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Project Number
RI301-00825/01 REV B

MATIMBA ASH DUMP CONTINUOUS ASHING - BASIC & DETAILED DESIGN

DETAILED DESIGN REPORT

Rev	Description	Date
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Knight Piésold CONSULTING

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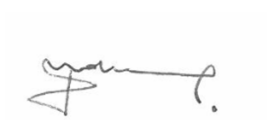
Prepared by

Jannie Viljoen, Pr Eng
Senior Civil Engineer

Reviewed by:



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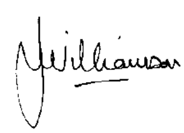

JRG (Rob) Williamson Pr Eng.
Technical Consultant

EXECUTIVE SUMMARY

General

The Umbani Joint Venture has been appointed by Eskom Holdings Limited, under Task Order (TO) #966 of 5 March 2019, to carry out the detailed design of the Matimba ash dump at the Matimba Power Station.

The current Matimba ash disposal facility (ADF) has an exemption area with a planned ash storage life of 5 years, until February 2022. This 5-year exemption capacity is nearing exhaustion and the next 4-year implementation phase is required as soon as possible since the standby system has reached the 5-year exemption line. From the basic design report, the radial ashing option was chosen as the preferred option to develop into detailed design. Radial Shifts on both the lower and top stacks of the ash dump extension apply. Stacking on top of the current facility and on the upper stack of the extension facility will be a combination of both parallel and radial stacking.

The radial ashing layout, scope and definition are presented under **Figure A** and **Figure B.**

