 2 and CWD 1 B) will receive water through rainfall and runoff from the rehabilitated areas on the ADF, as well as additional make-up water (from the clean raw water make-up lines connected to the CWDs) required for irrigation. This water will be used for irrigation purposes on the irrigated areas on the ADF.

4.2.4 Phase 4

Phase 4 – 11 will take place over the remaining 60 Years (from year 18 – end of operations) and before this phase commences, the dirty water containment system PCD 5 Complex (PCD 5 and PCD 6) should be constructed. Once this new complex is constructed there will be ten (10) containment facilities, seven of them will be PCDs and the remaining three will be the CWDs. Dirty runoff from the ADF will be conveyed to PCD 1 A, PCD 2, Road PCD, PCD 3 Complex and PCD 5 Complex. PCD 1 and PCD 2 will supply water for dust suppression purposes and will require make up water (from the HRD or clean raw water make-up lines connected to the PCDs). PCD 5 Complex will overflow into PCD 3 Complex; and any excess water from PCD 3 Complex will overflow into PCD 2. PCD 3 and PCD 5

Complex will transfer top up water to PCD 2 for dust suppression, respectively. The CWDs will receive inflows via rainfall, runoff and make-up water during the dry rainfall spells, and this water will be used for irrigation purposes. PCD 1 & 2 will transfer water to PCD 5 & 6 complex for extra buffer in flood conditions.

4.2.5 Phase 5

The available historical rainfall data, for any area of interest, gives a representation of the past rainfall patterns that took place, but limited information regarding the risk of droughts or flood like activities can be obtained from this data. To ensure that the current dirty water management system is evaluated against a whole range of different rainfall sequences and recurrence intervals, a stochastic rainfall generation model will be implemented in the GoldSim water balance model.

The stochastic rainfall generator is based on Boughton's model derived from report 366-511877 (Boughton, 1999). This rainfall generator generates daily rainfall that is statistically representative of the daily, monthly, and yearly historic rainfall. The concept adopts a simplified version of Boughton (1999) methods. Instead of the multi-state Markov chain, this method only has two states, wet or dry. The rainfall generator has been integrated into the original model so stochastic values are generated simultaneously with the runoff probabilities. Random daily precipitation is, therefore, simulated using a first-order, 2-state Markov chain. Markov chains change randomly over time between states and probabilities for a new state depends on the previous state. This generator assumes a gamma probability distribution based on the data provided.

The selection of station 0514618_W (Wilgerivier) is made since this is the closest station to the study area with an acceptable reliability record (95% good quality data) and a minor amount of patched data (patching of rainfall data requires filling in the missing data with data from nearby stations based on statistical rainfall parameters).

4.3 Surface Water Runoff

Expected runoff from the ADF was estimated using the Rational Formula. The Rational Method is consistent with the catchment analyses that were used in sizing the runoff conveyance systems as detailed in report 366-511877. The Rational Formula (Equation 1) is appropriate for urban and rural areas where areas do not exceed 2,500 Ha (25,000,000 m²).

$$Q = F_t * C * I * A/360 \qquad \text{Equation 1}$$

Where:

Q = Maximum /peak rate of runoff (m³/s)
F_t = Adjustment factor for the recurrence interval storm considered. F_t=1 for this analysis.
C = Runoff co-efficient per applicable tables
I = Rainfall Intensity (mm/hr.)
A = Area of catchment in hectares

The runoff co-efficient is a factor ranging from 0 to 1 which compensates for variations in rainfall over the catchment, infiltration, and overland flow velocity during a storm. C values used in this model were determined from the Drainage Manual, Sixth Edition (2013), published by SANRAL (South African National Roads Agency Limited).

The excess of water that is not captured as runoff is routed to infiltration. This may later be captured by the subsurface leachate collection system under active ADF disposal sites or alternatively be stored within the ash and lost to evaporation.

4.4 Surface Water Management

The dirty water runoff from the Ash Disposal Facility (ADF) is conveyed to the PCDs by means of concrete-lined channels. Runoff areas were based on an infrastructure layout design proposed by EPCM (**Error! Reference source not found.**). The only areas that will generate dirty runoff will be from the newly proposed ADF footprint, which is approximately 142 Ha (1,429,059 m²) during phase one, and the dirty runoff from the conveyor corridor area (233,990 m²). The conveyor area runoff is constantly conveyed to PCD 1 A throughout the years of operations. Commencing to Phase 3 of Kusile operations introduces clean water runoff into the system. This clean water runoff will be generated from the rehabilitated ADF footprint areas.

Figure 3 - Kusile ADF Operations Infrastructure Layout

Table 2 below lists the ADF footprint areas generating dirty runoff for each phase, as well as the footprint of the rehabilitated areas for each phase. Even though rehabilitation on the ADF only commences in phase 2, the model was simulated as follows:

During phase 2, water coming from the newly rehabilitated areas will not be deemed as clean water and will therefore be modelled as dirty runoff.

Table 2: Runoff Areas

Phase	Dirty Runoff Areas (m²)	Rehabilitated Runoff Areas (m²)
Phase 1	1 429 059	N/A
Phase 2	2 141 349	N/A
Phase 3	2 096 348	396 181
Phase 4	3 041 530	591 146
Phase 5	3 321 900	1 010 860
Phase 6	2 719 295	1 502 308
Phase 7	2 427 156	1 550 275
Phase 8	2 155 894	2 000 266
Phase 9	1 671 145	2 458 119
Phase 10	1 371 145	2 410 691
Phase 11	1 039 521	3 086 795

4.5 Containment Facilities

The operations at Kusile's ADF will require containment facilities to store and utilise both clean and dirty water. As previously mentioned, dirty water runoff from the Ash Disposal Facility (ADF) is conveyed to the dirty water containment facilities (PCDs). Clean water runoff will be generated from the rehabilitated ADF areas, and this runoff is contained in the CWDs. Throughout the 60 Years of operations, there will be a total of seven PCDs and three CWDs. During Phase 1 and 2, PCD 1 B will be utilized as a PCD after which it transforms to a CWD in phase 3. This utilization of capacity dirty water capacity results in a loss of approximately 42,875 m³ that could have potentially stored / utilised dirty water.

The dirty water containment facilities should be sized to ensure it does not reach the full supply level more than once in 50 years. Due to the current water use license approvals and following instruction from the client, the containment volumes from the previous engineering design were analysed in the water balance model to determine if the proposed capacities are adequate. These volumes for the PCDs and CWDs are listed in Table 3.

Table 3: Containment Facilities Characteristics

Description of Dam	Capacity (m^3)	TWL Area (m²)
PCD 1 A	102 875	32 066
PCD 2	113 471	30 039
Road PCD	24 000	11 065
PCD 3 Complex (PCD 3 & PCD 4)	97 400	34 486
PCD 5 Complex (PCD 5 & PCD 6)	97 400	34 443

PCD 1 B / CWD 1 B	42 875	18 307
CWD 1	18 555	10 641
CWD 2	48 917	17 193

4.6 Dust Suppression Demands

Kusile operations requires dust suppression for the ADF, and these dust suppression areas will change over time depending on the different phases and the progression of the ADF. The dust suppression areas have been reduced as it would not be feasible to dust suppress the entire footprint of the ADF. Certain areas were determined that would temporarily be covered with topsoil (phase 2 onwards) and these covered areas would reduce the area where dirty water would be sprayed for dust suppression purposes. The reduced dust suppression areas would need to be watered daily to prevent excessive dust generation. The dust suppression rate of 26.8 mm/m² will be applied during a 5-day cycle (20% of area per day), for 10-hours a day during summer months (Sep – Apr). This is equivalent to an application rate of 5.4 mm/m²/day over the entire reduced dust suppression area. A dust suppression rate of 13.6 mm/m², during a 5-day cycle of 10-hours per day was allocated for the winter months (Apr – Aug). This is equivalent to an application rate of 2.7 mm/m²/day over the entire reduced dust suppression area. The dust suppression rates were calculated using the equation obtained from (Blight & Kreuiter, 2000). The equation is as follows:

$$(S - E_A)\frac{h}{24} = E_{ss} - R + (E_s - \check{R})(A/A_s - 1) \qquad \text{Equation 2}$$

Where:

S = Application of wastewater by spraying.

EA = Evaporation rate of water in the air above the waste surface.
h = Hours of spraying in a day.
Ess = Evaporation rate from the saturated surface that is being sprayed.
R = Rainfall.
Ř = Average daily rainfall for days where no spraying occurred.
Es = Evaporation rate from the waste surface that is not being sprayed on a particular day.
A/As = Dust suppression cycle days (percentage of area that is sprayed on a daily basis inrelation to the overall area, i.e. 5 days = 20%).

The inputs to calculate dust suppression rates during the summer months is listed in Table 4 and for the winter months in Table 5.

Table 4: Summer Dust Suppression Spray Depths

Criteria	Amount	Unit
Dust suppression cycle	5	Days
A/As	20	%
Es	4.37	mm
Daily spray duration.	10	hours/day
EA	8	mm
Ess	6	mm
Rainfall	0	mm
Rate of Application.	64.35	mm/24h day
Spray Depth (S)	26.8	mm/spray duration

This indicates that during the summer months, if 20% of the ADF is sprayed every day (for 10 hours) on a 5-day rotation with no rain, 64.35 x 10/24 = 26.8 mm of water could be applied each day without increasing the water content stored within the ADF.

Table 5: Winter Dust Suppression Spray Depths

Criteria	Amount	Unit
Dust suppression cycle	5	Days
A/As	20	%
Es	2.23	mm
Daily spray duration	10	hours/day
EA	4	mm
Ess	3	mm
Rainfall	0	mm
Rate of Application	32.6	mm/24h day
Spray Depth (S)	13.6	mm/spray duration

As per the explanation above, during the winter months if 20% of the ADF is sprayed every day (for 10 hours) on a 5-day rotation with no rain, 32.6 x 10/24 = 13.6 mm of water could be applied each day without increasing the water content stored within the ADF.

The dust suppression rates over the various phases, and the containment facility that will be supplying the water is further elaborated in Table 6. The maximum dust suppression rate is supplied from PCD 1 (PCD 1 A and PCD 1 B) during phase 1. This rate is approximately 6,897 m³/d and does not exceed the maximum pump transfer rate that can be supplied from the pump station. The Low-Pressure Services (LPS) report indicated that there are two pump houses and at both pump houses there are three (3) pumps on duty and two (2) pumps on standby, with each pump transferring 270.7 m³/hr for dust suppression purposes. Assuming that each pump house is operational continuously for 10 hours during a day, a maximum of 8,121 m³/day (270 m³/hr x 3 pumps x 10 hours) of water can be pumped from a single pump house to the dust suppression sprinklers. The water balance was restricted to this maximum transfer of 8,121 m³/day as the flows from the LPS report are currently the basis of design for the pump systems.

The dust suppression demand will only be supplied on days where less than ten (10) mm of rainfall is simulated and when the PCDs volume is at 5% capacity or higher. This 5% control was incorporated into the model to accommodate for any future silt build up in the PCDs on site. It is therefore assumed that the PCDs would lose approximately 5% of operational capacity due to the increase in silt build up.

Table 6: Dust Suppression Rates

Phase	Dust Suppression Area (m²)	PCD 1 (m³/day)	PCD 2 (m³/day)
Phase 1	1 429 059	6 897*	766
Phase 2	1 357 296	4 731*	2 548
Phase 3	1 286 965	3 451	3 451
Phase 4	1 943 884	5 212	5 212
Phase 5	2 130 478	4 570	6 855
Phase 6	1 719 898	2 767	6 456
Phase 7	1 206 649	1 941	4 530
Phase 8	1 161 868	2 181	4 050
Phase 9	826 095	1 329	3 101

Phase 10	813 060	1 526	2 834
Phase 11	577 736	930	2 169

*PCD 1 A + PCD 1 B

4.7 Irrigation Demands

As mentioned previously in the report, portions of the ADF will be rehabilitated using a phased approach as the ADF footprint progresses over time.

Clean water will be sprayed on the rehabilitation areas of the ADF for irrigation purposes. CWD 1, CWD 1 B and CWD 2 will supply the clean water and the rate at which this irrigation demand would need to be supplied (dependent on each phase) is listed in Table 7.

A constant spray depth of 21 mm will be applied on a 7-day cycle to meet rehabilitation irrigation demands. This effectively means that approximately 3 mm/m^2/day of water will be sprayed on the entire irrigation area. The LPS report indicated that there are two pump houses and at both pump houses there are two (2) pumps, one (1) duty and one (1) standby. Each of the pumps can transfer 210 m^3/hr for irrigation purposes. Assuming that each pump house is operational continuously for 10 hours during a day, a maximum rate of 4,200 m^3/day (210 m^3/hr x 2 pumps x 10 hours) of water can be pumped from a single pump house to the irrigation sprinklers. The worst-case scenario for irrigation supply is simulated to be during phase 7, where a maximum of 2,243 m^3/day of clean water is required for irrigation purposes.

Table 7: Irrigation Rates

Phase	Irrigation Area (m^2)	CWD 1 (m^3/day)	CWD 2 (m^3/day)	CWD 1 B (m^3/day)
Phase 1	0	N/A	N/A	N/A
Phase 2	396 181	178.3	1 010.0	N/A
Phase 3	194 965	87.7	234.0	263.2
Phase 4	419 715	188.9	503.7	566.6

Phase 5	491 448	221.2	589.7	663.5
Phase 6	452 982	203.8	543.6	611.5
Phase 7	747 775	336.5	897.3	1 009.0
Phase 8	710 386	319.7	852.5	959.0
Phase 9	674 867	303.7	809.8	911.1
Phase 10	404 920	182.2	485.9	546.6
Phase 11	323 936	145.8	388.7	437.3

4.8 Make Up Requirements

There are instances where the dams (PCDs and CWDs) are empty and the volume in the dams are drawn down to the Dead Storage Volumes (DSVs) of the dams. At these instances there is an insufficient amount of water in the dams to meet dust suppression and / or irrigation demands and subsequently, make up water would need to be imported to ensure successful operations. Provision is made for make-up water pipe connections for HRD or clean raw water to PCD's and clean raw water to CWD's.

4.8.1 Dust Suppression Make Up Requirements

LPS report states that the make-up water system will supply make-up water from the Holding Recycle Dam (HRD) within the power station to PCD 1 by means of a gravity feed line or from the clean raw water make-up lines connected to the PCD’s. The make-up water line from the HRD is shared with the make-up line for the 10-year ash dump dam. Only one dam will be able to receive makeup water at any given time. The average make-up requirements, as well as a worst-case scenario (no water in the dams) for each phase are listed in Table 8 below. According to the design team, phase 5 is where the ADF will reach its maximum footprint, thus requiring the most dust suppression during this phase. Considering the worst-case scenario during phase 5; a dry period where none of the dams have water, the make-up requirements will be approximately 11,425 m³/day.

Table 8: Dust Suppression Make Up Water

Phase	Average Make Up (m^3/day)	Worst Case Scenario (m^3/day)	Percentage of Make Up (%)
Phase 1	5011	7663	65%
Phase 2	4507	7279	62%
Phase 3	4057	6902	60%
Phase 4	6249	10424	59%
Phase 5	6786	11425	60%
Phase 6	5399	9223	59%
Phase 7	3461	6471	59%
Phase 8	3370	6231	53%
Phase 9	2305	4430	52%
Phase 10	2361	4360	54%
Phase 11	1845	3099	60%

4.8.2 Irrigation Make Up Requirements

Irrigation of rehabilitated areas of the ADF only takes place during Phase 2 of operations and the only inflows into the CWDs prior to this phase, is that of rainfall on the footprint of the dam. The irrigation requirements (outflows) surpass the inflows into the dam and there are numerous instances where make up water (clean water) will be required to meet the irrigation demands. This make-up will be provided by the clean raw water make-up lines connected to the CWDs. The make-up requirements for each phase for the CWDs on site is listed in Table 8 below. Considering the worst-case scenario during phase 7; a dry period where none of the dams have water, the make-up requirements will be approximately 2,093 m^3/day. During the first seven phases, between 90 – 100 % of the water for irrigation purposes will need to be imported from the raw water make-up supply lines connected to the CWDs. All three CWDs supply a maximum of 10% of irrigation water during the first eight (8) phases. As the rehabilitated areas increases (generating more runoff) in the latter phases (Phases 9 – 11), less make up water is required as the CWDs have adequate volumes of water stored that will be used for irrigation purposes.

Table 8: Irrigation Make Up Water

Phase	Average Make Up (m^3/day)	Worst Case Scenario (m^3/day)	Percentage of Make Up (%)
Phase 1	N/A	N/A	N/A
Phase 2	1 107	1 109	100%
Phase 3	524	546	96%
Phase 4	1 137	1 175	97%
Phase 5	1 304	1 376	95%
Phase 6	1 144	1 269	90%
Phase 7	1 969	2 093	94%
Phase 8	1 810	1 987	91%
Phase 9	1 678	1 889	89%
Phase 10	915	1 133	81%
Phase 11	633	907	70%

4.9 Dirty Water Transfers and Control Philosophy

Operations at Kusile are presumed to take place over the next 60 Years. During this period the PCDs on site are only allowed one environmental discharge. The approach with regards to the water balance was to ensure that volumetric capacities of the PCDs are in line with the existing WUL. According to the Zitholele report the dust suppression areas needed to be significantly reduced as it was not possible to dust suppress the entire ADF footprint. Furthermore, it was advised to relocate the water quality monitoring point to the energy dissipation structures at the toe of the ADF. This will reduce the risk of contaminating either of the CWDs, as the runoff from the newly rehabbed areas are not considered to be “clean” in terms of the water quality requirement guidelines. This results in the runoff from a newly rehabilitated area (considered as dirty water) to be conveyed to the PCDs. The increased runoff and reduced dust suppression abstraction, means that at both PCDs, the inflows are more than outflows. This leads to an increase in storage and spillage from PCD 1 A into the environment over the 43-year simulation period.

To prevent PCD 1 A from spilling into the environment, it was necessary to transfer water to either PCD 3 Complex or the PCD 5 Complex (this transfer is dependent on each of the phases). The Low-Pressure Services (LPS) report indicated that there are two pump houses and at both pump houses there is only one (1) pump on duty and one (1) on standby, and this pump needs to transfer 336 m³/hr of water from PCD 1 A to PCD 3 / 5 Complex. Assuming that each pump house is operational continuously for 24 hours during a day, a maximum of 8,064 m³/day (336 m³/hr x 1 pumps x 24 hours) of water can be pumped from a single pump house to the PCD 3 / 5 Complex (dependent on phase). As previously mentioned, the water balance model was restricted to this maximum transfer rate of 8,064 m³/day as the designs from Low-Pressure Services (LPS) report are already submitted for tender phase and cannot be modified. In addition, provision is made to transfer water to the ADDD using the same dirty water pumps and flow rates.

The transfer rates and operational philosophy for Kusile’s 63-year ADF operations can be found in Table 9 below.

Table 9: Dirty Water Transfers

Source of Transfer	Average Rate (m³/day)	Max Rate (m³/day)	Transfer To	Transfer Philosophy
Road PCD	149	149	PCD 2	Road PCD Transfers excess water to PCD 2 when the volume in Road PCD is more than 10% of its volumetric capacity.
PHASE 2 and 3: Dirty Water Transfers				
PCD 1 and PCD 2	363	4 320	PCD 3 Complex	PCD 1 & 2 Transfers excess water to PCD 3 Complex, when the volume in PCD's is more than 70% of its volumetric capacity. (Transfer can also be done to the ADDD)
PCD 3 Complex				
PCD 3 Complex	1 063	2 063 Phase 3	PCD 2	Gravity drains from PCD 3 complex to PCD 2, when the volume in PCD 3 Complex is more than 5% of its volumetric capacity and when the volume in PCD 2, is below 5% of its volumetric capacity.
PHASE 4 - 11: Dirty Water Transfers				
PCD 5				

PCD 5 & 3 Complex	484	13 920	PCD 2	PCD 3 & 5 Complex Transfer excess water to PCD 2 by gravity, when the volumes in PCD 3 and 5 Complex are more than 80% and 70% of their volumetric capacities respectively.
PCD 3 Complex	831	6 285 Phase 5	PCD 2	Gravity drains from PCD 3 complex to PCD 2, when the volume in PCD 3 Complex is more than 5% of its volumetric capacity and when the volume in PCD 2, is below 5% of its volumetric capacity.
PCD 5 Complex	724	3 428 Phase 5	PCD 2	Gravity drains from PCD 5 complex to PCD 2, when the volume in PCD 5 Complex is more than 5% of its volumetric capacity and when the volume in PCD 2, is below 5% of its volumetric capacity.

4.9.1 Phase 1 Dirty Water Transfer

During phase 1 of operations, dirty water transfer will be from the Road PCD to PCD 2. Dirty water transfer from PCD 2, PCD 1A and PCD 1B can also be transferred to the ADDD via the dirty water transfer pumps located in pump houses PS1 and PS2. When the volume in the Road PCD exceeds 10%, the Road PCD is to be pumped to PCD2 at a minimum rate of 149 m^3/day by means of slurry pumps within the Road PCD extraction sump area. Other transfers will be from the pump houses to the dust suppression sprinklers.

4.9.2 Phase 2 and 3 Dirty Water Transfer

During phase 2 and 3 of operations, the footprint of the dirty areas (ADF) increases, and the dust suppression area decreases. This results in more dirty water entering the dams than what is abstracted from the dams. PCD 3 Complex will be constructed during phase 2 to provide additional storage on site to store the dirty water and supply PCD 2 with top up water for dust suppression purposes. PCD 1 & 2 will transfer excess water to the PCD 3 Complex to keep PCD 1 & 2 at a constant operating level of 70% or less. Once the dirty water volumes in PCD 1 & 2 surpasses the upper operating levels, this water will be transferred to PCD 3 Complex at a rate of 4,320 m³/day. PCD 3 Complex will transfer excess water to the PCD 2 to keep PCD 3 and 4 at a constant operating level of 80% or less. These transfers will be by gravity and at a maximum rate of 6,960 m³/day.

During a shortfall / dry rainfall month, PCD 3 complex will transfer top up water to PCD 2 when the water levels in PCD 2 is too low to supply water for dust suppression purposes. This water transfer rate will directly be linked to the dust suppression requirements of each specific phase. A maximum transfer of 3,040 m³/day can be transferred to PCD 2.

Further water transfer can be done to the ADDD. Additional capacity can be realised through the emergency usage of CWDs 1 & 2.

4.9.3 Phase 4 - 11 Dirty Water Transfer

Phase 4 - 11 of operations is where the worst-case scenario occurs. The ADF footprint reaches its maximum size during Phase 5, and this causes a major increase in the dust suppression areas, directly related to the dust suppression water requirement. This increase in water requirement for dust suppression then leads to a higher supply of top up water required from additional sources, due to the volumes of available water on site not being enough to supply this demand. The PCD 5 Complex will be constructed during phase 4 to provide additional buffer storage on site to store the dirty water and supply PCD 2 with top up water for dust suppression purposes. PCD 1 & 2 will transfer excess water to the PCD 5 Complex to keep PCD 1 & 2 at a constant operating level of 70% or less. Once the dirty water volumes in PCD 1 & 2 surpasses the upper operating levels, this water will be transferred to PCD 5 Complex at a rate of 4,320 m³/day. To avoid the overflow of PCD 3 and

5 Complex, the four PCDs will transfer excess water to PCD 2 to keep a constant operating level of 80% and 70% or less respectively. These transfers will be by gravity and at a maximum rate of 13,920 m³/day.

During a shortfall, the PCD 3 and the PCD 5 complex will transfer top up water to PCD 2 when the water levels in PCD 2 is too low to supply water for dust suppression purposes. This rate of transfer is directly linked to the dust suppression requirements. Looking at the worst-case scenario (phase 5), a maximum transfer rate of 6,855 m³/day should be transferred to PCD 2 (considering PCD 2 has no water) from PCD 3 or PCD 5 Complex.

5. WATER BALANCE RESULTS – PHASE 1

The dynamic water balance model for Kusile was simulated over a 43-year period using the GoldSim software to replicate likely operations on site. The water balance model is complex with many components constantly changing over time such as: the phasing of newly constructed PCDs over time, the change of PCD 1 B to a CWD, the progression of the ADF, changes to the ADF footprint (increase / decreased dirty runoff) and changes of the dust suppression and irrigation areas. Due to the phasing over time the results of the water balance will be presented for Phase 1, Phase 2, Phase 3, and Phase 4 – 11 respectively.

Phase 1 for Kusile operations will take place over seven (7) years and six water containment facilities will be constructed, four of them being PCDs and the remaining two will be the CWDs. Dirty runoff from the ADF will be conveyed to PCD 1 A, PCD 1 B, PCD 2 and Road PCD respectively and all three PCDs will supply water for dust suppression purposes. The CWDs will only receive water through direct rainfall onto the dam footprints and this water will either accumulate or evaporate over time.

5.1 Phase 1 Water Balance Results

5.1.1 PCD 1 A

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 1.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF and conveyor; and
- Make up water for Dust Suppression.

Outflows from the PCD include:

- Evaporation; and
- Water supply to the dust suppression demand.

The PCD volume probability result following the 250 realizations (first 7 years of simulation) in the GoldSim software can be seen in Figure 4. The PCD on average utilises approximately 5% of its capacity, which is due to assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time.

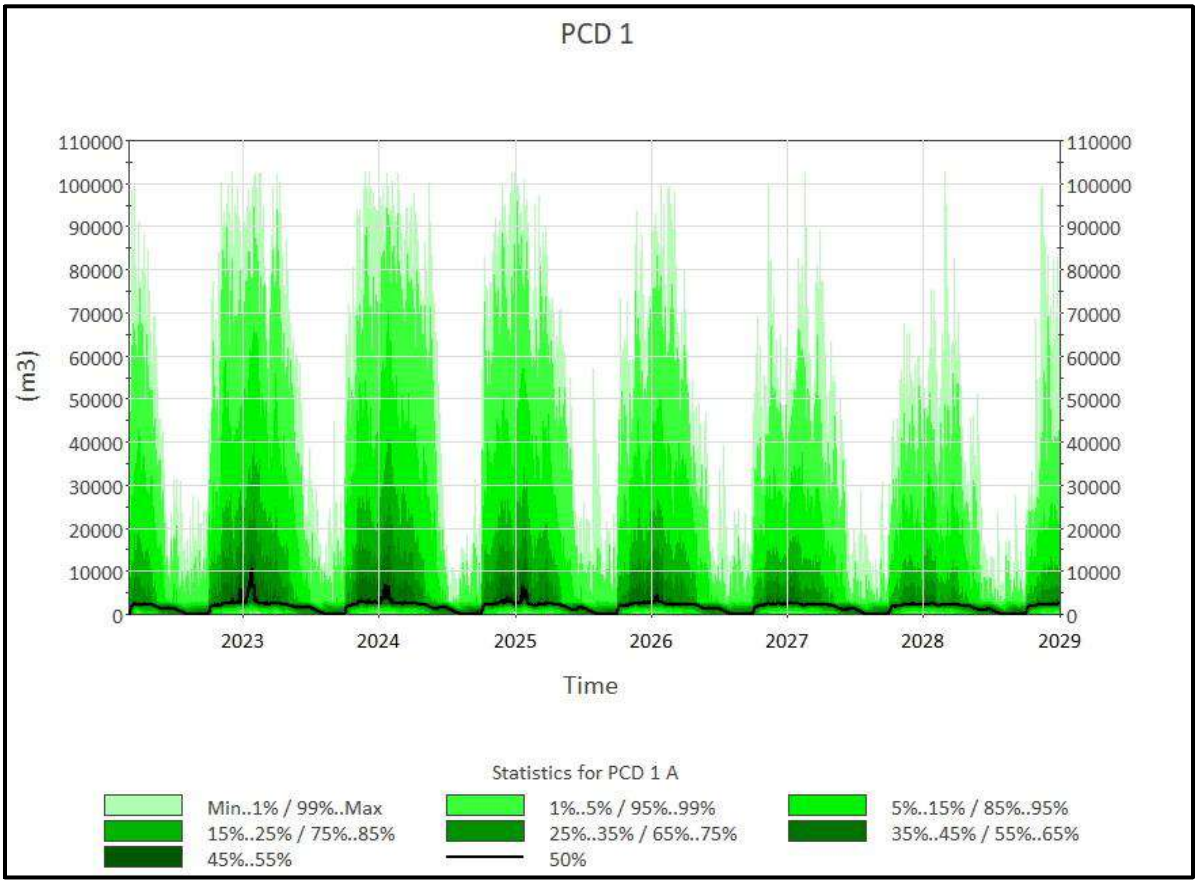


Figure 4: PCD 1 A Volume Probabilities (Phase 1)

5.1.2 PCD 1 B

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 1.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint; and
- Dirty water runoff from ADF.

Outflows from the PCD include:

- Evaporation; and
- Water supply to dust suppression.

The PCD volume probability result following the 250 realizations (first 7 years of simulation) in the GoldSim software can be seen in Figure 5. The PCD on average utilises approximately 6% of its capacity, which is due to assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time.

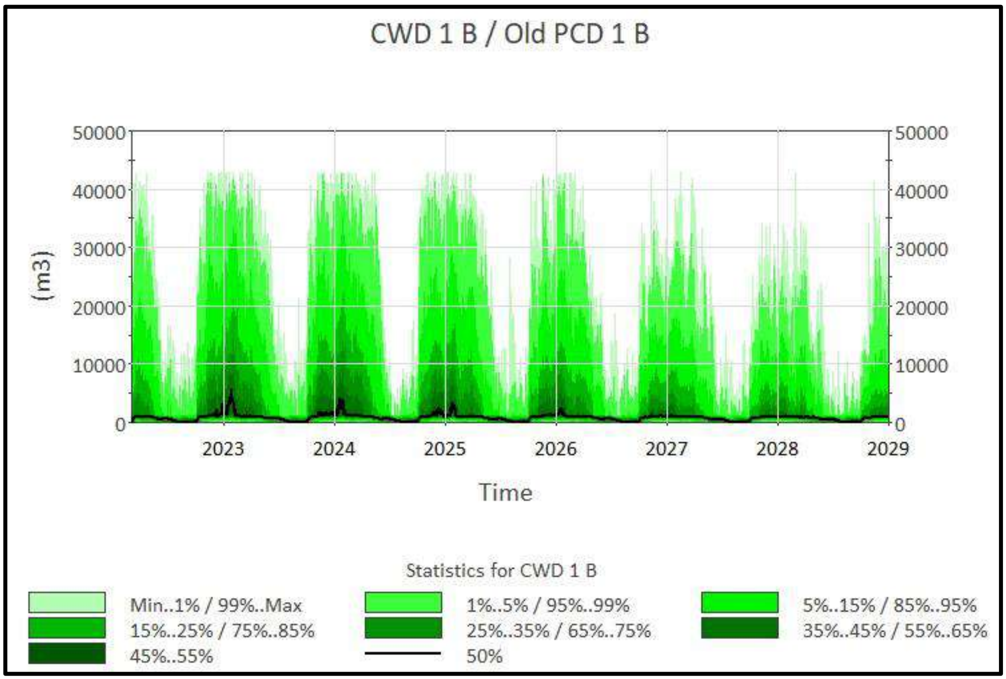


Figure 5: PCD 1 B Volume Probabilities (Phase 1)

5.1.3 PCD 2

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 1.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Excess water transferred from Road PCD;
- Dirty water runoff from ADF; and
- Make up water for Dust Suppression.

Outflows from the PCD include:

- Evaporation; and
- Water supply to dust suppression.

The PCD volume probability result following the 250 realizations (first 7 years of simulation) in the GoldSim software can be seen in Figure 6. The PCD on average utilises approximately 28% of its capacity, which is due to the excess water received from Road PCD and the assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time.

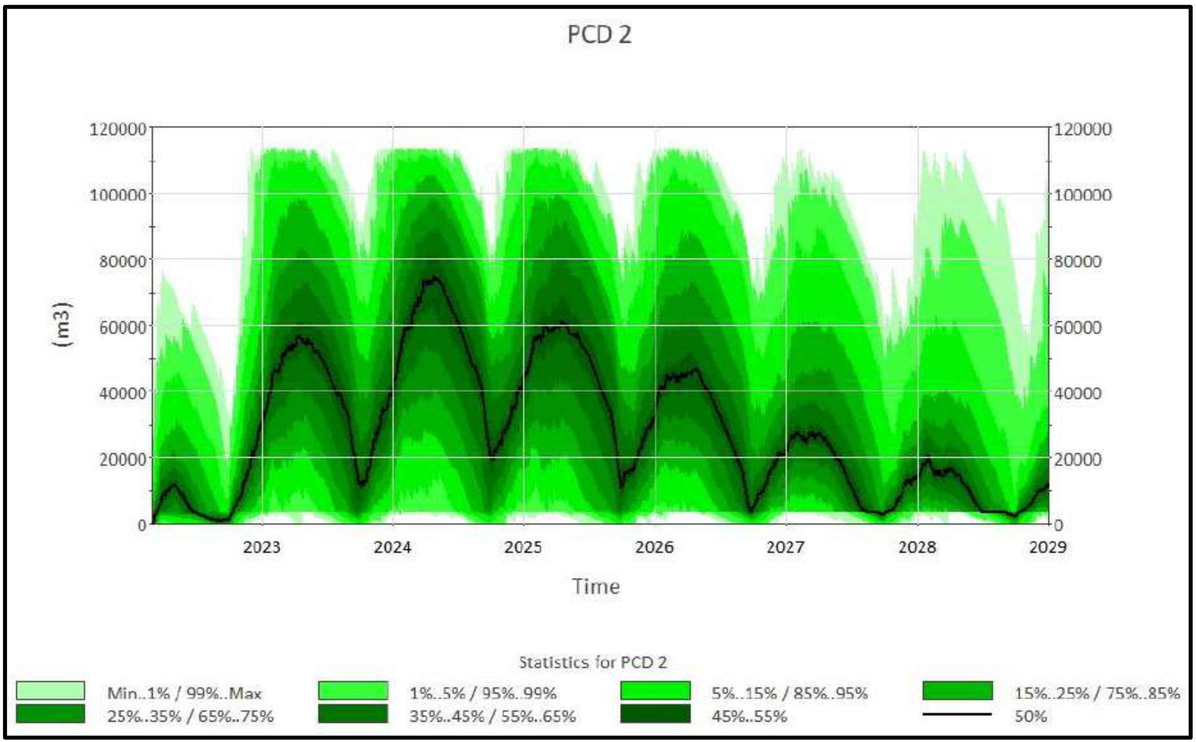


Figure 6: PCD 2 Volume Probabilities (Phase 1)

5.1.4 Road PCD

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 1.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint; and
- Dirty water runoff from ADF.

Outflows from the PCD include:

- Evaporation; and
- Excess water transferred to PCD 2.

The PCD volume probability result following the 250 realizations (first 7 years of simulation) in the GoldSim software can be seen in Figure 7. The PCD on average utilises approximately 51% of its capacity. To prevent the PCD overflowing in winter months, it transfers excess water to PCD 2.

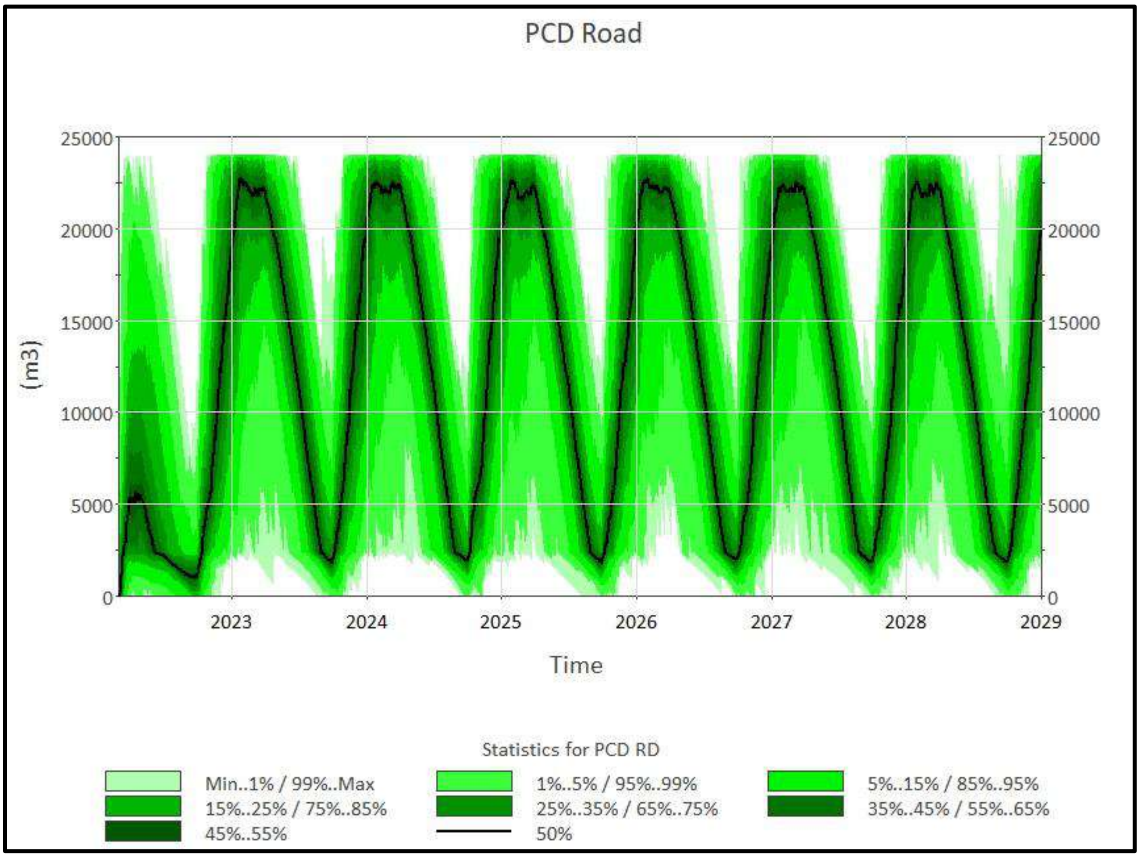


Figure 7: Road PCD Volume Probabilities (Phase 1)

5.1.5 CWDs

The CWDs were modelled in the GoldSim software to replicate likely operations on site for Phase 1.

Inflows into the CWDs (CWD 1 and 2) include:

- Direct rainfall onto the PCD footprint.

Outflows from the CWD include:

- Evaporation; and
- Increase in storage.

The CWD volume probability result following the 250 realizations (first 7 years) in the GoldSim software for CWD 1 can be seen in Figure 8 and CWD 2 in Figure 9. Neither of the CWDs will experience any "release to the environment" during phase 1, as there is not enough water accumulating in the dams (inflows is only that of rainfall). During Phase 1, CWD 2 is simulated to accumulate a maximum of 10,000 m³ of clean water, this is less than 30% of the CWD's volumetric capacity.

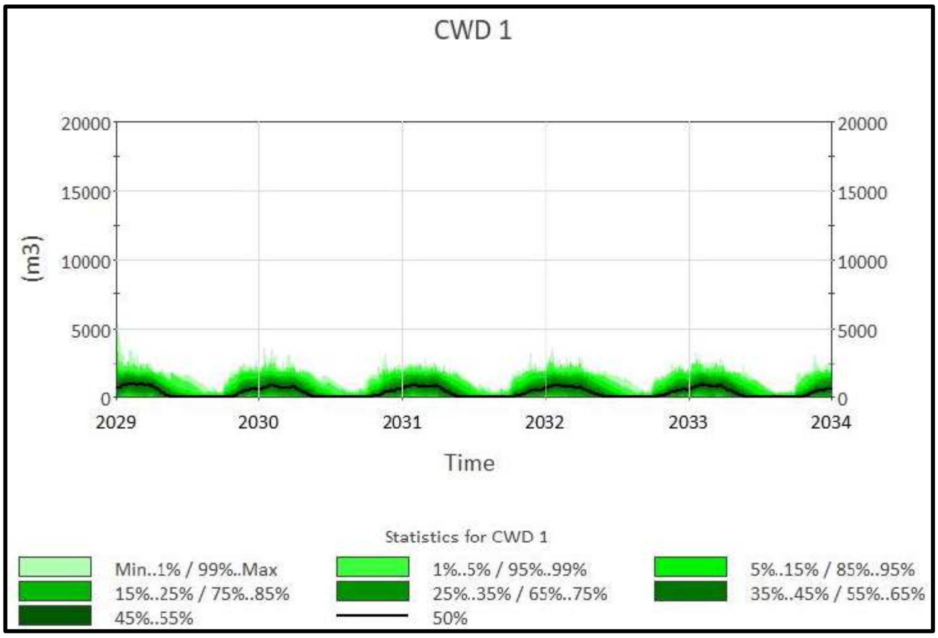


Figure 8: CWD 1 Volume Probabilities (Phase 1)

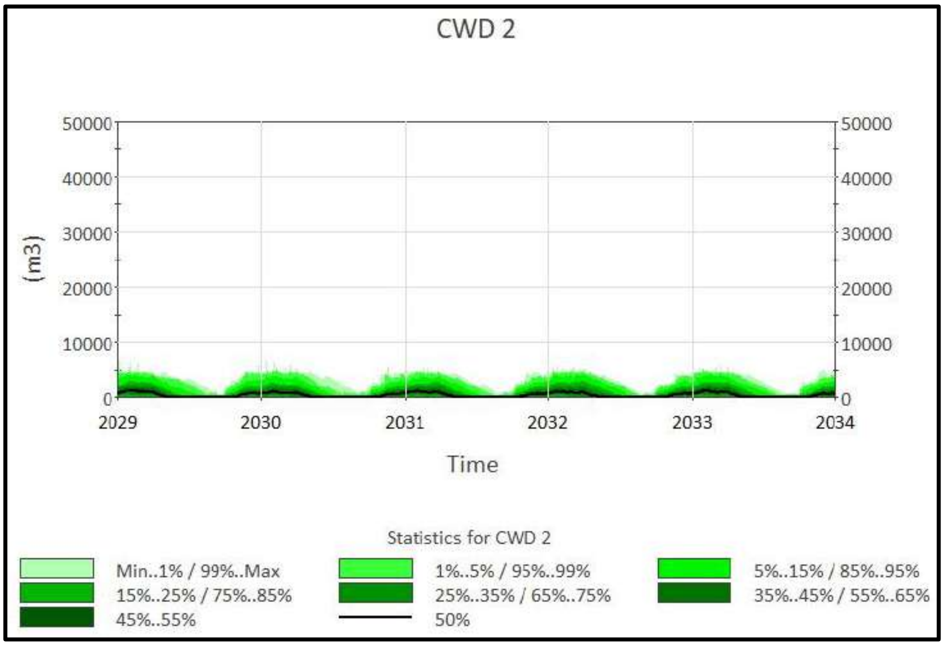


Figure 9: CWD 2 Volume Probabilities (Phase 1)

5.2 Water Balance Summary - Phase 1

The following section serves as a summary of the simulation results for Phase 1 of Kusile's 60 Year ADF Operations:

- The total capacity available on site for dirty water containment is 283,221 m³ during phase 1;
- The total mean average daily volume of dirty water contained in the PCDs on site is approximately 51,898 m³ during phase 1;
- The total 98th percentile (1:50-year) average daily volume of dirty water contained in the PCDs on site is approximately 157,374 m³ during phase 1;
- The total capacity available on site for clean water containment is 67,472 m³ during phase 1;
- The total mean average daily volume of clean water contained in the CWDs on site is approximately 1,227 m³ during phase 1;

- The total 98th percentile (1:50-year) average daily volume of clean water contained in the CWDs on site is approximately 6,187 m³ during phase 1;
- The total mean average daily dirty water available for dust suppression from PCD 1 is approximately 543.3 m³/day (198,290 m³/annum) and approximately 684.5 m³/day is required for dust suppression purposes during phase 1;
- The total mean average daily make-up water that needs to be transferred from the Holding Recycling Dam (HRD) within the power station to PCD 1 for dust suppression is approximately 141.2 m³/day during phase 1;
- The total mean average daily dirty water available for dust suppression from PCD 2 is approximately 97.2 m³/day (35,462 m³/annum) and approximately 432.7 m³/day is required for dust suppression purposes during phase 1; and
- The total mean average daily make-up water that needs to be transferred from the HRD within the power station to PCD 2 for dust suppression is approximately 291.5 m³/day during phase 1.

The daily water balance flow diagram for Kusile's Phase 1 operations is shown below in Figure 10 and the average annual water usage is listed in Table 10.