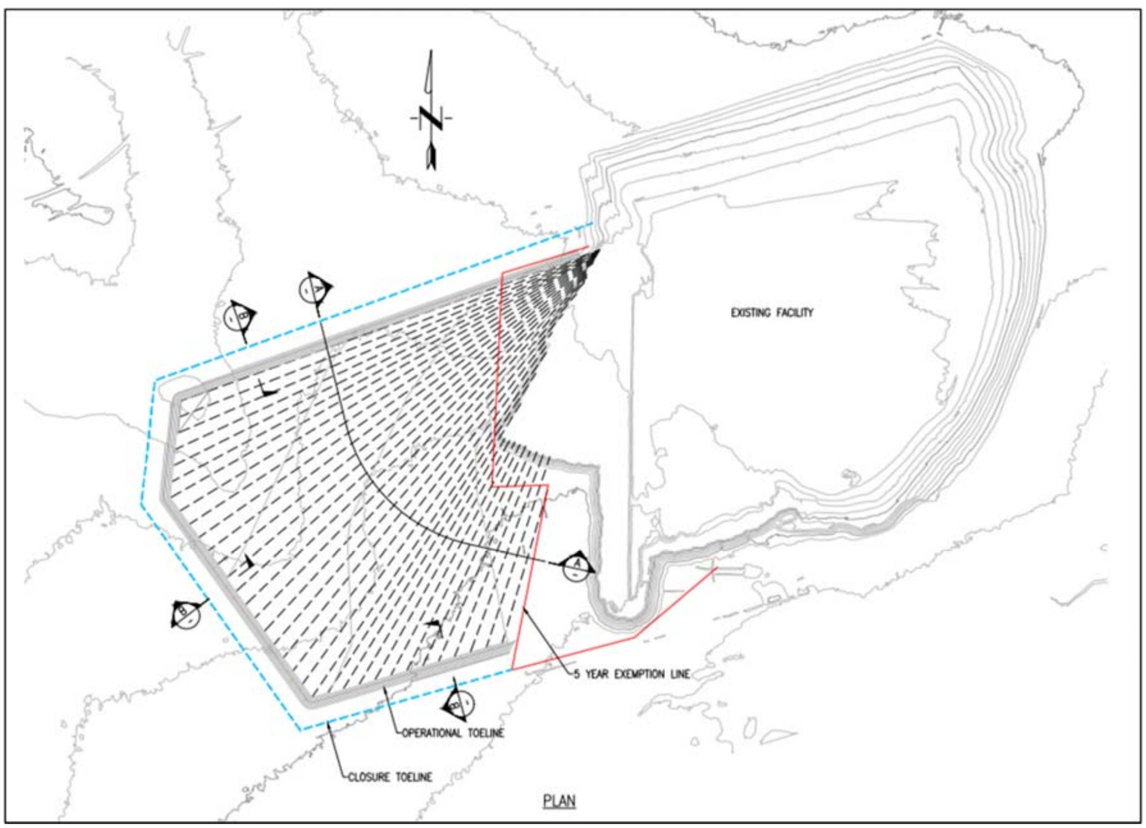

Figure A: Typical ash dump layout and scope definition

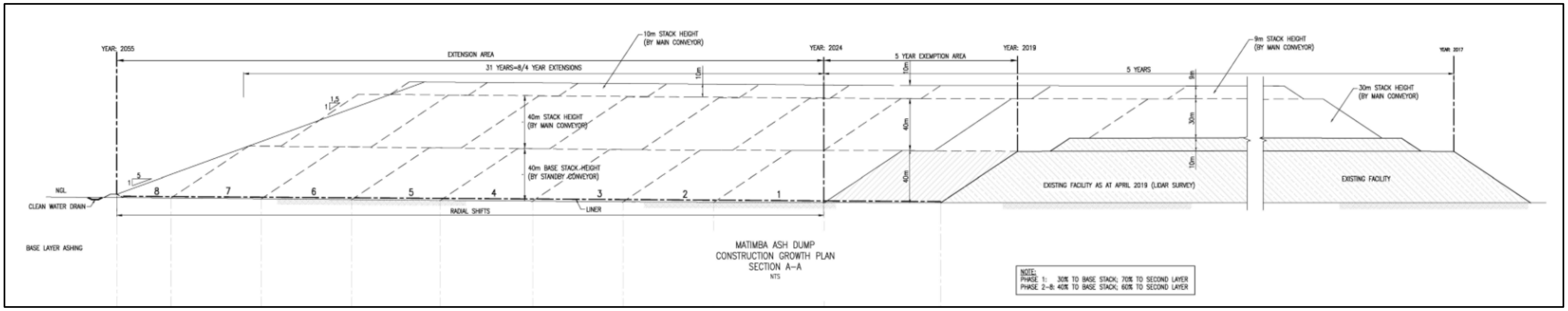

Figure B: Long section through ash dump

The detailed design is based on Geotechnical investigations performed during April 2019 and LIDAR Survey executed during March 2019.

Slope Stability

Slope stability assessments were done to ensure that the design intermediate 30^{o} slope of the ash dam in combination with the proposed base HDPE barrier system will have an operating factor of safety 1.3 or higher during operation. A factor of safety of 1.5 is desired for long term closure. With a rehabilitation plan in place and with proper operational practices the 1.3 factor of safety will be sufficient during operation and will increase to the desired 1.5 once the slopes have been reshaped and rehabilitated at a 1V to 5H final slope.

Liner Design

A 2.0 mm single textured HDPE liner will be installed according to the specifications of GRI GM13 and as a replacement for the standard Class C clay layer, a Geosynthetic Clay Liner (GCL) is proposed as it is currently the industry standard and proven to have a performance similar or better than the suggested clay layer. A 300 mm compacted clay liner has an expected permeability of 10^{-9} m/s, an equivalent GCL to be used should have a permeability of 4.6×10^{-11} m/s or better. A leak detection system consisting of 160 mm diameter slotted HDPE pipes will be installed below the GCL layer in the prepared foundation layer.

Geometric Design

During the basic design two geometric options were assessed in order to determine the viability of utilising either a parallel deposition method or a radial deposition method for the lower stack of the extension. The outcome of this assessment had resulted in the option of utilising the radial deposition for the lower stack of the ash dump extension. A combination of radial and parallel stacking will apply for the upper stack of the ash dump extension. This option, referred to as option 2 in the basic design, serves the basis of this ash deposition method used for this report.

This report focuses on the geometry and growth plan for the 4- and 60-year life of facility of the ash dump extension. The purpose of this report is to provide a detailed design with regards to the Ash dump geometry and growth plan.

The design of the geometry of the Ash dump facility extension is based on data provided by Eskom and testing and analysis conducted by KP.

A LIDAR aerial survey was conducted in April 2019 for the entire Ash Dump facility. The purpose of the lidar survey was to indicate the current available deposition area and ground contours of the facility and extension area. The survey also indicated the current position of the conveyors that are depositing the ash on the facility. The operators of the Ash dump are currently utilising both a Main and a standby conveyor system in order to deposit the Ash on to the facility.