Figure 10: Kusile ADF Operations - Average Daily Water Usage CWDs (Phase 1)

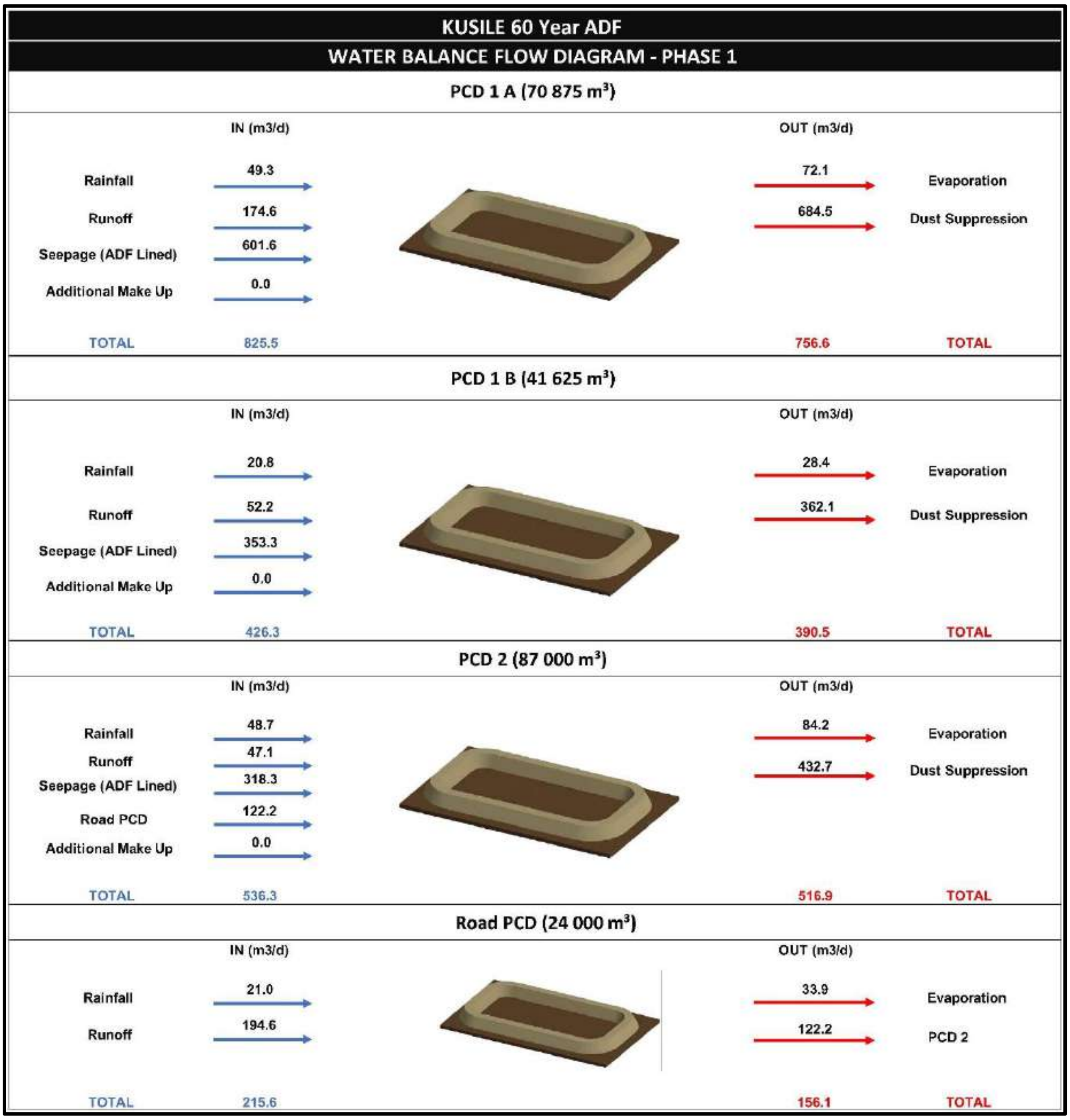


Figure 11: Kusile ADF Operations - Average Daily Water Usage PCDs (Phase 1)

Table 10: Kusile ADF Operations - Average Annual Water Usage during Phase 1

Kusile 60 Year ADF - Phase 1 Water Balance Flow Diagram (m^3/annum)						
Facility	Water In		Water Out		Balance	Comment
	Water Stream	Quantity (m^3/a)	Water Stream	Quantity (m^3/a)		
Runoff Areas	ADF Runoff	139 754	PCD 1 A	63 738		
	Rehabbed Areas	0	PCD 1 B	19 071		No Rehabilitation
	Conveyor	31 266	PCD 2	17 181		
			Road PCD	71 029		
	Total Inflow	171 020	Total Out	171 020	0	Adequate
PCD 1 A	Rainfall	17 984	Evaporation	26 327		
	Runoff	63 738	Dust Suppression	249 828		
	Seepage	219 591				ADF Lined
	Additional Make Up	0				HRD Make Up
	Total Inflow	301 314	Total Out	276,152	25 162	Adequate
PCD 1 B	Rainfall	7 595	Evaporation	10 359		
	Runoff	19 071	Dust Suppression	132 170		
	Seepage	128 966				ADF Lined
	Additional Make Up	0				HRD Make Up
	Total Inflow	155 632	Total Out	142 527	13 105	Adequate
	Rainfall	17 767	Evaporation	30 742		

PCD 2	Runoff	17 181	Dust Suppression	157 920		
	To PCD 2	44 610				
	Seepage	116 186				ADF Lined
	Additional Make Up	0				HRD Make Up
	Total Inflow	195 767	Total Out	188 661	7 106	Adequate
Road PCD	Rainfall	7 653	Evaporation	12 363		
	Runoff	71 029	To PCD 2	44 610		
	Total Inflow	78 682	Total Out	56 977	21 705	Adequate
CWD 1	Rainfall	8 763	Evaporation	8 526		
			Increase in Storage	237		
	Total Inflow	8 763	Total Out	8 526	0	Adequate
CWD 2	Rainfall	15 689	Evaporation	15 266		
			Increase in Storage	423		
	Total Inflow	15 689	Total Out	15 266	0	Adequate

6. WATER BALANCE RESULTS – PHASE 2

Phase 2 for Kusile operations will take place over five (5) years (Year 8 – 12 of simulations) and two more water containment facilities will be constructed. Considering the existing six containment facilities, six of them will be PCDs and the remaining two will be the CWDs. Dirty runoff from the ADF will be conveyed to PCD 1 A, PCD 1 B, PCD 2 Road PCD and PCD 3 Complex (PCD 3 and PCD 4). PCD 1 and PCD 2 will supply water for dust suppression purposes. PCD 3 Complex will transfer top up water to PCD 2 and any excess water from the PCD 3 Complex will overflow into PCD 2. The CWDs will receive water through rainfall and make-up water and this water will be used for irrigation purposes. The rehabilitation of the ADF will commence in this phase.

6.1 Phase 2 Water Balance Results

6.1.1 PCD 1 A

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 2.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF and conveyor; and
- Make up water for Dust Suppression.

Outflows from the PCD include:

- Evaporation;
- Excess water transferred to PCD 3 Complex (PCD 4); and
- Water supply to dust suppression.

The PCD volume probability result following the 250 realizations (from year 8 – 12) in the GoldSim software can be seen in Figure 12. The PCD on average utilises approximately 16% of its capacity, which is due to assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time and the additional dirty water runoff that the PCD will contain.

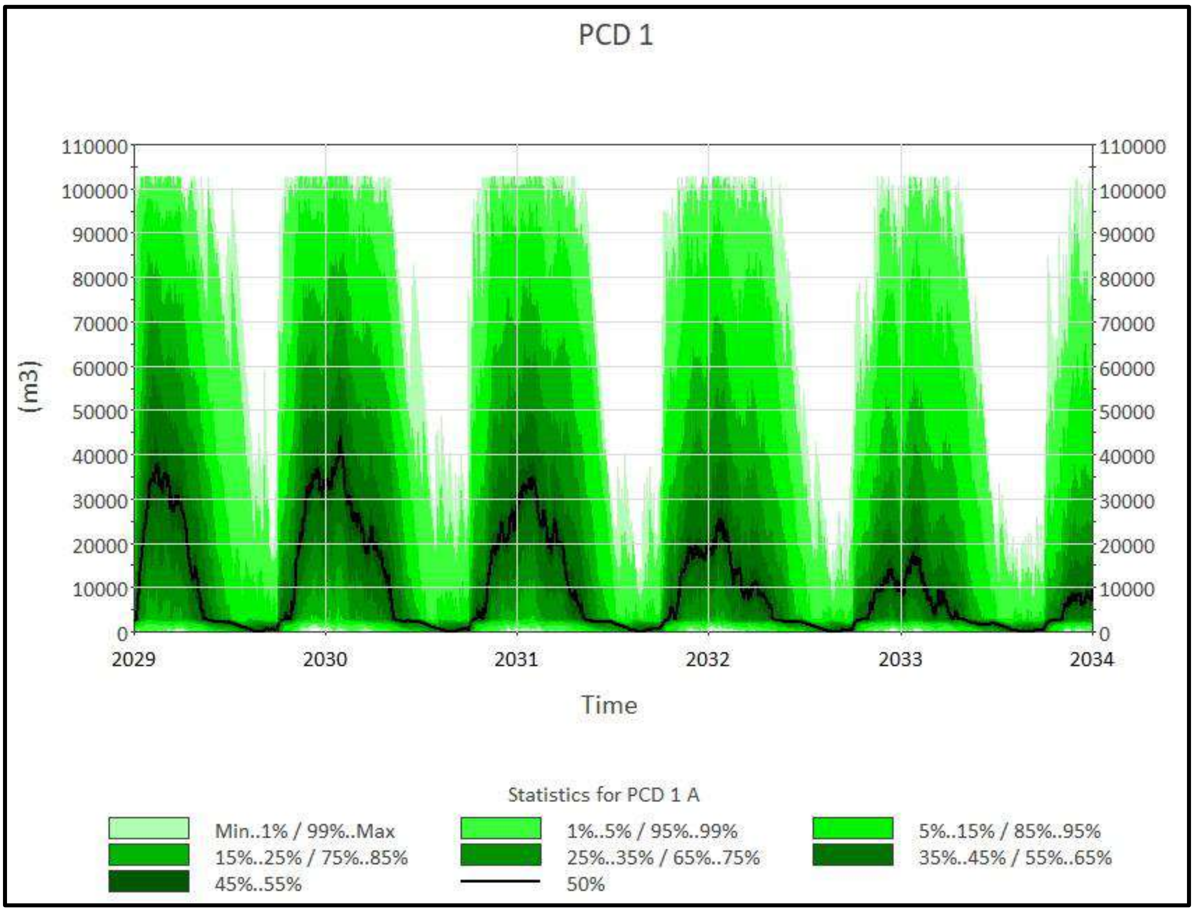


Figure 12: PCD 1 A Volume Probabilities (Phase 2)

6.1.2 PCD 1 B

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 2.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint; and
- Dirty water runoff from ADF.

Outflows from the PCD include:

- Evaporation; and
- Water supply to dust suppression.

The PCD volume probability result following the 250 realizations (from year 8 – 12) in the GoldSim software can be seen in Figure 13. The PCD on average utilises approximately 7% of its capacity, which is due to assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time. While the proportion of ADF runoff allocated to the PCD reduces from 28% to 18%, there is a larger overall increase in PCD area thus explains the slight increase in utilised capacity.

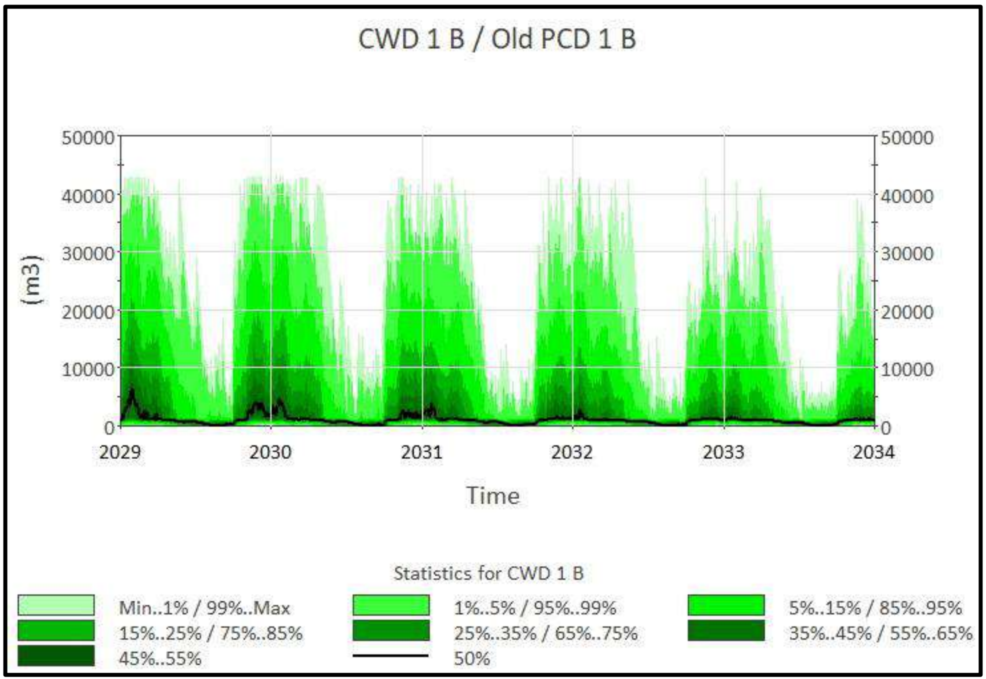


Figure 13: PCD 1 B Volume Probabilities (Phase 2)

6.1.3 PCD 2

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 2.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF;
- Top up water from PCD 3 Complex;
- Excess water transferred from Road PCD;
- Excess water from PCD 3 Complex; and
- Make up water for Dust Suppression.

Outflows from the PCD include:

- Evaporation;
- Water supply to dust suppression.

The PCD volume probability result following the 250 realizations (from year 8 – 12) in the GoldSim software can be seen in Figure 14. The PCD on average utilises approximately 10% of its capacity, which is due to the excess water received from Road PCD and the assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time.

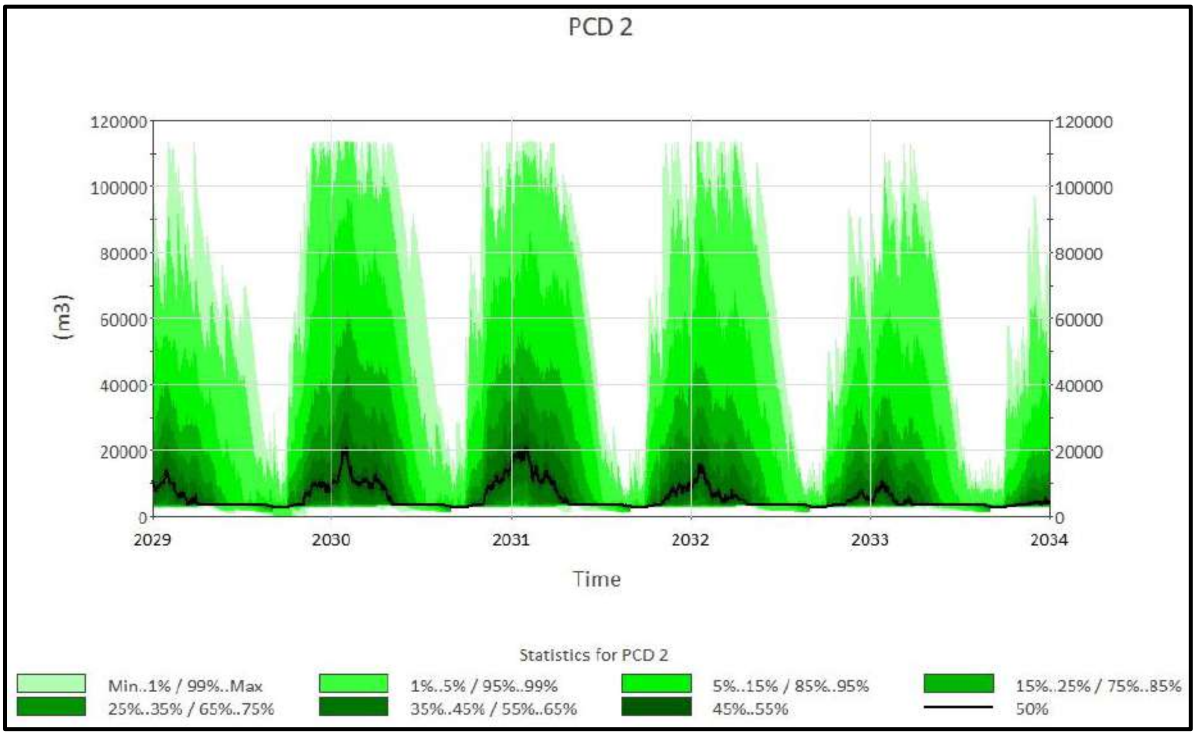


Figure 14: PCD 2 Volume Probabilities (Phase 2)

6.1.4 Road PCD

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 2.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint; and
- Dirty water runoff from ADF.

Outflows from the PCD include:

- Evaporation; and
- Excess water transferred to PCD 2.

The PCD volume probability result following the 250 realizations (from year 8 – 12) in the GoldSim software can be seen in Figure 15. The PCD on average utilises approximately

54% of its capacity. To prevent the PCD overflowing in winter months, it transfers excess water to PCD 2.

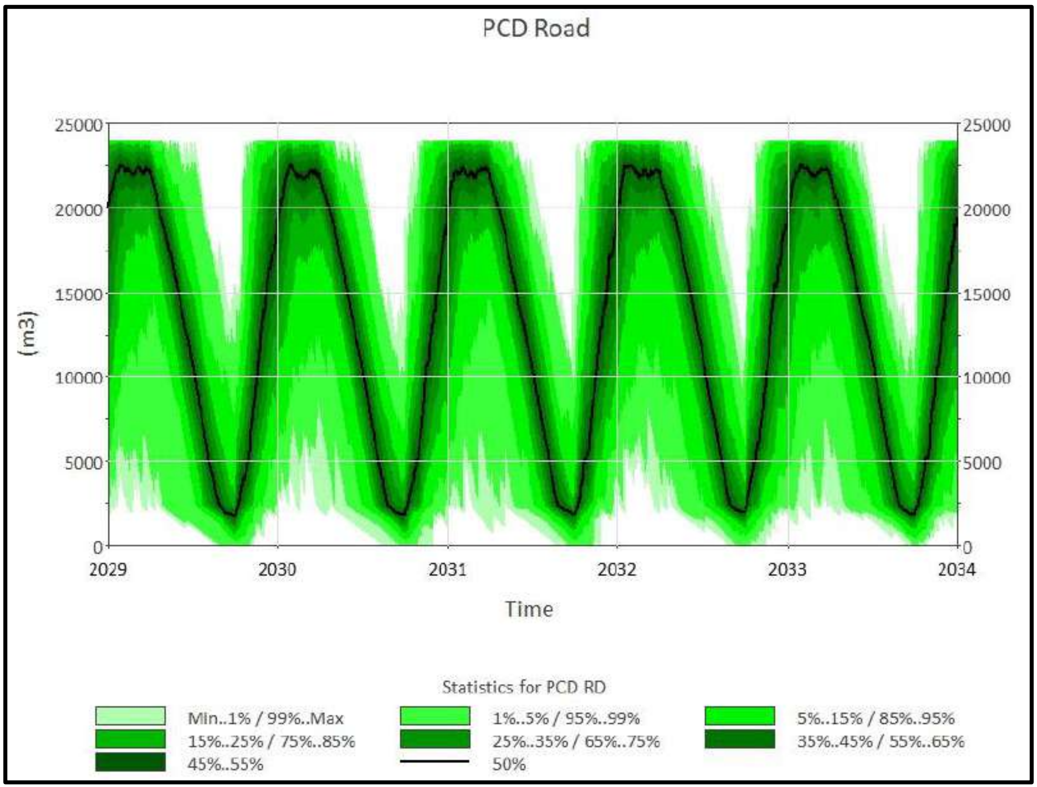


Figure 15: Road PCD Volume Probabilities (Phase 2)

6.1.5 PCD 3 Complex

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 2.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF; and
- Excess water from PCD 1;

Outflows from the PCD include:

- Evaporation;

- Top up water transferred to PCD 2; and
- Excess water to PCD 2.

The PCD volume probability result following the 250 realizations (from year 8 – 12) in the GoldSim software can be seen in Figure 16. The PCD on average utilises approximately 70% of its capacity. The PCD receives transfers from PCD 1 A-thus the large storage utilised. This PCD maintains an average of below 80% because it overflows to PCD 2 when its capacity exceeds 80%.

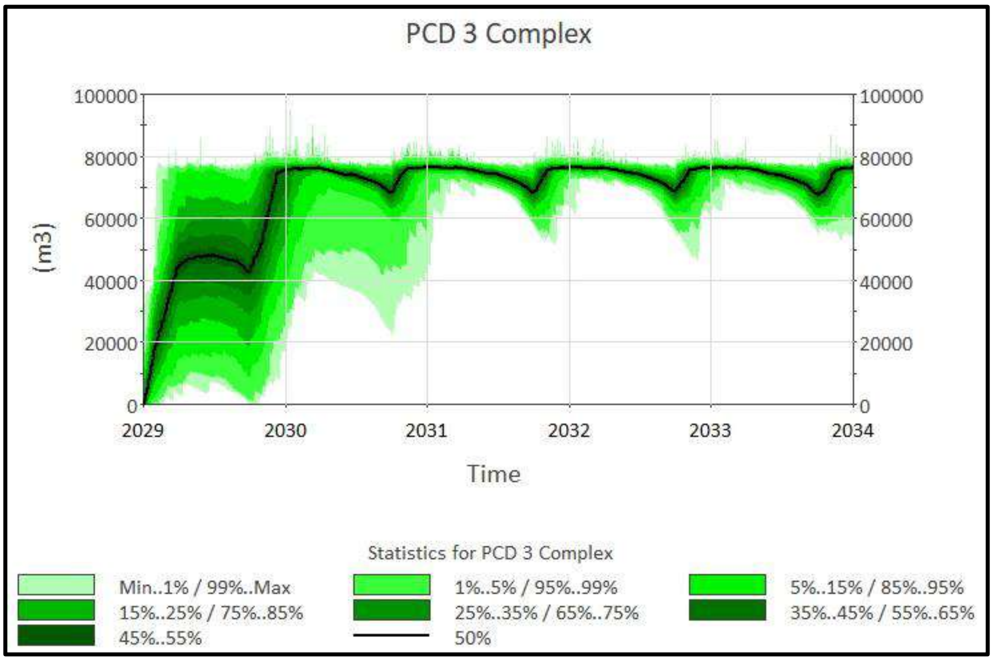


Figure 16: PCD 3 Complex Volume Probabilities (Phase 2)

6.1.6 CWDs

The CWDs were modelled in the GoldSim software to replicate likely operations on site for Phase 2.

Inflows into the CWDs (CWD 1 and 2) include:

- Direct rainfall onto the PCD footprint; and
- Make up water for irrigation purposes;

Outflows from the CWD include:

- Evaporation; and
- Clean water abstraction for Irrigation supply.

The CWD volume probability result following the 250 realizations (from year 8 – 12) in the GoldSim software for CWD 1 can be seen in Figure 18 and CWD 2 in Figure 17. Neither of the CWDs will experience any "release to the environment" during phase 2, as there is not enough water accumulating in the dams (inflows is only that of rainfall) and all water contained is used to supply water for irrigation purposes. During Phase 2, CWD 2 is simulated to accumulate a maximum of 7,000 m³ of clean water, this is less than 20% of the CWD's volumetric capacity.

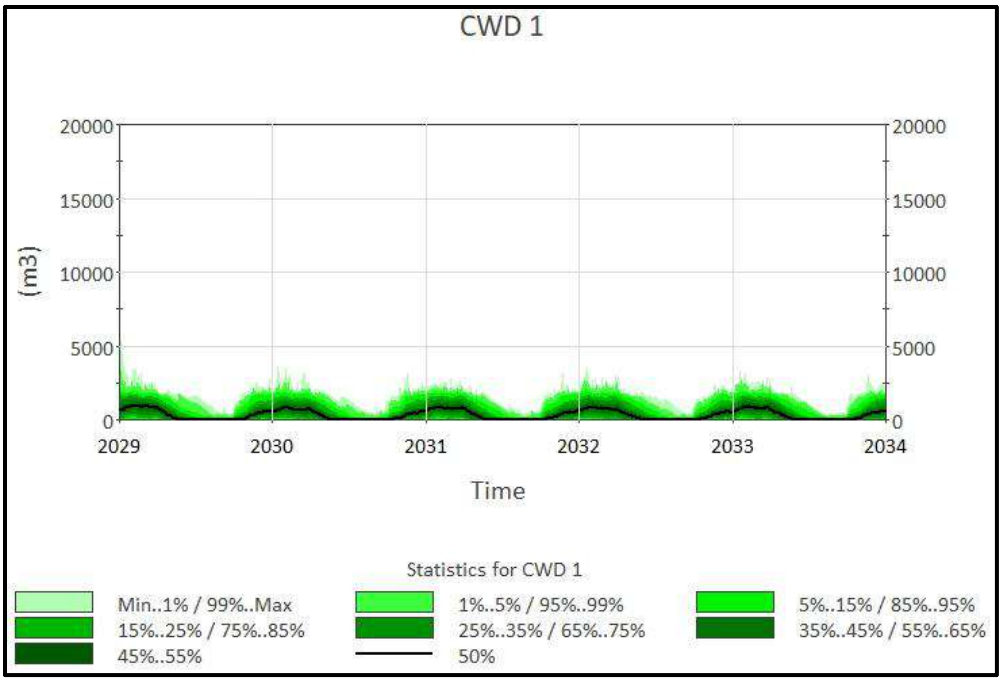


Figure 18: CWD 1 Volume Probabilities (Phase 2)

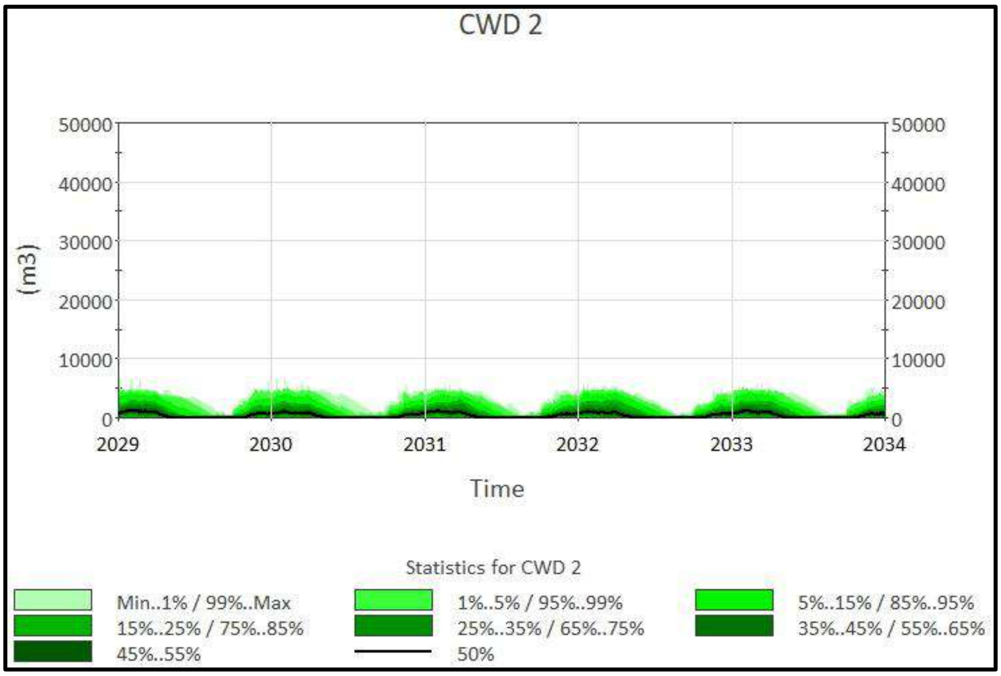


Figure 17: CWD 2 Volume Probabilities (Phase 2)

6.2 Water Balance Summary – Phase 2

The following section serves as a summary of the simulation results for Phase 2 of Kusile's 60 Year ADF Operations:

- The total capacity available on site for dirty water containment is 380,621 m³ during Phase 2;
- The total mean average daily volume of dirty water contained in the PCDs on site is approximately 42,941 m³ during Phase 2;
- The total 98th percentile (1:50-year) average daily volume of dirty water contained in the PCDs on site is approximately 233,424 m³ during Phase 2;
- The total capacity available on site for clean water containment is 67,472 m³ during Phase 2;
- The total mean average daily volume of clean water contained in the CWDs on site is approximately 1,001 m³ during Phase 2;
- The total 98th percentile (1:50-year) average daily volume of clean water contained in the CWDs on site is approximately 4,048 m³ during Phase 2;
- The total mean average daily dirty water available for dust suppression from PCD 1 (PCD 1 A and B) is approximately 619.6 m³/day (226,139 m³/annum) and approximately 3,454.3 m³/day (1,260,817 m³/annum) is required for dust suppression purposes during phase 2;
- The total mean average daily make-up water that needs to be transferred from the Holding Recycling Dam (HRD) within the power station to PCD 1 for dust suppression is approximately 2,834.7 m³/day (1,034,678 m³/annum) during phase 2;
- The total mean average daily dirty water available for dust suppression from PCD 2 is approximately 163.2 m³/day (59,569 m³/annum) and approximately 1,800 m³/day (657,007 m³/annum) is required for dust suppression purposes during phase 2;
- The total mean average daily make-up water that is transferred from PCD 3 Complex to PCD 2 for dust suppression is approximately 60 m³/day (21,901 m³/annum) during Phase 2;

- The total mean average daily make-up water that needs to be transferred from the HRD within the power station to PCD 2 for dust suppression is approximately 1,636.8 m³/day (597,438 m³/annum) during phase 2;
- The total mean average daily clean water available for irrigation from the CWDs is approximately 2.5 m³/day (917 m³/annum) and approximately 1,108.8 m³/day (404,714 m³/annum) is required for irrigation purposes during phase 2;
- The total mean average daily make-up water for irrigation is approximately 1,106.3 m³/day (403,797 m³/annum) during Phase 2, effectively 99.8% of water for irrigation purposes will need to be supplied from an external source;
- The total mean average daily dirty water transferred from PCD 1 and 2 to PCD 3 Complex is approximately 99.1 m³/day (36,164 m³/annum) during Phase 2; and
- PCD 3 Complex has an initial storage of 0 m³ when the simulation began. Over the 5-year period the volume in the PCD increased by approximately 23 m³/day (8,411 m³/annum) over 5 years. At the end of Phase 2 the PCD will have approximately 42,000 m³ of water stored.

The daily water balance flow diagram for CWDs is shown below in Figure 19, the PCDs are shown in Figure 20 and the average annual water usage for Kusile's Phase 2 operations is listed in Table 11.

KUSILE 60 Year ADF - CWDs
WATER BALANCE FLOW DIAGRAM - PHASE 2

Runoff Areas

	IN (m3/d)	OUT (m3/d)	
ADF Runoff	482.9	221.1	PCD 1 A
Rehabbed Areas	0.0	51.4	PCD 1 B
Conveyor	86.8	71.5	PCD 2
		28.6	PCD 3 & 4
		197.1	Road PCD
		0.0	CWD 1 & 2
TOTAL	569.7	569.7	TOTAL

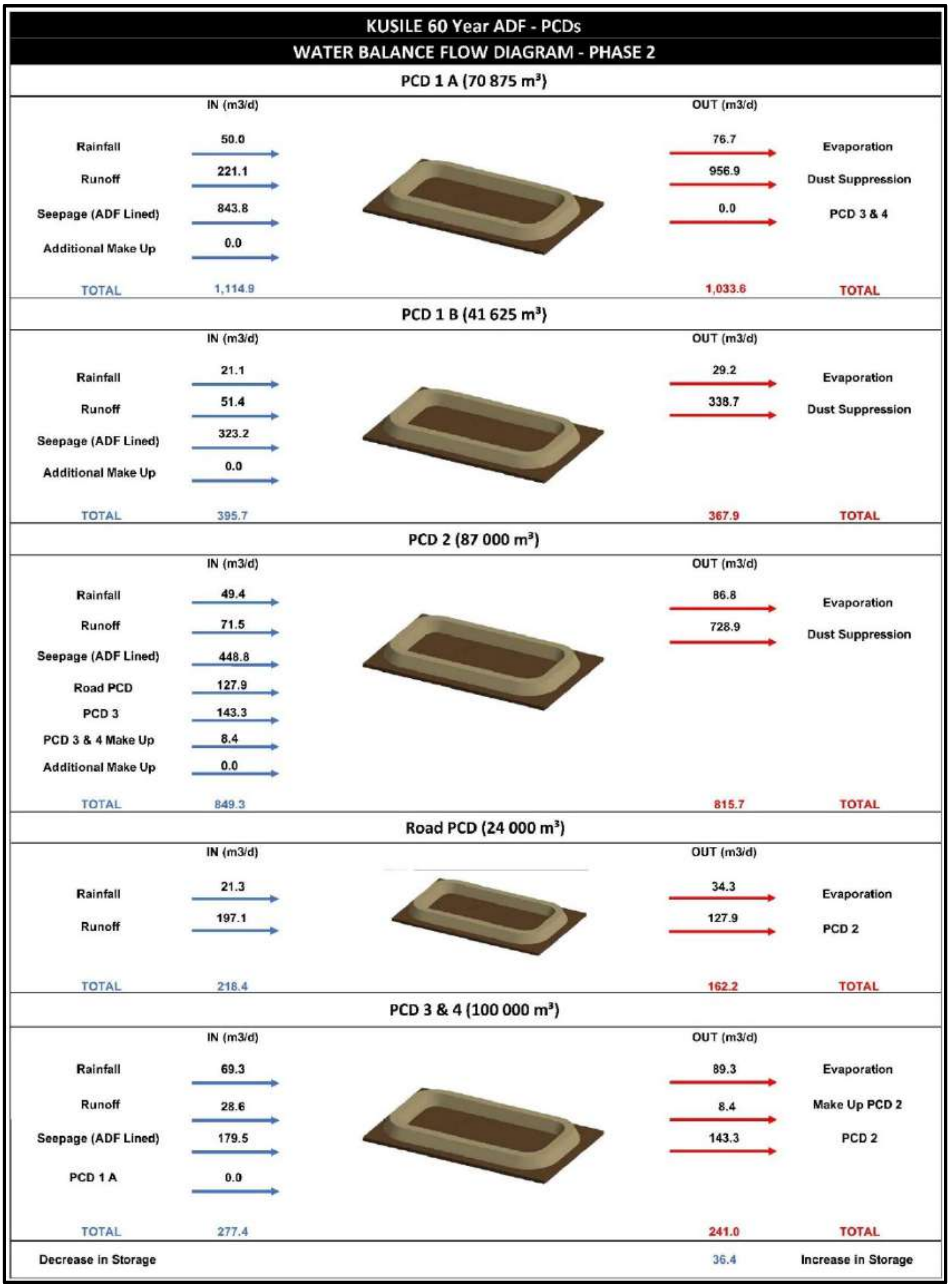
CWD 1 (15 000 m³)

	IN (m3/d)	OUT (m3/d)	
Rainfall	24.4	22.9	Evaporation
Runoff	0.0	0.9	Irrigation
Additional Make Up	0.0	0.0	Environmental Discharge
TOTAL	24.4	23.7	TOTAL
Decrease in Storage		0.7	Increase in Storage

CWD 2 (55 000 m³)

	IN (m3/d)	OUT (m3/d)	
Rainfall	43.6	42.5	Evaporation
Runoff	0.0	0.2	Irrigation
Additional Make Up	0.0	0.0	Environmental Discharge
TOTAL	43.6	42.7	TOTAL
Decrease in Storage		0.9	Increase in Storage

Figure 19: Kusile ADF Operations - Average Daily Water Usage CWDs (Phase 2)



KUSILE 60 Year ADF - PCDs

WATER BALANCE FLOW DIAGRAM - PHASE 2

PCD 1 A (70 875 m^3)

	IN (m3/d)	OUT (m3/d)	
Rainfall	50.0	76.7	Evaporation
Runoff	221.1	956.9	Dust Suppression
Seepage (ADF Lined)	843.8	0.0	PCD 3 & 4
Additional Make Up	0.0		
TOTAL	1,114.9	1,033.6	TOTAL

PCD 1 B (41 625 m^3)

	IN (m3/d)	OUT (m3/d)	
Rainfall	21.1	29.2	Evaporation
Runoff	51.4	338.7	Dust Suppression
Seepage (ADF Lined)	323.2		
Additional Make Up	0.0		
TOTAL	395.7	367.9	TOTAL

PCD 2 (87 000 m^3)

	IN (m3/d)	OUT (m3/d)	
Rainfall	49.4	86.8	Evaporation
Runoff	71.5	728.9	Dust Suppression
Seepage (ADF Lined)	448.8		
Road PCD	127.9		
PCD 3	143.3		
PCD 3 & 4 Make Up	8.4		
Additional Make Up	0.0		
TOTAL	849.3	815.7	TOTAL

Road PCD (24 000 m^3)

	IN (m3/d)	OUT (m3/d)	
Rainfall	21.3	34.3	Evaporation
Runoff	197.1	127.9	PCD 2
TOTAL	218.4	162.2	TOTAL

PCD 3 & 4 (100 000 m^3)

	IN (m3/d)	OUT (m3/d)	
Rainfall	69.3	89.3	Evaporation
Runoff	28.6	8.4	Make Up PCD 2
Seepage (ADF Lined)	179.5	143.3	PCD 2
PCD 1 A	0.0		
TOTAL	277.4	241.0	TOTAL
Decrease in Storage		36.4	Increase in Storage

Figure 20: Kusile ADF Operations - Average Daily Water Usage PCDs (Phase 2)

Table 11: Kusile ADF Operations - Average Annual Water Usage during Phase 2

Kusile 60 Year ADF - Phase 2 Water Balance Flow Diagram (m^3/annum)						
Facility	Water In		Water Out		Balance	Comment
	Water Stream	Quantity (m^3/a)	Water Stream	Quantity (m^3/a)		
Runoff Areas	ADF Runoff	176 277	PCD 1 A	80 704		
	Rehabbed Areas	0	PCD 1 B	18 778		Rehab Areas
	Conveyor	31 673	PCD 2	26 080		
			Road PCD	71 954		
			PCD 3 & 4	10 432		
			CWD 1 & 2	0		
	Total	207 949	Total	207 975	≈ 0.00	Adequate
PCD 1 A (70 875 m^3)	Rainfall	18 245	Evaporation	27 989		
	Runoff	80 704	Dust Suppression	349 269		
	Seepage	307 993	PCD 3 & 4	0		ADF Lined
	Additional Make Up	0				HRD Make Up
	Total	406 940	Total	377 255	29 685	Adequate
PCD 1 B (41 625 m^3)	Rainfall	7 705	Evaporation	10 669		
	Runoff	18 778	Dust Suppression	123 631		
	Seepage	117 953				ADF Lined
	Additional Make Up	0				HRD Make Up

	Total	144 437	Total	134 299	10 138	Adequate
PCD 2 (87 000 m^3)	Rainfall	18 025	Evaporation	31 675		
	Runoff	26 080	Dust Suppression	266 038		
	Seepage	163 826	PCD 4	0		ADF Lined
	PCD 3	52 322				
	Road PCD	46 687				
	PCD 3 & 4 Make Up	3 079				
	Additional Make Up	0				HRD Make Up
	Total	300 441	Total	297 714	2 727	Adequate
Road PCD	Rainfall	7 764	Evaporation	12 513		
	Runoff	71 954	To PCD 2	46 687		
	Seepage	0				ADF Lined
	Additional Make Up	0				HRD Make Up
	Total Inflow	79 718	Total Out	59 205	20 513	Adequate
PCD 3 & 4 (100 000 m^3)	Rainfall	25 296	Evaporation	32 603		
	Runoff	10 432	Make Up PCD 2	3 079		
	Seepage	65 530	To PCD 2	52 322		ADF Lined
	PCD 1 & 2	36 164	Increase in Storage	8 411		Initial Volume = 0 m^3

	Total	101 259	Total	88 004	13 255	Adequate
CWD 1 (15 000 m³)	Rainfall	8 890	Evaporation	8 362		
	Runoff	0	Irrigation	326		
	Additional Make Up	0				
	Total	8 890	Total	8 688	202	Adequate
CWD 2 (55 000 m³)	Rainfall	15 917	Evaporation	15 514		
	Runoff	0	Irrigation	64		
	Additional Make Up	0				
	Total	15 917	Total	15 578	339	Adequate

7. WATER BALANCE RESULTS – PHASE 3

Phase 3 for Kusile operations will take place over five (5) years (Year 13 – 17 of simulations) and in this phase, PCD 1 B will be transitioned into a clean water dam (CWD 1 B). Considering this transition and the existing eight containment facilities, five of them will be PCDs and the remaining three will be the CWDs. Dirty runoff from the ADF will be conveyed to PCD 1 A, PCD 2, Road PCD and PCD 3 Complex (PCD 3 and PCD 4). PCD 1 and PCD 2 will supply water for dust suppression purposes. PCD 3 Complex will transfer top up water to PCD 2 and any excess water from PCD 3 Complex will overflow into PCD 2. The CWDs will receive water through rainfall, runoff from the rehabilitated areas on the ADF and make-up water and this water will be used for irrigation purposes on the irrigated areas of the ADF.

7.1 Phase 3 Water Balance Results

7.1.1 PCD 1 A

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 3.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF and conveyor; and
- Make up water for Dust Suppression.

Outflows from the PCD include:

- Evaporation;
- Excess water transferred to PCD 3 Complex (PCD 4); and
- Water supply to dust suppression.

The PCD volume probability result following the 250 realizations (Year 13 – 17) in the GoldSim software can be seen in Figure 21. The PCD on average utilises approximately 11% of its capacity, which is due to assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time and the additional dirty water runoff that the

PCD will contain. The average storage capacity of the PCD reduces from 16% to 11% because of an overall reduction in dirty runoff areas conveying water to the PCD. In phase 2, more water is conveyed to PCD 2 and PCD 3 Complex.

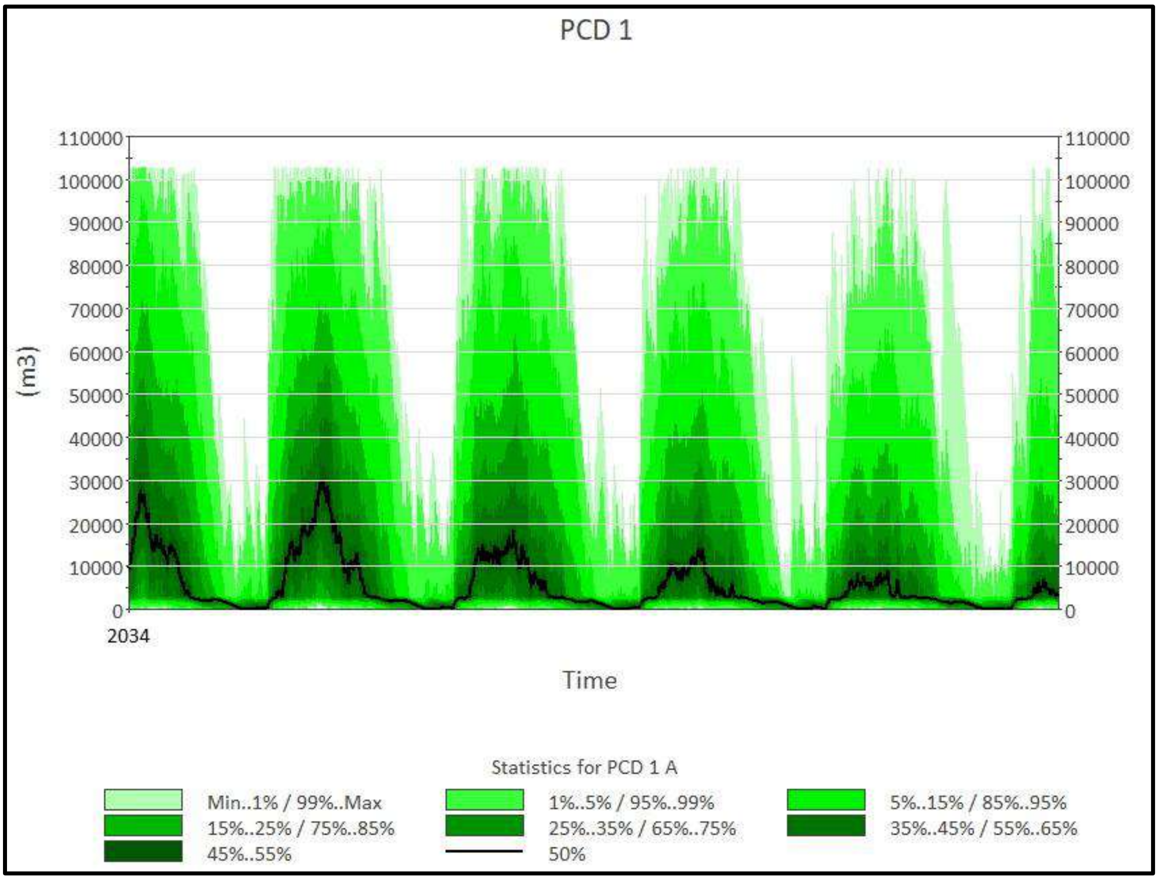


Figure 21: PCD 1 A Volume Probabilities (Phase 3)

7.1.2 PCD 2

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 3.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF;
- Top up water from PCD 3 Complex;
- Excess water from PCD 3 Complex; and
- Make up water for Dust Suppression.