The existing ash dump has an exemption area with a planned ash storage life of 5 years from the 10th February 2017. This area is estimated to be fully utilised by February 2022.

The **Figure C** below indicates the position of the Main and Standby conveyor systems as per the Lidar survey that was conducted in April 2019.

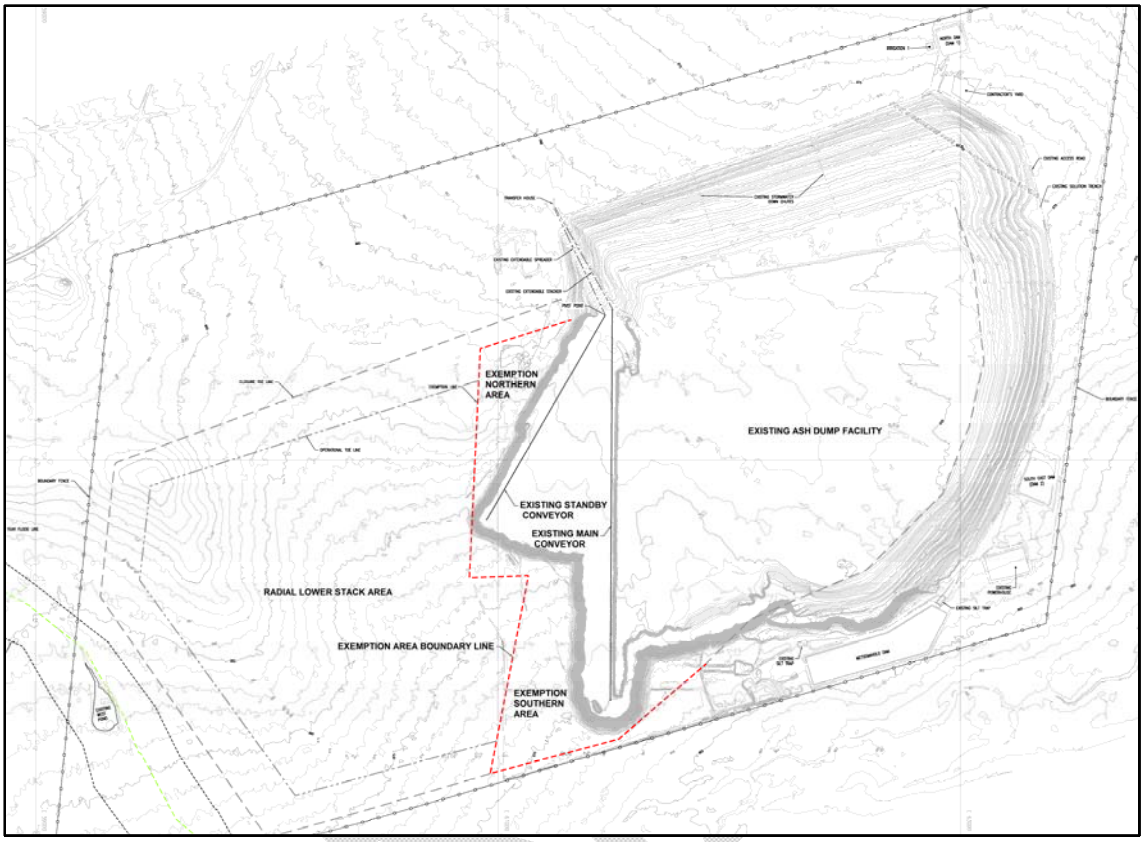

Figure C: Conveyor systems as on 14 April 2019

The Northern exemption area has a storage capacity of 3,782,237 m^3, which, at a 100% deposition rate would provide a storage life of approximately seven (7) months. Reducing the deposition rate to 30% will increase the storage life to approximately 23 months.

The Southern exemption area has a storage capacity of 8 046 774 m^3, which, at a 100% deposition rate would provide a storage life of approximately 15 months. Reducing the deposition rate to 30% will increase the storage life to approximately 50 months.

The remaining 36-year life of the facility has been split into nine (9) individual phases i.e. phase 0 to phase 8, with each phase consisting of a period of four (4) years. Phase 0 has been defined as the phase in which deposition occurs in the Northern and Southern exemption areas by means of the Standby System and in a portion of the existing ash dump (piggybacking) by means of the Main System.