Outflows from the PCD include:

- Evaporation;
- Water supply to dust suppression.

The PCD volume probability result following the 250 realizations (Year 13 – 17) in the GoldSim software can be seen in Figure 22. The PCD on average utilises approximately 14% of its capacity, which is due to the excess water received from Road PCD and the assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time. The average storage capacity of the PCD increases from 10% to 14% because of an overall increase in dirty runoff areas conveying water to the PCD. In phase 2, more water is conveyed to PCD 2 and PCD 3 Complex.

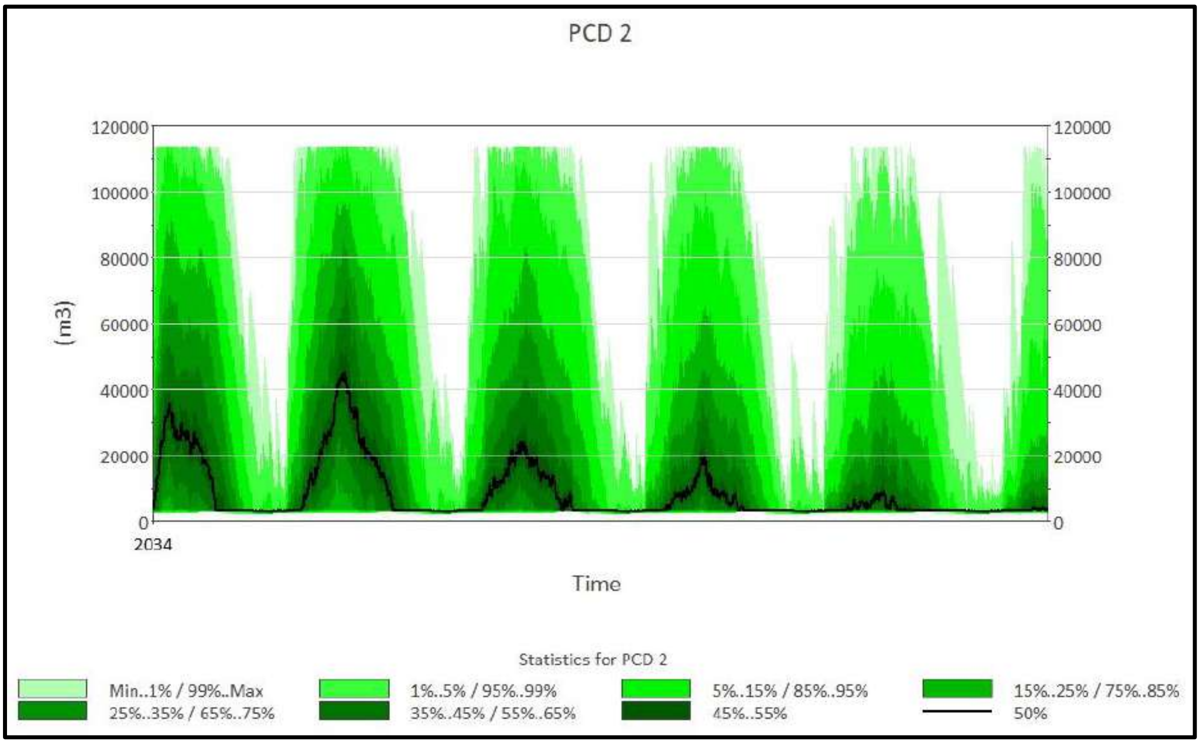


Figure 22: PCD 2 Volume Probabilities (Phase 3)

7.1.3 Road PCD

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 3.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint; and
- Dirty water runoff from ADF.

Outflows from the PCD include:

- Evaporation; and
- Excess water transferred to PCD 2.

The PCD volume probability result following the 250 realizations (Year 13 – 17) in the GoldSim software can be seen in Figure 23. The PCD on average utilises approximately

54% of its capacity. To prevent the PCD overflowing in winter months, it transfers excess water to PCD 2.

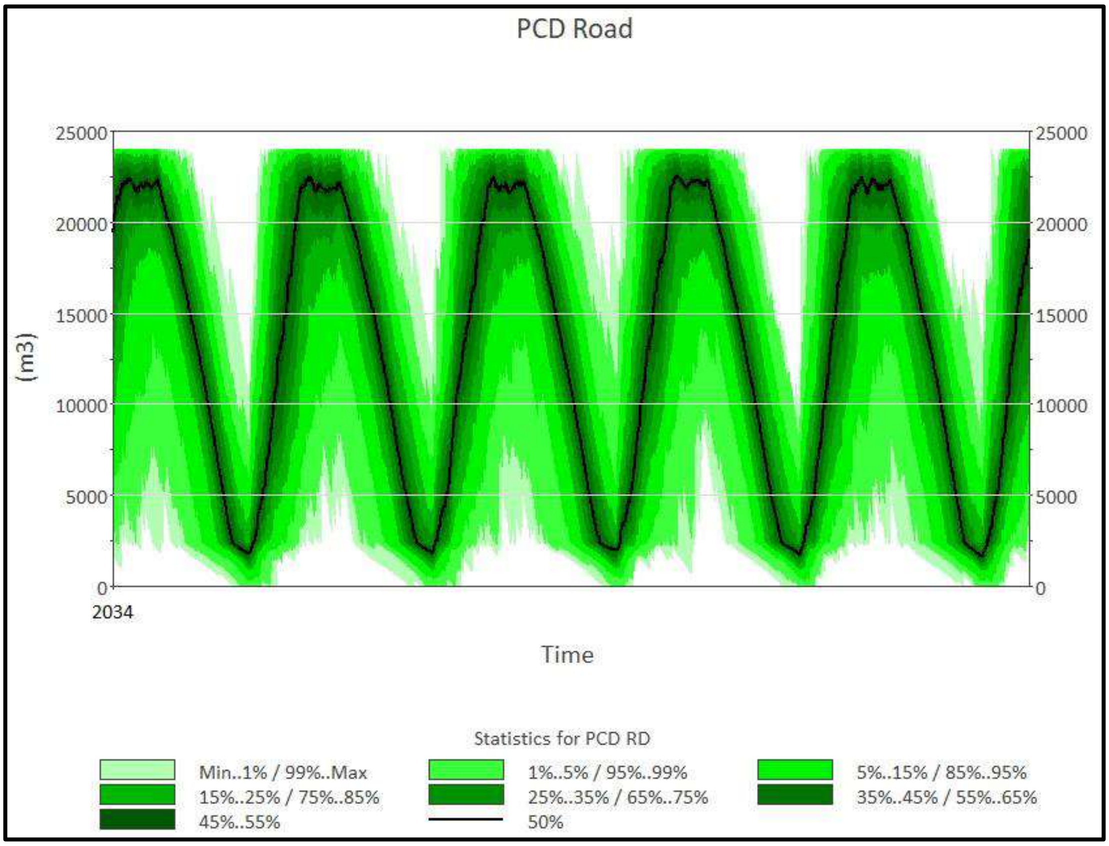


Figure 23: Road PCD Volume Probabilities (Phase 3)

7.1.4 PCD 3 Complex

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 3.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF; and
- Excess water from PCD 1;

Outflows from the PCD include:

- Evaporation;
- Top up water transferred to PCD 2; and
- Excess water to PCD 2.

The PCD volume probability result following the 250 realizations (Year 13 – 17) in the GoldSim software can be seen in Figure 23. The PCD on average utilises approximately 70% of its capacity. The PCD receives transfers from PCD 1 A-thus the large storage utilised. This PCD maintains an average of below 80% because it overflows to PCD 2 when its capacity exceeds 80%.

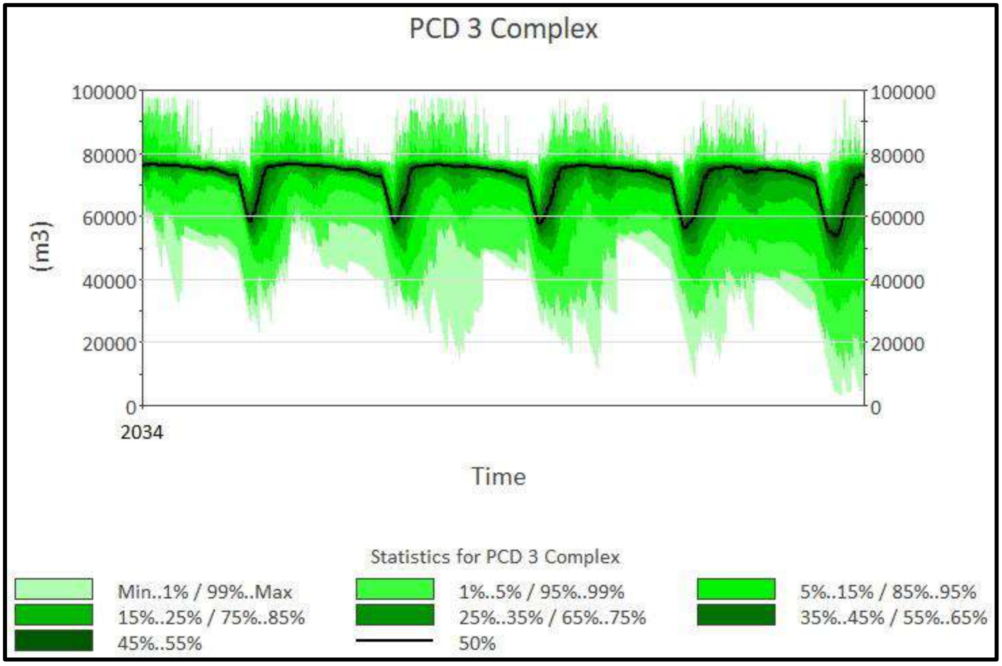


Figure 24: PCD 3 Complex Volume Probabilities (Phase 3)

7.1.5 CWDs

The CWDs were modelled in the GoldSim software to replicate likely operations on site for Phase 3.

Inflows into the CWDs (CWD 1, CWD 1 B and CWD 2) include:

- Direct rainfall onto the PCD footprint;
- Clean water runoff from rehabilitated ADF areas; and
- Make up water for irrigation purposes;

Outflows from the CWD include:

- Evaporation; and
- Clean water abstraction for Irrigation supply.

The CWD volume probability result following the 250 realizations (from year 13 – 17) in the GoldSim software for CWD 1 can be seen in Figure 25, CWD 2 in Figure 26 and CWD 1 B in Figure 27. Neither of the CWDs will experience any excess water that needs to be “released into the environment” but there are instances where the CWDs are simulated to reach Full Supply Level (FSL) during Phase 3, as more water is accumulating in the dams (due to the clean water runoff generated from the rehabilitated areas on the ADF) and all water contained is used to supply water for irrigation purposes.

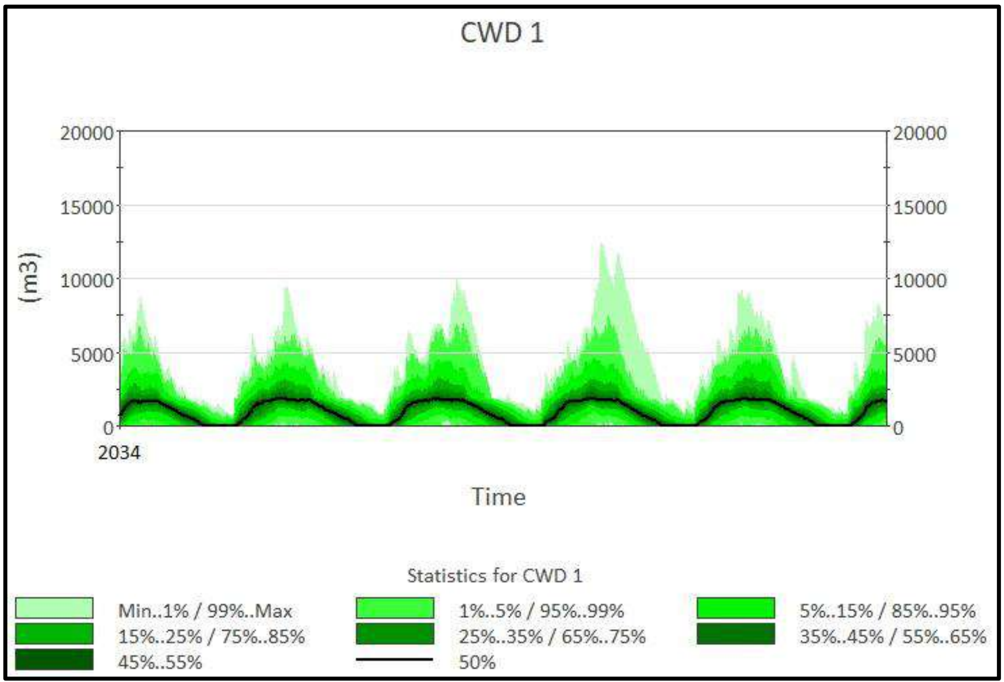


Figure 25: CWD 1 Volume Probabilities (Phase 3)

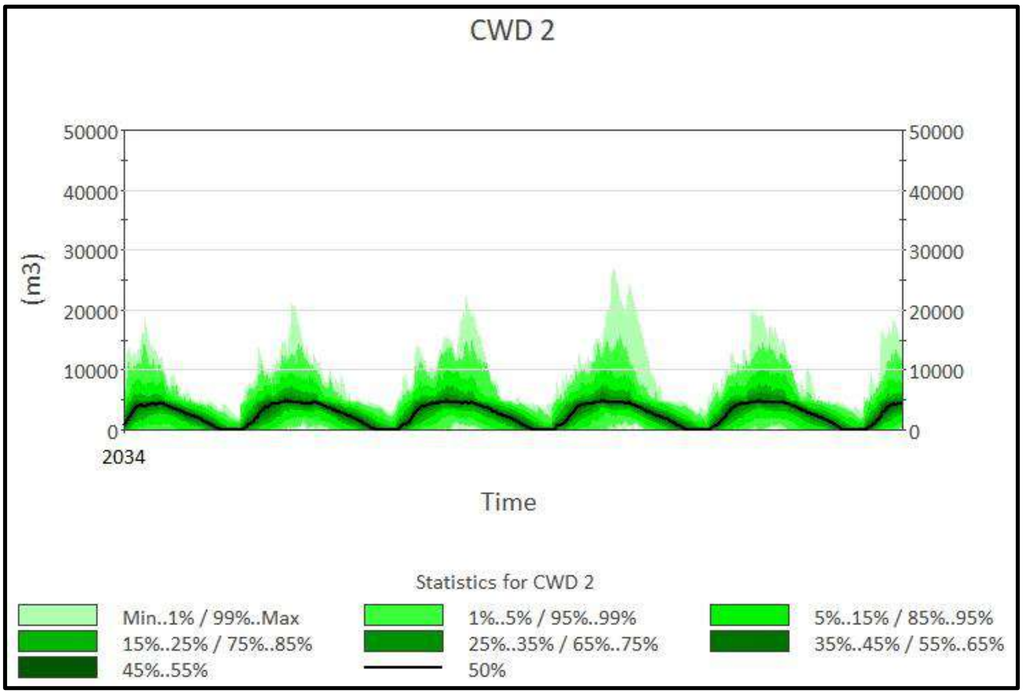


Figure 26: CWD 2 Volume Probabilities (Phase 3)

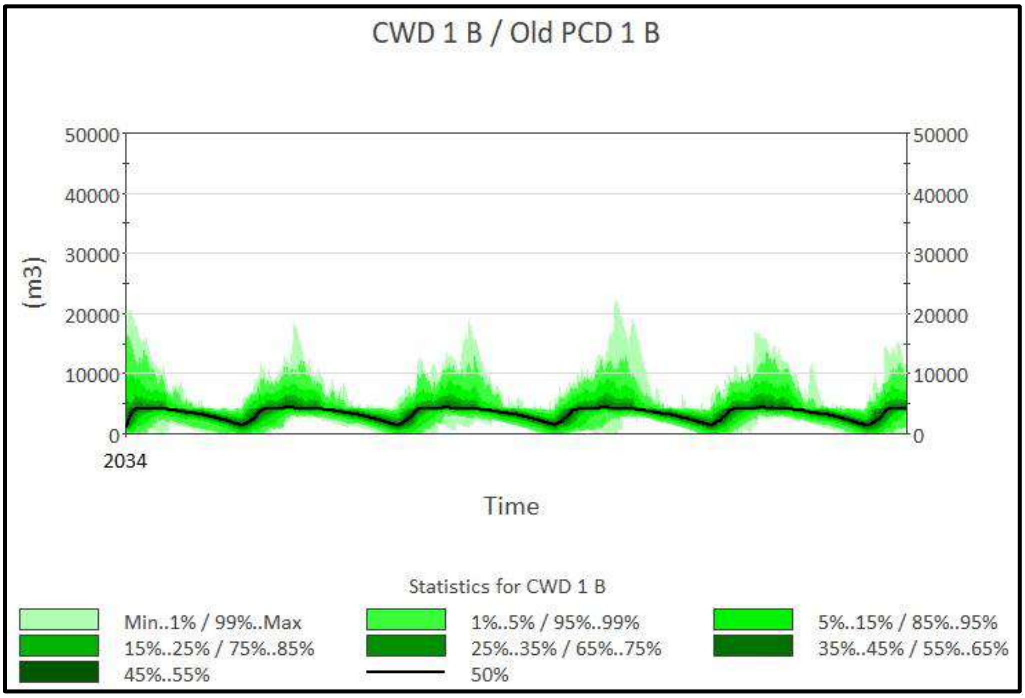


Figure 27: CWD 1B / Old PCD 1 B Volume Probabilities (Phase 3)

7.2 Water Balance Summary – Phase 3

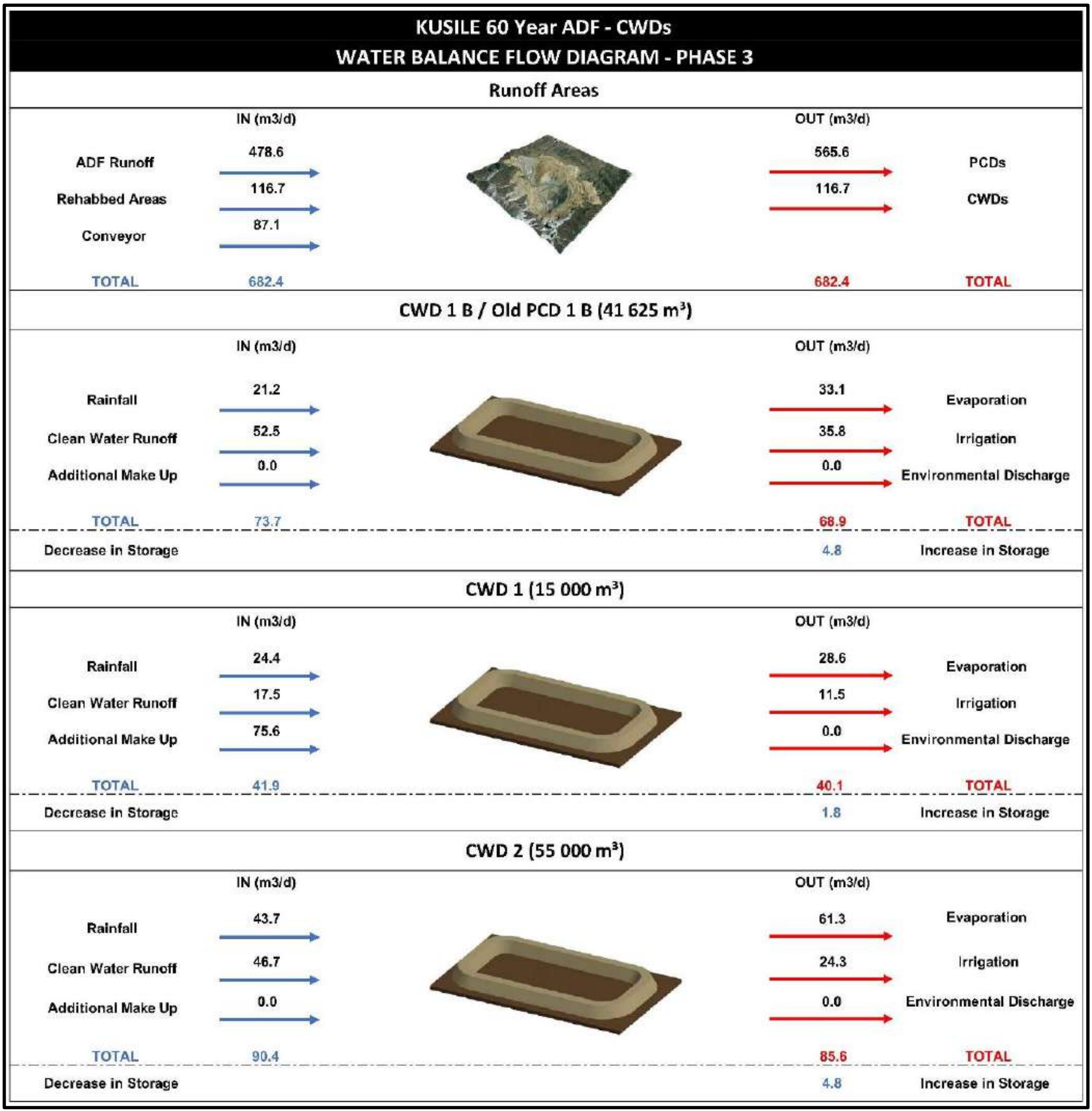
The following section serves as a summary of the simulation results for Phase 3 of Kusile's 60 Year ADF Operations:

- The total capacity available on site for dirty water containment is 337,746 m³, approximately 42,875 m³ of storage for dirty water is lost during this phase;
- The total mean average daily volume of dirty water contained in the PCDs on site is approximately 39,290 m³ during Phase 3;
- The total 98th percentile (1:50-year) average daily volume of dirty water contained in the PCDs on site is approximately 222,172 m³ during Phase 3;
- The loss of the 42,875 m³ to contain dirty water causes one overflow at PCD 2 during simulations, however, one spillage occurrence is allowed according to NWA (GN 704);
- Due to the transition of PCD 1 B to a CWD, the total capacity available on site for clean water containment increased to 110,413 m³ during Phase 3;

- The total mean average daily volume of clean water contained in the CWDs on site is approximately 7,294 m³ during Phase 3;
- The total 98th percentile (1:50-year) average daily volume of clean water contained in the CWDs on site is approximately 16,825 m³ (increased by five times the amount when compared to Phase 1 and 2) during Phase 3;
- The total mean average daily dirty water available for dust suppression from PCD 1 is approximately 487 m³/day (177,763 m³/annum) and approximately 2,523.3 m³/day (921,000 m³/annum) is required for dust suppression purposes during Phase 3;
- The total mean average daily make-up water that needs to be transferred from the Holding Recycling Dam (HRD) within the power station to PCD 1 for dust suppression is approximately 2,036.3 m³/day (743,237 m³/annum) during Phase 3;
- The total mean average daily dirty water available for dust suppression from PCD 2 is approximately 400.3 m³/day (146,114 m³/annum) and approximately 2,523.3 m³/day (921,000 m³/annum) is required for dust suppression purposes during Phase 3;
- The total mean average daily make-up water that is transferred from PCD 3 Complex to PCD 2 for dust suppression is approximately 190.2 m³/day (69,433 m³/annum) during Phase 3;
- The total mean average daily make-up water that needs to be transferred from the HRD within the power station to PCD 2 for dust suppression is approximately 2,123 m³/day (774,886m³/annum) during Phase 3;
- The total mean average daily clean water available for irrigation from the CWDs is approximately 26.4 m³/day (9,637 m³/annum) and approximately 546.3 m³/day (199,413 m³/annum) is required for irrigation purposes during Phase 3;
- The total mean average daily make-up water for irrigation is approximately 519.9 m³/day (189,776 m³/annum) during Phase 3, approximately 95% of water for irrigation purposes will need to be supplied from an external source;
- The total mean average daily dirty water transferred from PCD 1 and 2 to PCD 3 Complex is approximately 101.2 m³/day (36,920 m³/annum) during Phase 3; and
- Following the results obtained from Phase 2, PCD 3 Complex had an increase in storage capacity of approximately 42,000 m³, resulting in this storage being he initial storage on the first day of simulations in phase 3. On the last day of simulation for phase 3, a

“decrease in storage” was noticed at PCD 3 complex. The total mean average daily decrease in storage at PCD 3 Complex is approximately 21 m³/day (7,669 m³/annum) during Phase 3.

The daily water balance flow diagram for CWDs is shown below in Figure 28, the PCDs are shown in Figure 29 and the average annual water usage for Kusile’s Phase 3 operations is listed in **Table 12**.



KUSILE 60 Year ADF - CWDs

WATER BALANCE FLOW DIAGRAM - PHASE 3

Runoff Areas

	IN (m3/d)	OUT (m3/d)	
ADF Runoff	478.6	565.6	PCDs
Rehabbed Areas	116.7	116.7	CWDs
Conveyor	87.1		
TOTAL	682.4	682.4	TOTAL

CWD 1 B / Old PCD 1 B (41 625 m³)

	IN (m3/d)	OUT (m3/d)	
Rainfall	21.2	33.1	Evaporation
Clean Water Runoff	52.5	35.8	Irrigation
Additional Make Up	0.0	0.0	Environmental Discharge
TOTAL	73.7	68.9	TOTAL
Decrease in Storage		4.8	Increase in Storage

CWD 1 (15 000 m³)

	IN (m3/d)	OUT (m3/d)	
Rainfall	24.4	28.6	Evaporation
Clean Water Runoff	17.5	11.5	Irrigation
Additional Make Up	75.6	0.0	Environmental Discharge
TOTAL	41.9	40.1	TOTAL
Decrease in Storage		1.8	Increase in Storage

CWD 2 (55 000 m³)

	IN (m3/d)	OUT (m3/d)	
Rainfall	43.7	61.3	Evaporation
Clean Water Runoff	46.7	24.3	Irrigation
Additional Make Up	0.0	0.0	Environmental Discharge
TOTAL	90.4	85.6	TOTAL
Decrease in Storage		4.8	Increase in Storage

Figure 28: Kusile ADF Operations - Average Daily Water Usage CWDs (Phase 3)

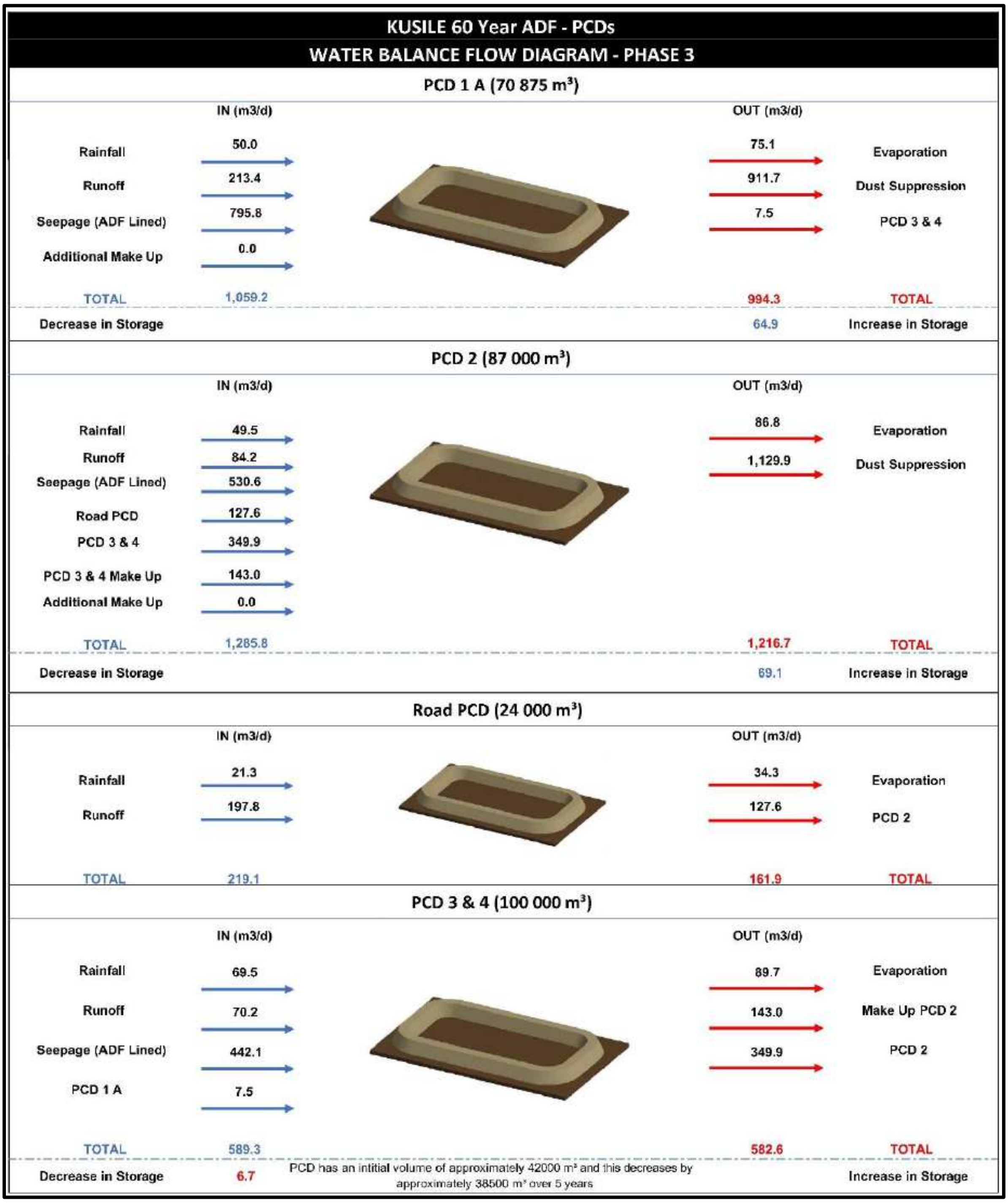


Figure 29: Kusile ADF Operations - Average Daily Water Usage PCDs (Phase 3)

Table 12: Kusile ADF Operations - Average Annual Water Usage during Phase 3

Kusile 60 Year ADF - Phase 3 Water Balance Flow Diagram (m^3/annum)						
Facility	Water In		Water Out		Balance	Comment
	Water Stream	Quantity (m^3/a)	Water Stream	Quantity (m^3/a)		
Runoff Areas	ADF Runoff	174 679	PCDs	206 461		
	Rehabbed Areas	42 608	CWDs	42 608		Rehab Areas
	Conveyor	31 782				
	Total	249 068	Total	249 068	0.00	Adequate
PCD 1 A (70 875 m^3)	Rainfall	18 289	Evaporation	27 424		
	Runoff	77 897	Dust Suppression	332 765		
	Seepage	290 482	PCD 3 & 4	2 720		ADF Lined
	Additional Make Up	0				
	Total	386 665	Total	362 909	23 756	Adequate
CWD 1 B / Old PCD 1 B (41 625 m^3)	Rainfall	7 723	Evaporation	12 098		
	Clean Water Runoff	19 173	Irrigation	13 069		
	Additional Make Up	0	Environmental Discharge	0		
	Total	26 896	Total	25 167	1 729	Adequate

PCD 2 (87 000 m³)	Rainfall	18 068	Evaporation	31 688		
	Runoff	30 743	Dust Suppression	412 418		
	Seepage	193 655	PCD 3 & 4	21 587		ADF Lined
	PCD 3 & 4	127 705				
	Road PCD	46 585				
	PCD 3 & 4 Make Up	52 197				
	Additional Make Up	0				
	Total	469 333	Total	444 110	25 223	🟩 Adequate
Road PCD	Rainfall	7 782	Evaporation	12 512		
	Runoff	72 201	To PCD 2	46 585		
	Seepage	0				ADF Lined
	Additional Make Up	0				HRD Make Up
	Total Inflow	79 984	Total Out	59 101	20 883	🟩 Adequate
PCD 3 & 4 (100 000 m³)	Rainfall	25 357	Evaporation	32 734		
	Runoff	25 619	Make Up PCD 2	52 197		
	Seepage	161 379	To PCD 2	127 705		ADF Lined
	PCD 1A	2 720				

	Decrease in Storage	7 669				Initial Volume = 42,000 m³
	Total	215 087	Total	212 637	2 450	Adequate
CWD 1 (15 000 m³)	Rainfall	8 912	Evaporation	10 424		
	Clean Water Runoff	6 391	Irrigation	4 197		
	Additional Make Up	0	Environmental Discharge	0		
	Total	15 303	Total	14 621	682	Adequate
CWD 2 (55 000 m³)	Rainfall	15 955	Evaporation	22 358		
	Clean Water Runoff	17 043	Irrigation	8 877		
	Additional Make Up	0				
	Total	32 998	Total	31 235	1 763	Adequate

8. WATER BALANCE RESULTS – PHASE 4 – 11

Phase 4 – 11 for Kusile operations will take place over the remaining 60 Years (from year 18 – end of operations) and during this phase, PCD 5 (PCD 5 and PCD 6) Complex will have been constructed to add additional buffer storage capacity to the dirty water containment on site. Once this new complex is constructed there will be ten (10) containment facilities, seven of them will be PCDs and the remaining three will be the CWDs. Dirty runoff from the ADF will be conveyed to PCD 1 A, PCD 2, Road PCD, PCD 3 Complex and PCD 5 Complex. PCD 1 and PCD 2 will supply water for dust suppression purposes. PCD 5 Complex will flow into PCD 3 Complex when its capacity is greater than 80%; and any excess water from PCD 3 Complex will overflow into PCD 2. The CWDs will receive water via rainfall, runoff and make-up water and this water will be used for irrigation purposes.

8.1 Phase 4 – 11 Water Balance Results

8.1.1 PCD 1 A

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 4 - 11.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF and Conveyor; and
- Make up water for Dust Suppression.

Outflows from the PCD include:

- Evaporation;
- Excess water transferred to PCD 5 Complex (PCD 6); and
- Water supply to dust suppression.

The PCD volume probability result following the 250 realizations (from year 18 – end of operations) in the GoldSim software can be seen in Figure 30. The PCD on average utilises approximately 12% of its capacity (first 9 phases), which is due to assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time and the additional dirty water runoff that the PCD will contain.

There are instances where the PCD would reach FSL, however, the transfer of dirty water prevents PCD 1 A from overflowing more than once in 50 years. During the latter phases (Phase 10 – 11) in simulations it is noted that there will be a significant increase in storage utilisation by approximately 52% of PCD 1 A's volumetric capacity. During these phases, the dust suppression areas will be reduced to half of the ADF footprint (meaning, less abstraction for dust suppression), this results in a reduced dust suppression rate causing the increase in storage at PCD 1 A.

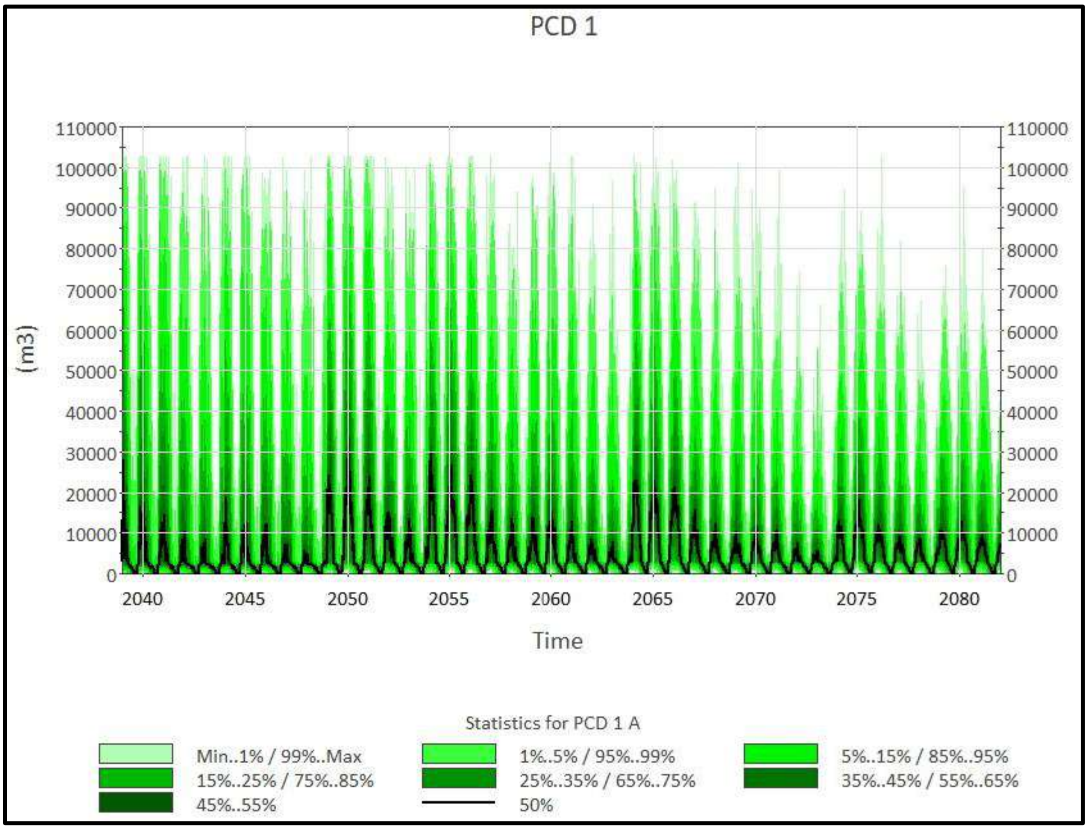


Figure 30: PCD 1 A Volume Probabilities (Phase 4 - 11)

8.1.2 PCD 2

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 4 - 11.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF;
- Top up water from PCD 3 Complex;
- Excess water from PCD 3 Complex; and
- Make up water for Dust Suppression.

Outflows from the PCD include:

- Evaporation;
- Water supply to dust suppression.

The PCD volume probability result following the 250 realizations (from year 18 – end of operations) in the GoldSim software can be seen in Figure 31. The PCD on average utilises approximately 12% of its capacity (first 9 phases), which is due to the excess water received from Road PCD and the assumption that an increase in silt build up will result in a loss of roughly 5% capacity in the PCD over time.

During the latter phases (Phase 10 – 11) in simulations it is noted that there will be an increase in storage utilisation by approximately 18% of PCD 2’s volumetric capacity. During these phases, the dust suppression areas will be reduced to half of the ADF footprint (meaning, less abstraction for dust suppression), this results in a reduced dust suppression rate causing the increase in storage.

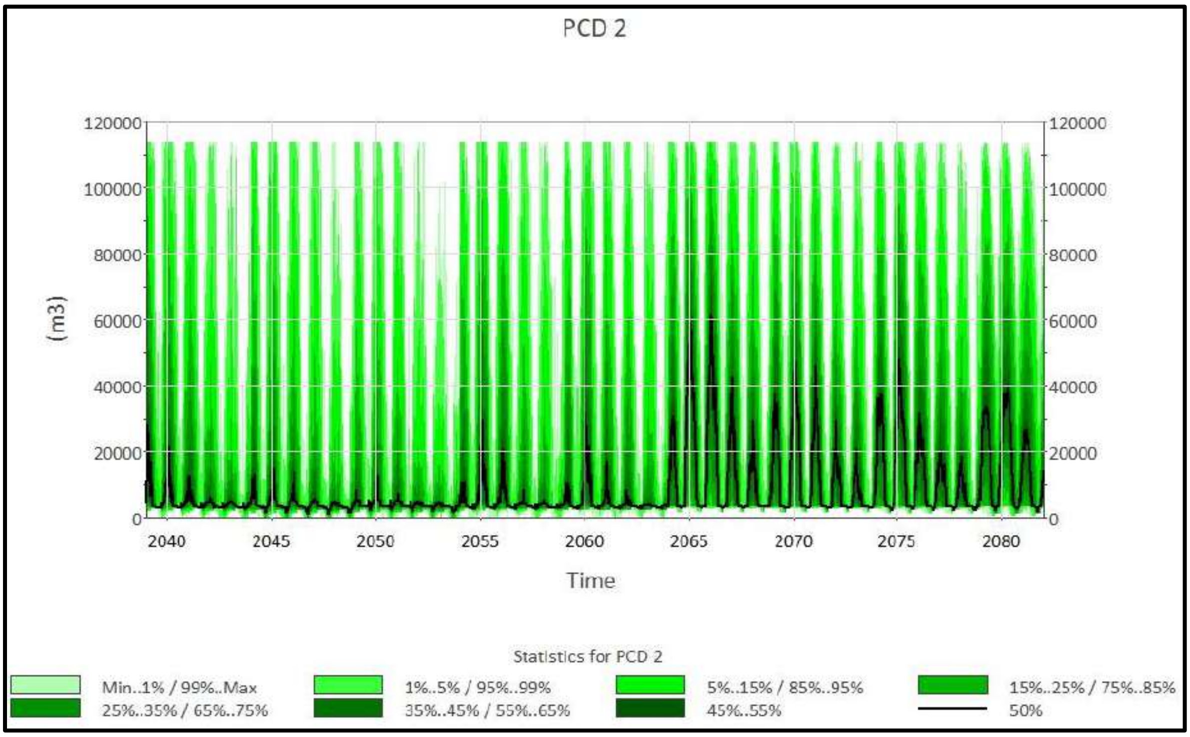


Figure 31: PCD 2 Volume Probabilities (Phase 4 - 11)

8.1.3 *Road PCD*

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 4 - 11.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint; and
- Dirty water runoff from ADF.

Outflows from the PCD include:

- Evaporation; and
- Excess water transferred to PCD 2.

The PCD volume probability result following the 250 realizations (from year 18 – end of operations) in the GoldSim software can be seen in Figure 32. The PCD on average utilises approximately 54% of its capacity. To prevent the PCD overflowing in winter months, it transfers excess water to PCD 2.

Figure 32: Road PCD Volume Probabilities (Phase 4 - 11)

8.1.4 PCD 3 Complex

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 4 - 11.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF; and
- Dirty water transfers from PCD 5 Complex (PCD 5);

Outflows from the PCD include:

- Evaporation;
- Top up water transferred to PCD 2; and

- Dirty water transfers into PCD 2.

The PCD volume probability result following the 250 realizations (from year 18 – end of operations) in the GoldSim software can be seen in Figure 33. The PCD on average utilises approximately 57% of its capacity (first 9 phases). The PCD receives transfers from PCD 5 Complex-thus the large storage utilised. The reduction in capacity utilised from phase 3 is the result of transfers from PCD 1A being directed to PCD 5 Complex in these phases. Water is stored in PCD 3 Complex, and this stored water will supply make up water to PCD 2 during the dry winter months. During storm events overflows will be conveyed into PCD 2.

From phase 10 until the end of simulations, the PCD 3 Complex's average volume increases to 77%. The reduced abstraction rates for dust suppression means that there's fewer outflows from the system.

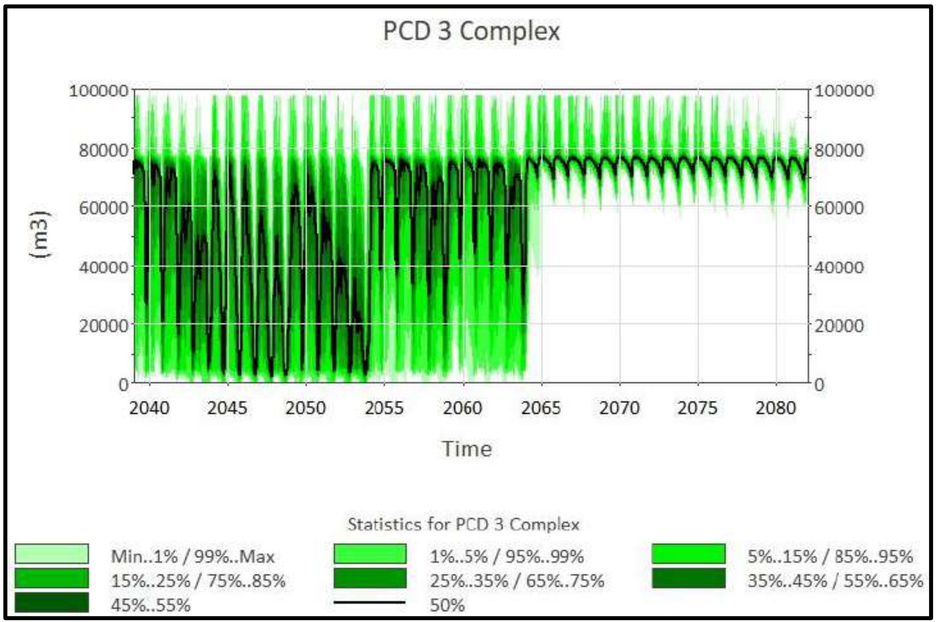


Figure 33: PCD 3 Complex Volume Probabilities (Phase 4 - 11)

8.1.5 PCD 5 Complex

The PCD was modelled in the GoldSim software to replicate likely operations on site for Phase 4 - 11.