Phases 1 to 8 have been defined as the phases where deposition occurs on the lined area post the exemption line by means of the Standby System and on the existing ash dump (piggybacking) by means of the Main System. Deposition will occur in the phases as defined above with each phase being lined prior to deposition taking place for that phase. A proposed timeframe for the construction of each lined phase is provided in section 6.4 of this report. The detailed design covers the cost analysis for each phase.

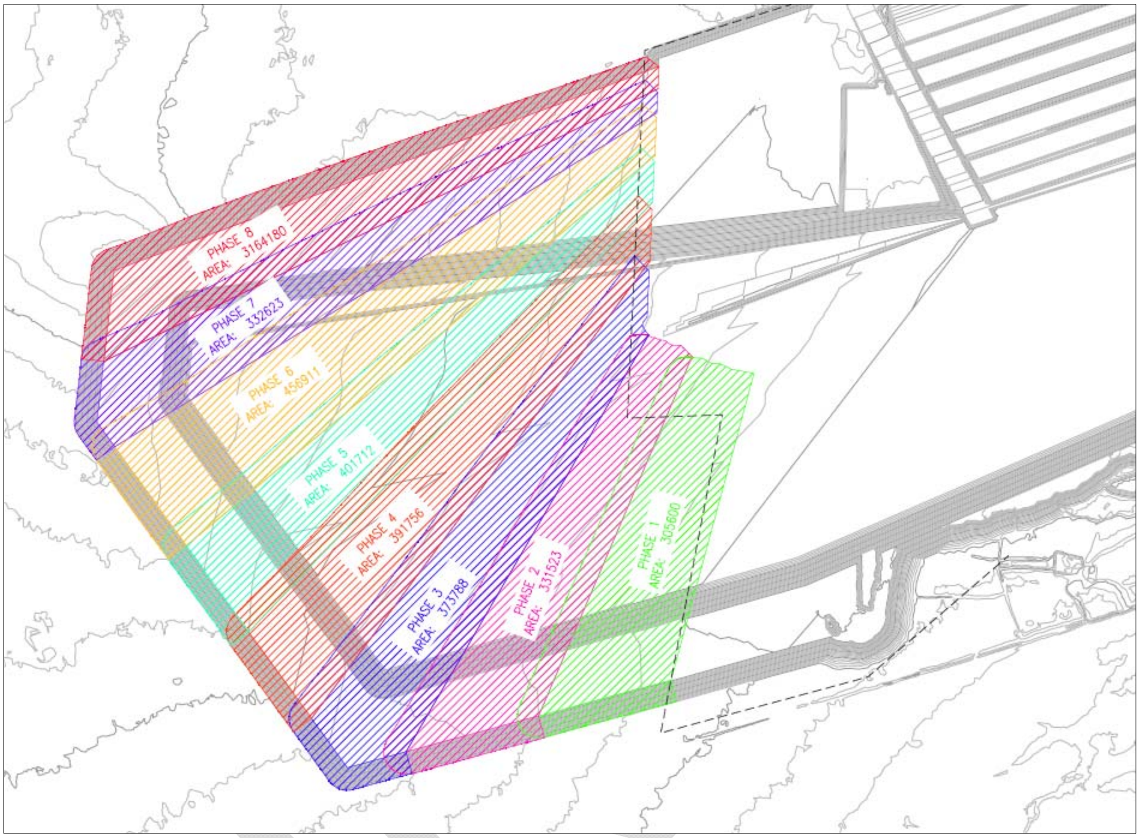

Figure D: Liner phases

Hydrology

The Matimba Ash Dump falls within the A42J quaternary catchment, the Mean Annual Precipitation (MAP) was found to be 428 mm. The Daily Rainfall Data Extraction Utility (ICFR, 2012) indicates a MAP for the nearest reliable rainfall station (providing good quality rain data) located at the Lephalale (Ellisras) Police Station of 455mm (SAWS 0674400_W). The Symons Pan, Mean Annual Evaporation (MAE) of 1949mm, reported in WR2012 (WRC, 2012).

Holding Dams

A 55-year rainfall record (SAWS 0674400_W) was used to model the proposed new North and South RWDs. The dams were sized iteratively to meet the GN 704 guidelines as well the required water re-use. The analysis determined that a dam capacity of 60 000 m^3 for the North Return Water Dam and 80 000 m^3 for the South Return Water Dam, is sufficient to meet the spill criteria, with a maximum pumping rate of 2,000 m^3/d and 6,000 m^3/d from each of the dams respectively.