Inflows into the PCD include:

- Direct rainfall onto the PCD footprint;
- Dirty water runoff from ADF; and
- Excess water from PCD 1.

Outflows from the PCD include:

- Evaporation;
- Top up water transferred to PCD 2; and
- Dirty water transfers into PCD 3 Complex (PCD 4).

The PCD volume probability result following the 250 realizations (from year 18 – end of operations) in the GoldSim software can be seen in Figure 34. The PCD on average utilises approximately 41% of its capacity (first 9 phases). The PCD receives transfers from PCD 1 A-thus the large storage utilised. This PCD maintains an average of below 70% because it overflows to PCD 2 when its capacity exceeds 70%. Water is stored in PCD 5 Complex, and this stored water will supply make up water to PCD 2 during the dry winter months. During storm events overflows will be conveyed into PCD 3 Complex.

From phase 10 until the end of simulations, the PCD 5 Complex's average volume increases to 66%. The reduced abstraction rates for dust suppression means that there's fewer outflows from the system.

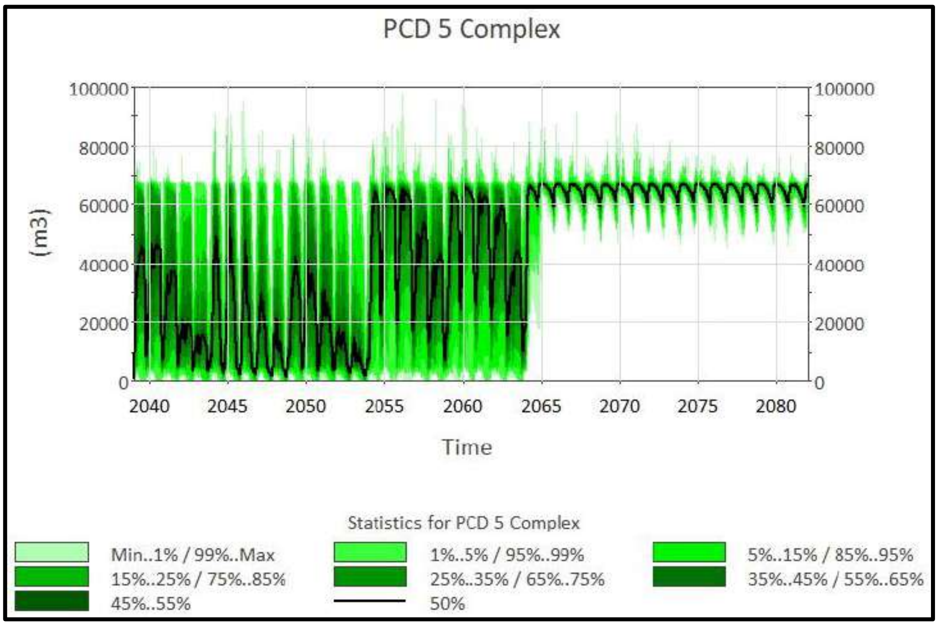


Figure 34: PCD 5 Complex Volume Probabilities (Phase 4 - 11)

8.1.6 CWDs

The CWDs were modelled in the GoldSim software to replicate likely operations on site for Phase 4 - 11.

Inflows into the CWDs (CWD 1, CWD 1 B and CWD 2) include:

- Direct rainfall onto the PCD footprint;
- Clean water runoff from rehabilitated ADF areas; and
- Make up water for irrigation purposes.

Outflows from the CWD include:

- Evaporation; and
- Clean water abstraction for Irrigation supply.

The CWD volume probability result following the 250 realizations (from year 18 – 43) in the GoldSim software for CWD 1 can be seen in Figure 35, CWD 2 in Figure 36 and CWD 1 B in Figure 37. All the CWDs will have instances where excess water is “released to the environment”, this is where the dams reach Full Supply Level (FSL) during Phase 4 – 11 and excess CLEAN water spills / overflows into the environment. During the latter stages of operations, it is noticed that the storage of water increases at all the CWDs, this is due to the increase in the footprint of the rehabilitated areas. These increased areas will generate more clean water runoff when compared to the early phases of operations. During the final phases of operations, the capacity of the CWDs on average are nearly at FSL during the wet summer months.

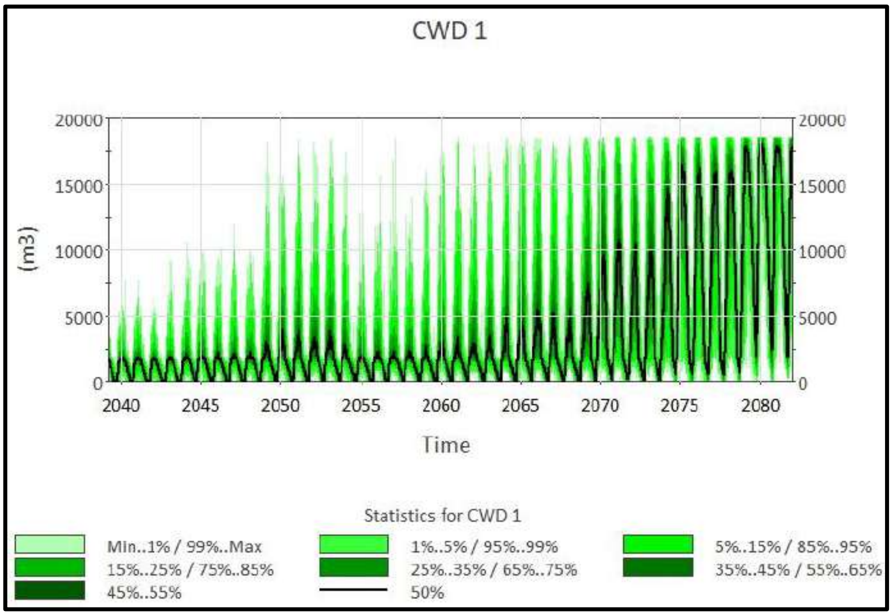


Figure 35: CWD 1 Volume Probabilities (Phase 4 - 11)

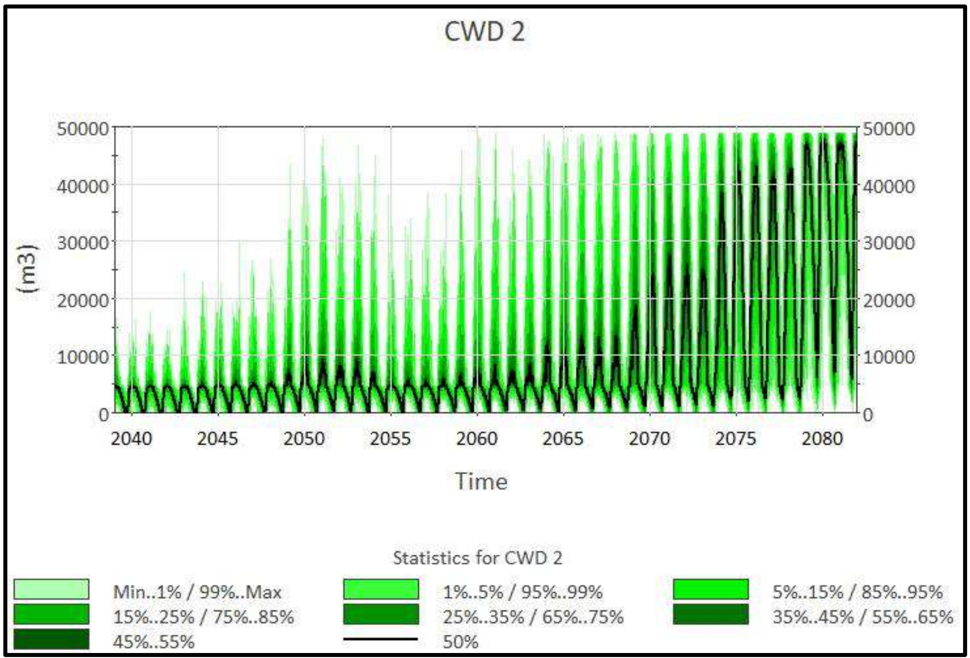


Figure 36: CWD 2 Volume Probabilities (Phase 4 - 11)

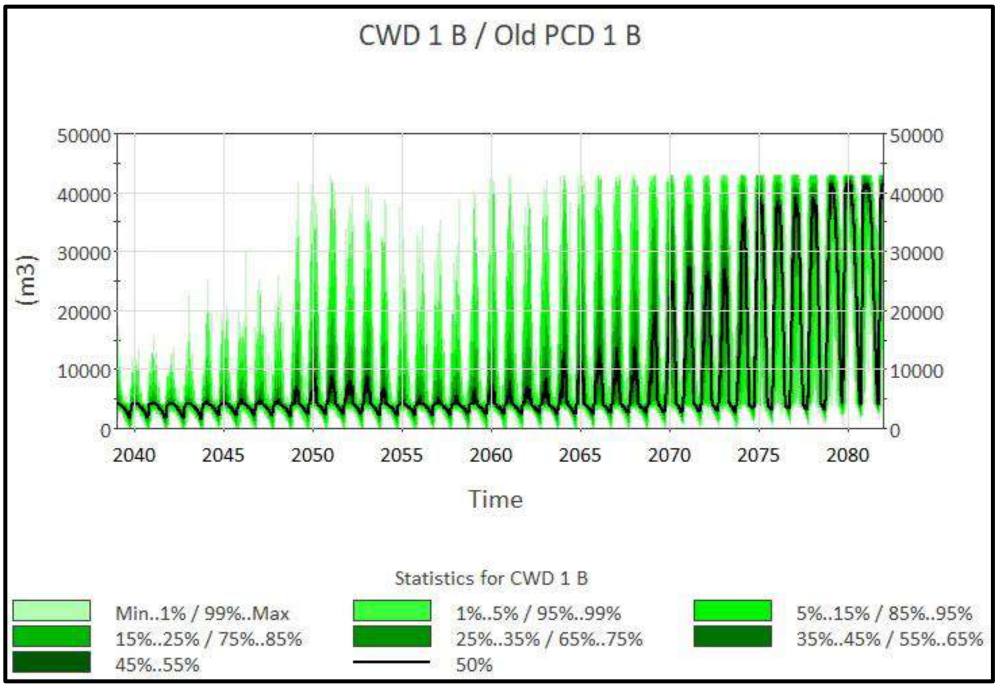


Figure 37: CWD 1B / Old PCD 1 B Volume Probabilities (Phase 4 - 11)

8.2 Water Balance Summary - Phase 4 – 11

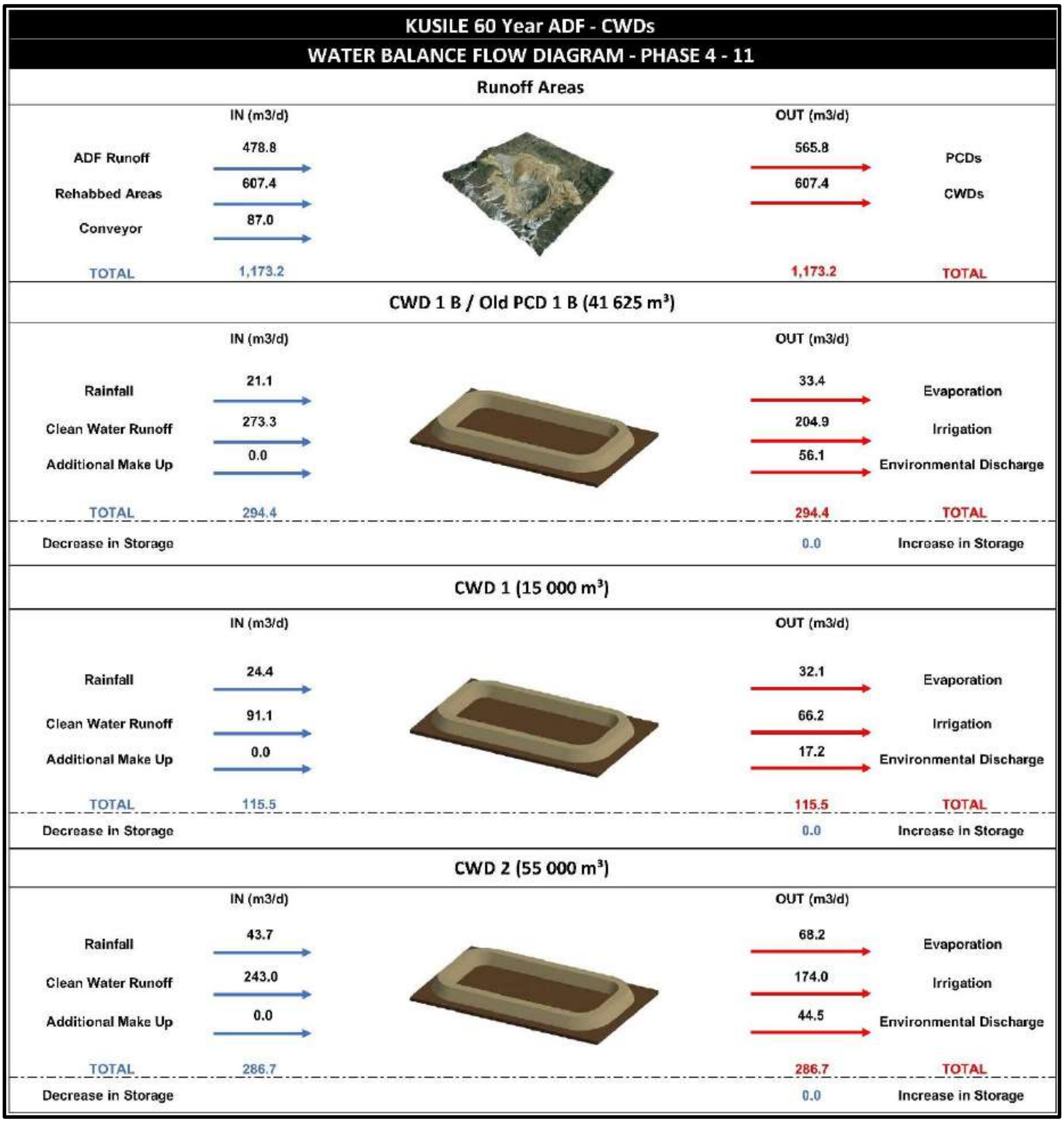
The following section serves as a summary of the simulation results for Phase 4 -11 of Kusile's 60 Year ADF Operations:

- The construction of PCD 5 Complex leads to the increase in storage of dirty water containment on site, the total capacity available on site for dirty water containment is 435,146 m³ from Phase 4 onwards;
- The total mean average daily volume of dirty water contained in the PCDs on site is approximately 37,299 m³ from Phase 4 onwards;
- The total 98th percentile (1:50-year) average daily volume of dirty water contained in the PCDs on site is approximately 279,454 m³ from Phase 4 onwards;
- The total capacity available on site for clean water containment is 110,413 m³ from Phase 4 onwards;
- The total mean average daily volume of clean water contained in the CWDs on site is approximately 22,506 m³ from Phase 4 onwards;
- The total 98th percentile (1:50-year) average daily volume of clean water contained in the CWDs on site is approximately 50,802 m³ from Phase 4 onwards;
- The total mean average daily dirty water available for dust suppression from PCD 1 is approximately 387.9 m³/day (141,595 m³/annum) and approximately 1,765.1 m³/day (644,261 m³/annum) is required for dust suppression purposes during Phase 4 onwards;
- The total mean average daily make-up water that needs to be transferred from the Holding Recycling Dam (HRD) within the power station to PCD 1 for dust suppression is approximately 1,377.2 m³/day (502,686 m³/annum) during Phase 4 onwards;
- The total mean average daily dirty water available for dust suppression from PCD 2 is approximately 451 m³/day (164,613 m³/annum) and approximately 3,042 m³/day (1,110,314 m³/annum) is required for dust suppression purposes during Phase 4 onwards;
- The total mean average daily make-up water that is transferred from PCD 3 Complex and PCD 5 Complex to PCD 2 for dust suppression is approximately 270.4 m³/day (98,686 m³/annum) during Phase 4 onwards;

- The total mean average daily make-up water that needs to be transferred from the HRD within the power station to PCD 2 for dust suppression is approximately 2,591 m³/day (945,785 m³/annum) during Phase 4 onwards;
- The total mean average daily clean water available for irrigation from the CWDs is approximately 172.1 m³/day (62,803 m³/annum) and approximately 1,419.8 m³/day (518,237 m³/annum) is required for irrigation purposes during Phase 4 onwards;
- The total mean average daily make-up water for irrigation is approximately 1247.8 m³/day (455,433 m³/annum) during Phase 4 onwards, approximately 88% of water for irrigation purposes will need to be supplied from an external source;
- The total mean average daily dirty water transferred from PCD 1 and 2 to PCD 3 Complex is approximately 63 m³/day (23,007 m³/annum) during Phase 4 onwards;
- Looking at Figure 35 – Figure 37, an overall increase in clean water is noted at the CWDs. The total mean average daily clean water stored in the CWDs on site is approximately 12.4 m³/day (4,544 m³/annum) during Phase 4 onwards;
- It is clear from Figure 33 and Figure 34, an overall increase in dirty water is noted at the PCDs. The total mean average daily dirty water stored in PCD 3 and 5 Complex is approximately 12.5 m³/day (4,566 m³/annum) during Phase 4 onwards;

The daily water balance flow diagram for CWDs is shown below in Figure 38, the PCDs are shown in Figure 39 and the average annual water usage for Kusile's Phase 4 - 11 operations is listed in

Table 13.



KUSILE 60 Year ADF - CWDs

WATER BALANCE FLOW DIAGRAM - PHASE 4 - 11

Runoff Areas

	IN (m3/d)	OUT (m3/d)	
ADF Runoff	478.8	565.8	PCDs
Rehabbed Areas	607.4	607.4	CWDs
Conveyor	87.0		
TOTAL	1,173.2	1,173.2	TOTAL

CWD 1 B / Old PCD 1 B (41 625 m³)

	IN (m3/d)	OUT (m3/d)	
Rainfall	21.1	33.4	Evaporation
Clean Water Runoff	273.3	204.9	Irrigation
Additional Make Up	0.0	56.1	Environmental Discharge
TOTAL	294.4	294.4	TOTAL
Decrease in Storage		0.0	Increase in Storage

CWD 1 (15 000 m³)

	IN (m3/d)	OUT (m3/d)	
Rainfall	24.4	32.1	Evaporation
Clean Water Runoff	91.1	66.2	Irrigation
Additional Make Up	0.0	17.2	Environmental Discharge
TOTAL	115.5	115.5	TOTAL
Decrease in Storage		0.0	Increase in Storage

CWD 2 (55 000 m³)

	IN (m3/d)	OUT (m3/d)	
Rainfall	43.7	68.2	Evaporation
Clean Water Runoff	243.0	174.0	Irrigation
Additional Make Up	0.0	44.5	Environmental Discharge
TOTAL	286.7	286.7	TOTAL
Decrease in Storage		0.0	Increase in Storage

Figure 38: Kusile ADF Operations - Average Daily Water Usage CWDs (Phase 4 - 11)

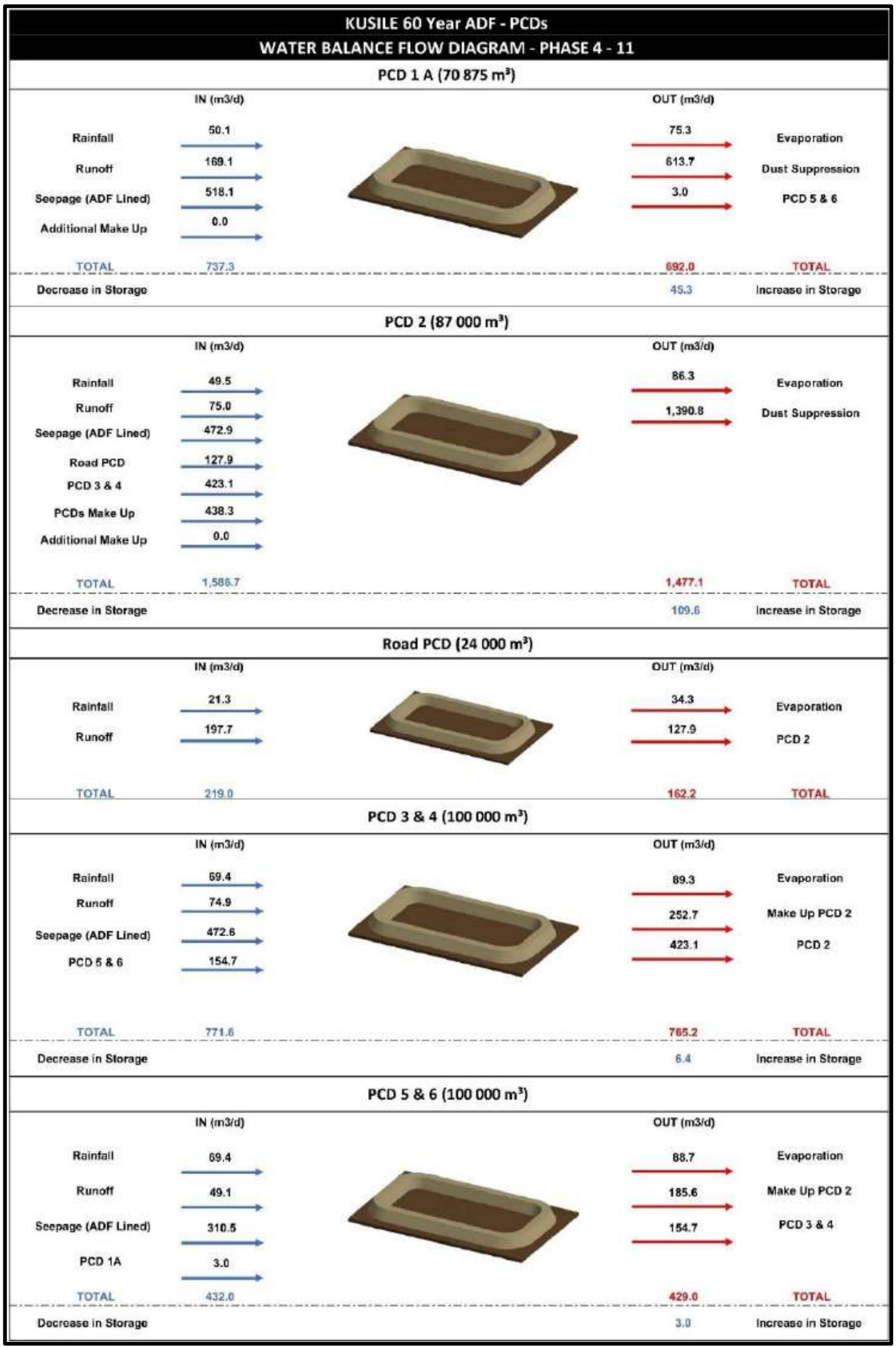


Figure 39: Kusile ADF Operations - Average Daily Water Usage PCDs (Phase 4 -11)

Table 13: Kusile ADF Operations - Average Annual Water Usage during Phase 4 - 11

Kusile 60 Year ADF - Phase 3 Water Balance Flow Diagram (m^3/annum)						
Facility	Water In		Water Out		Balance	Comment
	Water Stream	Quantity (m^3/a)	Water Stream	Quantity (m^3/a)		
Runoff Areas	ADF Runoff	174 740	PCDs	206 498		
	Rehabbed Areas	221 704	CWDs	221 704		
	Conveyor	31 758				
	Total	428 202	Total	428 202	0.00	Adequate
PCD 1 A (70 875 m^3)	Rainfall	18 277	Evaporation	27 489		
	Runoff	61 730	Dust Suppression	224 017		
	Seepage	189 124	PCD 5 & 6	1 102		ADF Lined
	Additional Make Up	0				HRD Make Up
	Total	269 132	Total	252 608	16 524	Adequate
CWD 1 B / Old PCD 1 B (41 625 m^3)	Rainfall	7 719	Evaporation	12 185		
	Clean Water Runoff	99 767	Irrigation	74 785		
	Additional Make Up	0	Environmental Discharge	19 134		
			Increase in Storage	20 514		

	Total	107 485	Total	86 971	20 514	Adequate
PCD 2 (87 000 m³)	Rainfall	18 057	Evaporation	31 489		
	Runoff	27 374	Dust Suppression	507 641		
	Seepage	172 596	PCD 5 & 6	0		ADF Lined
	PCD 3 & 4	21 183				
	Road PCD	46 691				
	PCDs Make Up	159,987				
	Additional Make Up	0				HRD Make Up
	Total	577 596	Total	539 129	38 467	Adequate
Road PCD	Rainfall	7 778	Evaporation	12 517		
	Runoff	72 146	To PCD 2	46 691		
	Seepage	0				ADF Lined
	Additional Make Up	0				HRD Make Up
	Total Inflow	79 924	Total Out	59 212	20 712	Adequate
PCD 3 & 4 (100 000 m³)	Rainfall	25 341	Evaporation	32 591		
	Runoff	27 323	Make Up PCD 2	92 251		
	Seepage	172 494	To PCD 2	154 437		ADF Lined
	PCD 5 & 6 Overflows	17 531	Increase in Storage	2 268		

	Total	277 889	Total	279 281	0.00	Adequate
CWD 1 (15 000 m³)	Rainfall	8 906	Evaporation	11 729		
	Clean Water Runoff	33 256	Irrigation	24 177		
	Additional Make Up	0	Environmental Discharge	5 453		
			Increase in Storage	6 256		
	Total	42 162	Total	35 906	6 256	Adequate
CWD 2 (55 000 m³)	Rainfall	15 945	Evaporation	24 883		
	Clean Water Runoff	88 682	Irrigation	63 495		
	Additional Make Up	0	Environmental Discharge	14 023		
			Increase in Storage	16 249		
	Total	104 627	Total	88,378	16 249	Adequate

9. CONCLUSIONS AND RECOMMENDATIONS

The dynamic water balance model was run over 60 Years and the worst-case scenario will take place during phase 5. The ADF footprint will be at its maximum, this area is over two times larger than the footprint of the ADF during Phase 1. This phase will generate the most runoff from the ADF and it will also require vast amounts of water for dust suppression. A maximum rate of 6,855 m³/day will need to be supplied from PCD 2 and 4,570 m³/day from PCD 1A for dust suppression. Should both PCDs have no water during this phase, 11,435 m³/day of make-up water will need to be supplied from the Holding Recycling Dam (HRD) or clean water from the raw water make-up line, both which are connected to the PCDs. Over the 43-year operations, the PCDs on site, on average will only supply approximately 43% of the required dust suppression while the remaining 57% needs to be supplied from the HRD or from the clean water make-up line.

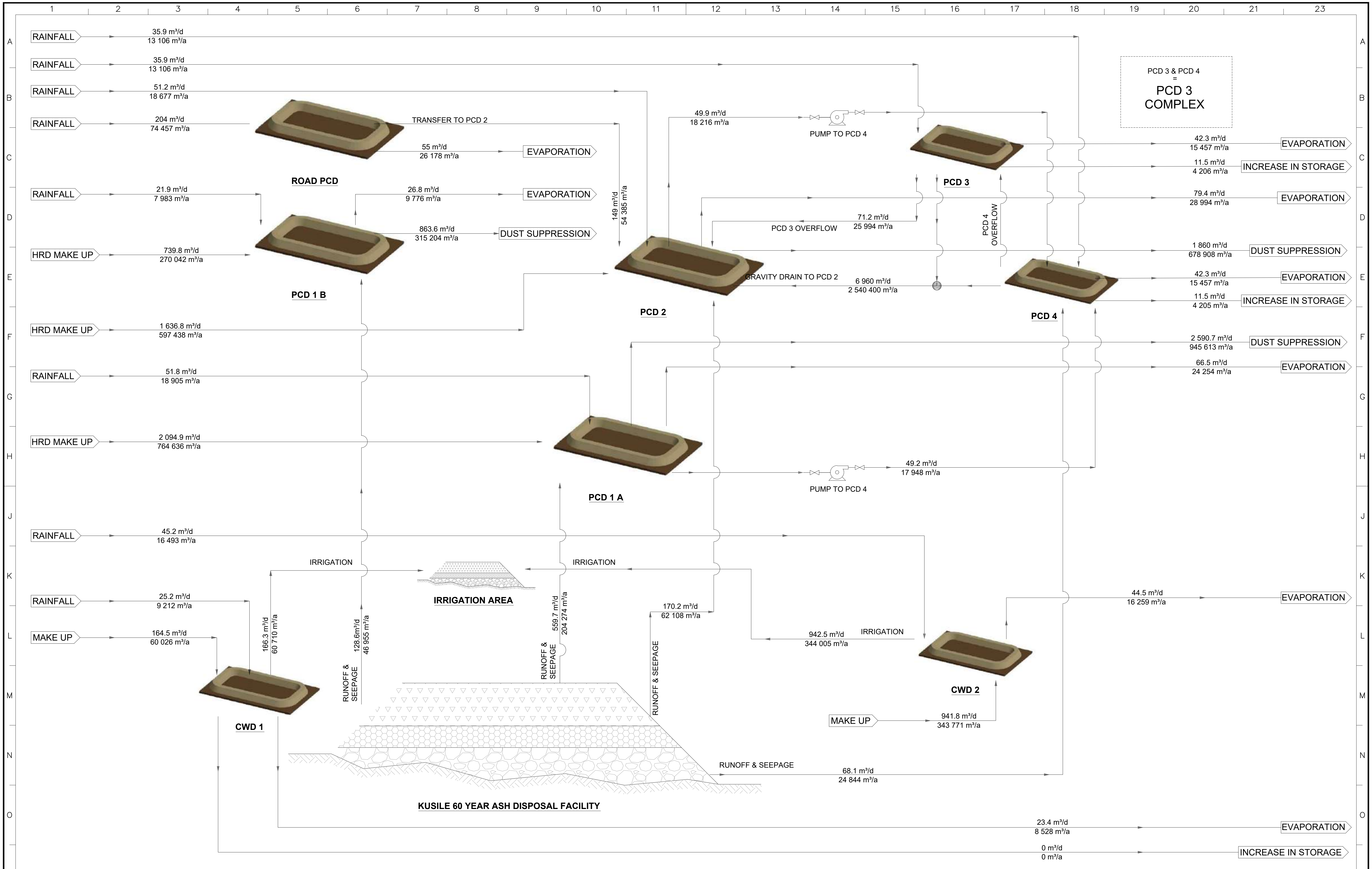
During the early phases (Phase 2 – 8) of the operations approximately 90 – 99% of make-up water is required for irrigation purposes. This water needs to be clean water and water from the HRD cannot be used for irrigation purposes. Water from the CWDs alone will not be sufficient to meet the irrigation requirements and make-up water will be supplied by the clean raw water make-up lines connected to the CWDs.

To improve the accuracy of the water balance model, the client is advised to install flow meters and monitor the flow meter readings to obtain accurate flow rates from the water users / suppliers on site (PCDs / Sprinklers) during the initial phases of operation of the ADF. This accurate flow meter data could then be incorporated into the model in the future, resulting in a more accurate water balance model that could advise potential changes in the operational philosophy and or the construction of PCDs and CWDs.

10. REFERENCES

- Blight, G. E. and Kreuiter, A. (2000). Disposal of industrial waste liquids by evaporation and capillary storage in waste dumps.
- Boughton, W. (2004). The Australian water balance model. Environmental Modelling & Software, (pp. 943-956).
- Boughton, W. C. (1999). A daily rainfall generating model for water yield and flood studies. CRC for Catchment Hydrology.
- Davies, M.P. and Rice, S., 2001. An alternative to conventional tailing management–“dry stack” filtered tailings. (pp. 411-420). CRC Press.
- Lynch, S. (2003). The development of a raster database of annual, monthly and daily rainfall for Southern Africa, (K5/1156)
- Eskom. (2020). Low pressure services Basic Design Report for Kusile Power Station 60 Year Ash Disposal Facility.
- Zitholele Consulting JV (ZCJV), (2018). Kusile 60Yr ADF Detail Design Report. Report No: 15167-45-Rep-004 Rev 6.

11. APPENDIX A: KUSILE 60 YEAR ADF - PROCESS FLOW DIAGRAMS



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60YR ADF SCOPE LEGEND

NAME	LABEL

KKS | PBS: 00UEQ01

ASH DISPOSAL FACILITY

NOTES:
COORDINATE SYSTEM : LO29 WGS-84

Sign:
K.P.MATULOVICH
Pr. Eng. 20190729
Date: 27-02-2024

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KUSILE POWER STATION
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PHASE 2
WATER BALANCE DIAGRAM

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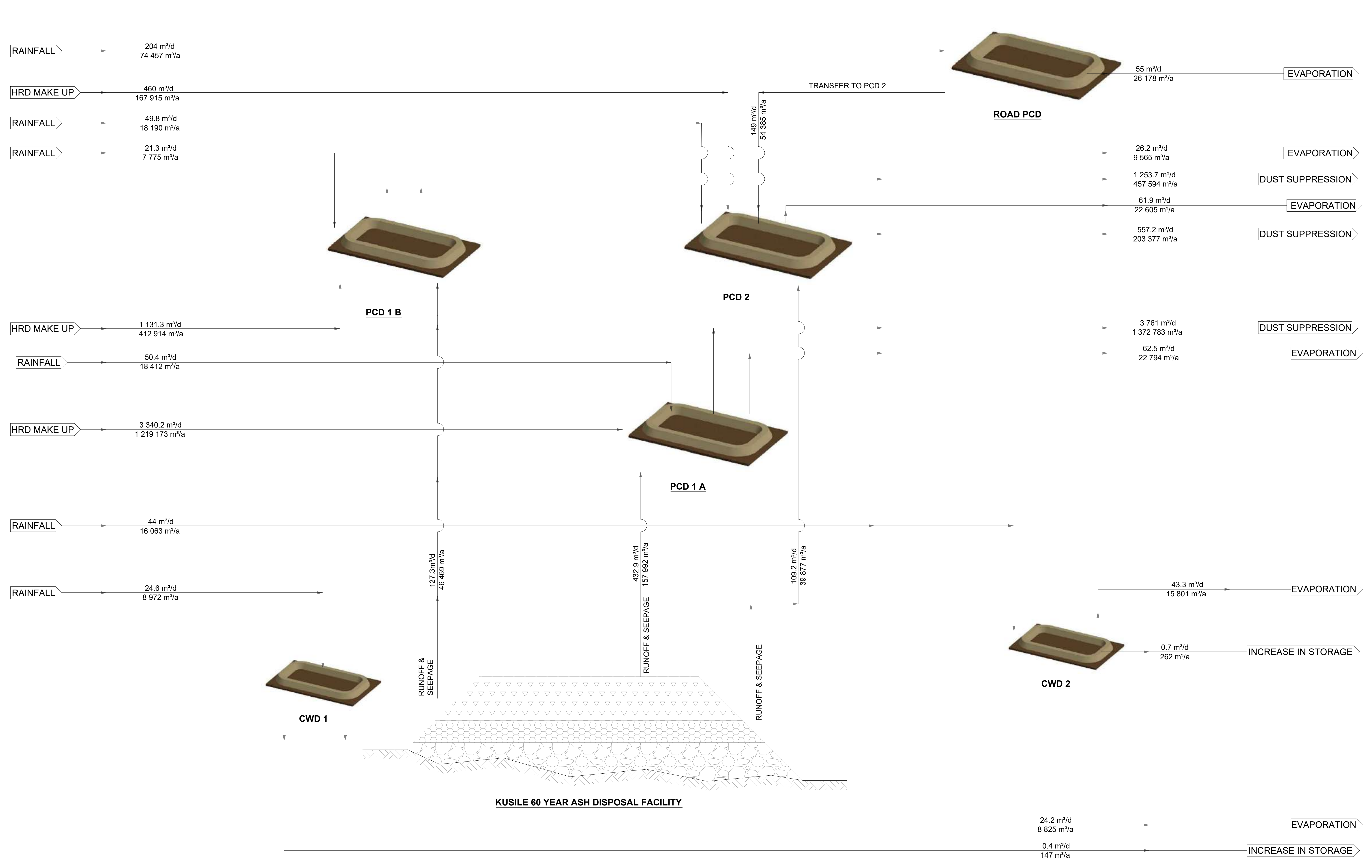

60YR ADF SCOPE LEGEND

NAME	LABEL

KKS PBS: 00UEQ01

ASH DISPOSAL FACILITY

NOTES:
COORDINATE SYSTEM : LO29 WGS-84

Sign:
K.P.MATULOVICH
Pr. Eng. 20190729
Date: 27-02-2024

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REG. No.
2002/015527/06

KUSILE POWER STATION
ASH DISPOSAL FACILITY
PHASE 1
WATER BALANCE DIAGRAM

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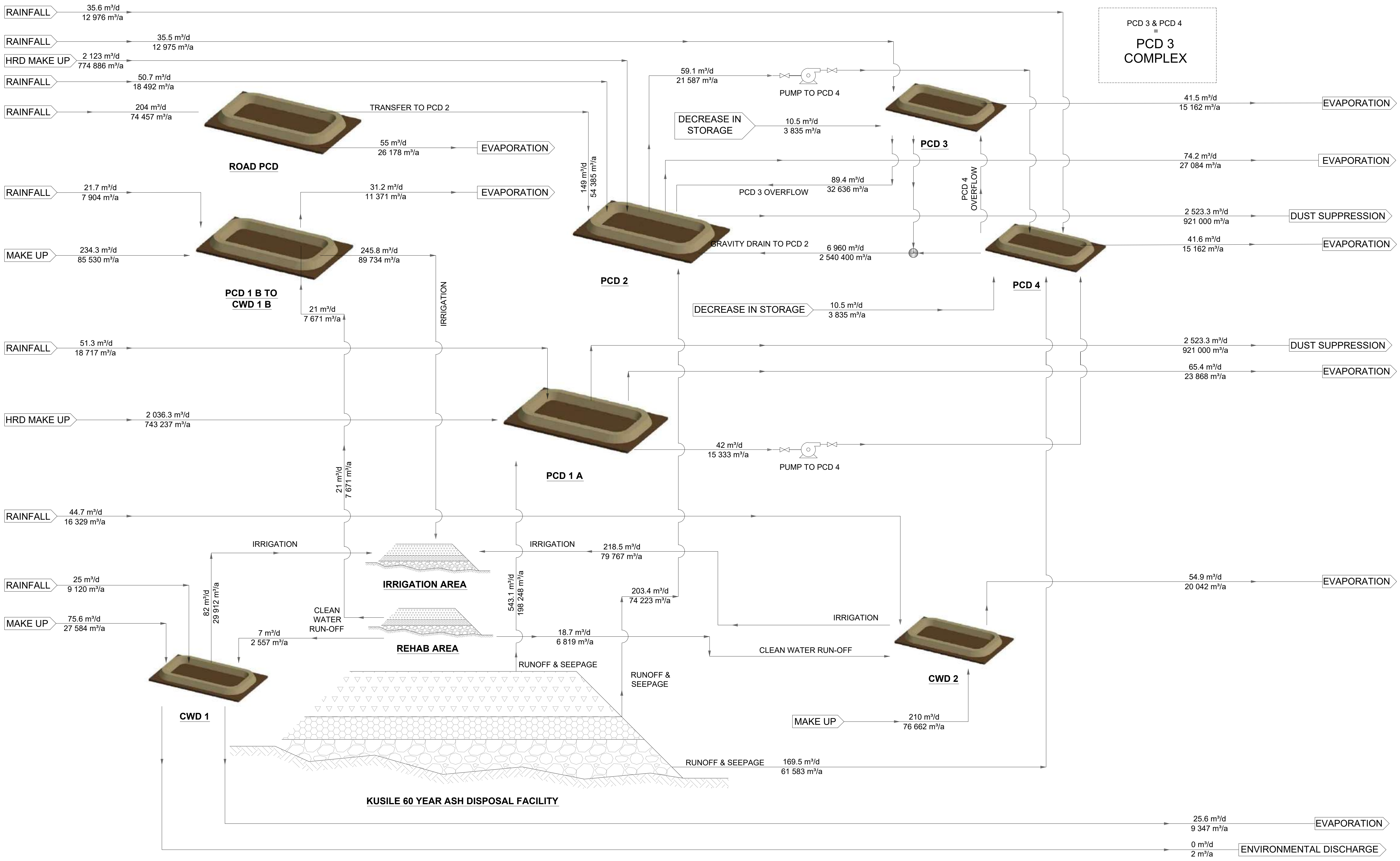

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NAME	LABEL	ASH DISPOSAL FACILITY		

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ECSA Pr.Eng 20190729
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KUSILE POWER STATION
ASH DISPOSAL FACILITY
PHASE 3
WATER BALANCE

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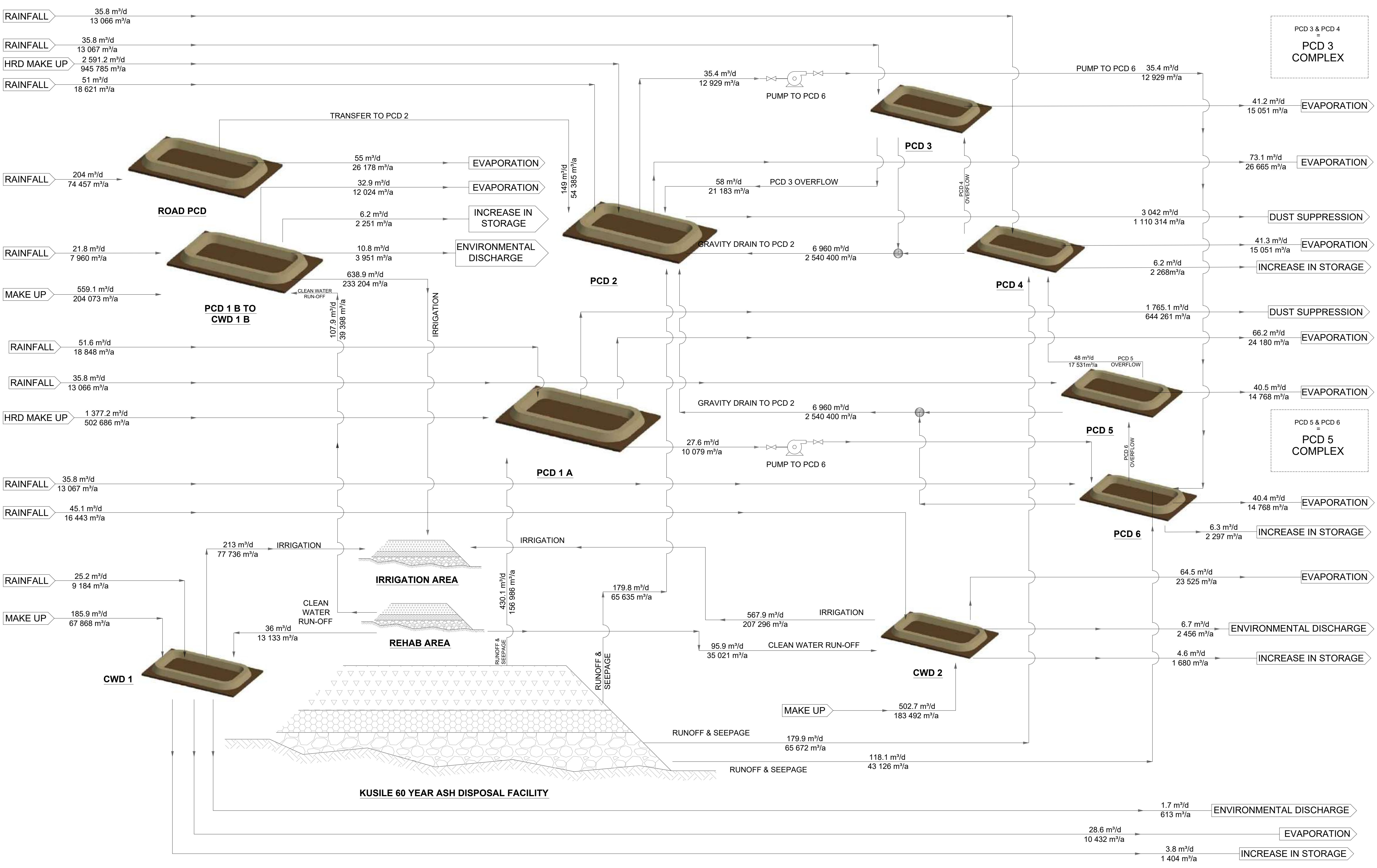

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60YR ADF SCOPE LEGEND

NAME	LABEL

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ASH DISPOSAL FACILITY

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KP Matulovich
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KUSILE POWER STATION
ASH DISPOSAL FACILITY
PHASE 4–12
WATER BALANCE

Eskom

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