Topsoil Management Plan

A topsoil availability assessment was done to evaluate if there is an inadequate volume of topsoil for rehabilitation. The assessment confirmed that there will be a nett volume deficit of 645,930 m^3 of topsoil for rehabilitation from the ash dump extension floor.

Stormwater Management Plan

Clean Stormwater

The proposed progressive up-slope radial stacking will result in clean storm water run-off running towards the active ashing face. For Phase 1 and 2, external clean stormwater run-off will be diverted using a temporary concrete lined trapezoidal gravity channel.

For phases 3 to 8, a buried pipe decant system will be used as the clean run-off is trapped in the valley-depression against the Phase West wall. The valley-depression makes it impossible to divert the clean run-off using gravity as run-off collects in the depression. A temporary concrete lined channel traveling parallel to the Phase toe lines will be used to collect the clean run-off and lead it to a decant pipe inlet for discharge by buried penstock pipeline to the South perimeter drain. The clean water channels will drain to a common low point within the depression zone for each phase.

The clean run off will be decanted using decant inlets outside each phase lined deposition area. For phases 3 to 8, these inlet positions are strategically placed at the proximity of the lowest point in the depression on each phase. The inlets will connect into a buried 650 NB outfall penstock pipe.

The penstock will be constructed in stages. Inlets will be constructed, and the outfall pipe extended every 4-years. The outfall pipe however needs to be constructed up to the first inlet prior to the start of the lining works for Phase 2 as it passes under the lined area.

At closure of the decant facility, the pipe will be plugged and decommissioned. The clean run-off catchment will be further reduced by constructing a gravity diversion trench on the north west edge of the clean water footprint. The trench will divert the run-off in north-east direction essentially halving the clean run-off catchment area. The remaining clean catchment run-off between the trench and the ash dump slopes will be left in the depression to evaporate.

Dirty Stormwater

All dirty stormwater run-off from the ash dump extension will be transferred to the North and South water storage dams by means of gravity channels and pumping.

At the North-west edge of each lined phase, a temporary HDPE lined channel will be used to collect dirty run-off from the active ashing slopes. For the Northern exemption area only, the dirty run-off will flow southwards to the HDPE lined Phase trenches. Since the stacking is up-slope, run-off ponds against the active ashing face in the lined area.

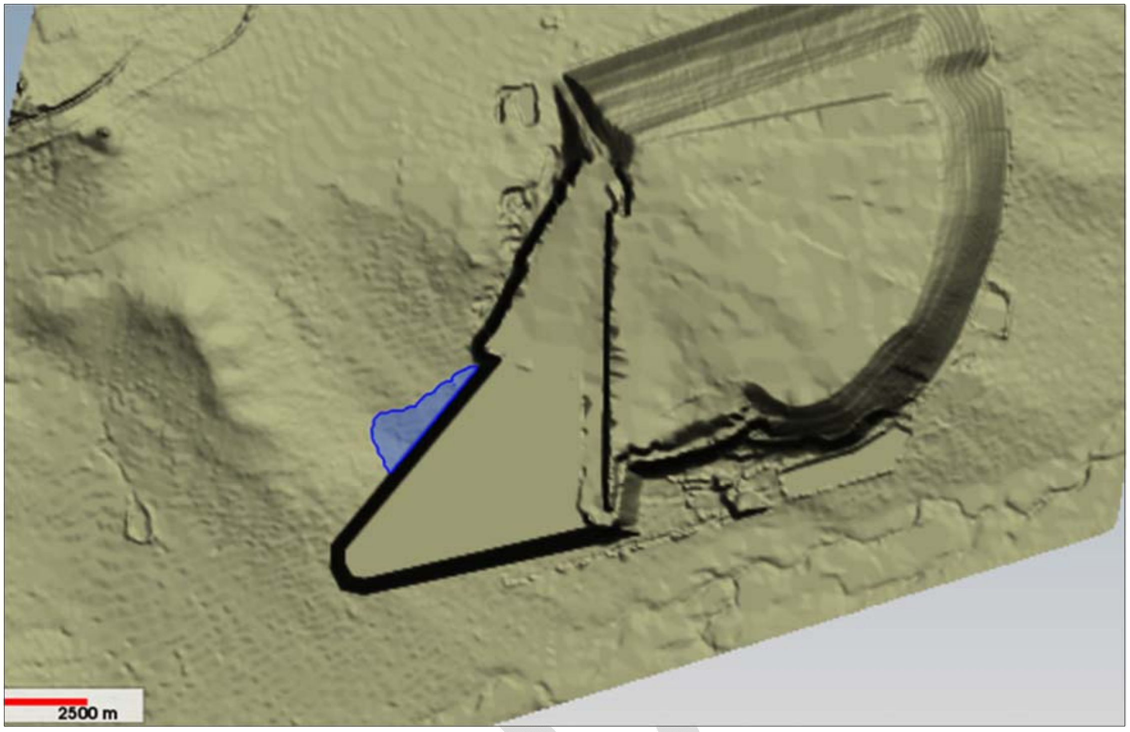

Figure E: Stormwater run-off trapped against advancing ash face

Because of the upstream progression of stacking, the dirty stormwater pool is always at a lower elevation relative to the dirty drain (further aggravated by the lowering of the terrace levels by top-soil stripping and borrow material removal to stockpile for the 300mm protection layer above the liner). A temporary lined trench system will be constructed leading Westwards away from the advancing ash face, to provide a low point near the temporary dirty drain at the West edge of each phase. This will allow lifting of the run-off into the phase edge drain by means of a trailer mounted diesel pump. The trench is designed to allow for the collected stormwater to flow to a single low point were pumping can be facilitated.

A diesel driven; trailer mounted pump unit will be used to lift the dirty run-off into the dirty water drain.

An earth-fill pump platform will be constructed for each phase where the pump can be placed during or after a rainfall event. The dirty run-off collected by the trench will flow into a silt trap then to the south return water dam. During ashing throughout phases 1 to 6 the dirty run-off will be flowing to the South dam. The North dam and dirty drains from phase 5 until closure of facility.

A network of collector trapezoidal drains will be used to collect the run-off from the final top surfaces of the ash dump. The ash dump extension top surface is sloping in an easterly direction and with a raised central crest. The drains are at 300 m spacing and discharge in a south-west direction for the southern half of the extension and north-east for the northern half of the extension.

Construction schedule

The construction of the project will be completed in 8 phases as per the programme presented in **Appendix G**. Each phase will be constructed in 4-year increments.

Cost Estimate

The total estimated cost of works is R1,546,812,175.53 and a detailed cost breakdown is presented in **Appendix F**.

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FIGURES

APPENDICES

ABBREVIATIONS

ADF .... Ash disposal facility
DEA .... Department of Environmental Affairs
DPSH .... Dynamic Probing Super Heavy
DWS .... Department of Water and Sanitation
Eskom .... Eskom Holdings Limited
FOS .... Factor of safety
GCL .... Geosynthetic clay liner
GM .... Grading modulus
MDD .... Maximum dry density
PCD .... Pollution control dam
PI .... Plasticity index
RRD .... Rehabilitation runoff dams
SG .... Specific gravity
TO .... Task order
TP .... Test pit

DRAWINGS

All detailed design drawings are provided in **Appendix A** with a complete drawing list.

1.0 INTRODUCTION

The Umbani Joint Venture has been appointed by Eskom Holdings Limited, under Task Order (TO) #966 of 5 March 2019, to carry out the detailed design of the Matimba ash dump extension at the Matimba Power Station. The ash dump is located approximately 3 km south of the station, on the farm Zwartwater, south of the main road (Nelson Mandela drive) from the town Lephalale to the Power Station as shown on **Figure 1-1**.

The Matimba Power Station dry ash dump was designed in 1985 without a foundation barrier system or underdrainage system. Regulations regarding waste disposal and management have recently changed, during 2013 which require waste classification according to chemical composition and the allowed minimum leachate threshold. Eskom has applied for a 5-year ashing exemption period to allow it to do the necessary designs to comply with minimum requirement and construction of the required works.

This document provides the detailed design information for the continuous ashing at the Matimba Power Station ash dump post the exemption period until 2055. The document contents provide the outcome of the detailed design related activities with references to design and other documents.

A final basic design report was submitted to Eskom on 20 August 2019 and approved. The purpose of the basic design was to establish a system design baseline for all system elements in sufficient detail to procure the detail design, fabrication, construction, hardware production and software coding of the system. The major technical uncertainties or risks are resolved through analysis. The technology readiness level of new components and innovative systems is increased through analyses and / or physical development testing. Ambiguities in the initial system requirements are resolved and the design requirements are validated.

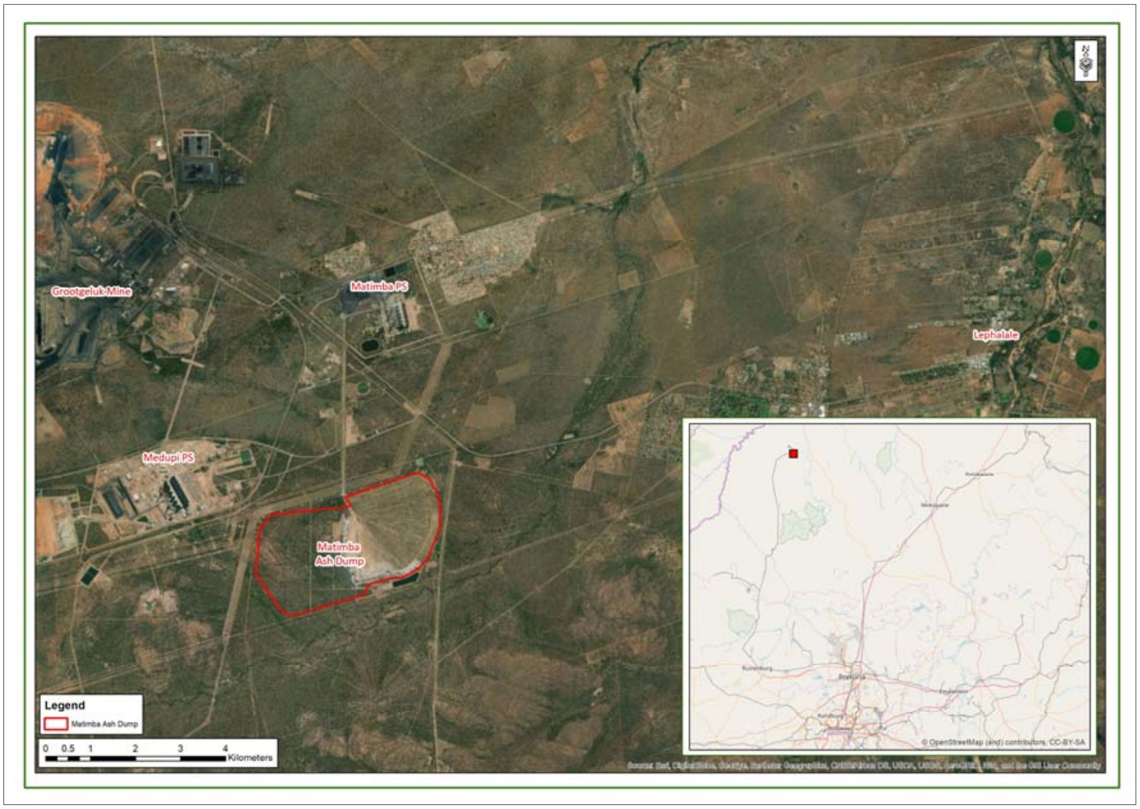


Figure 1-1: Locality Map

1.1 SCOPE OF WORKS

Umbani Joint Venture is responsible for the engineering design and construction drawings for the continuous ashing detailed design at the Matimba Power Station ash dump extension. The following components are defined within the scope of Task Order (TO) #966 for both the basic design and detailed design phase.

Basic design phase:

- A new LIDAR aerial survey;
- Detailed geotechnical investigations;
- Finalise the ash dump and pollution control dam's liner design;
- Final stability analysis;
- Geometric design of the ash disposal facility;
- Review and finalise the sizing, siting and layout of the existing and new pollution control dams;
- Optimizing and finalizing the water balance and clean and dirty water control canals and dams;
- Rehabilitation runoff holding dams;
- Determine the main & standby system growth plan;
- Prepare a topsoil management plan;

- Dust suppression;
- Stormwater management plan;
- Construction duration and cost estimate;
- Arrange meetings with Department of Water and Sanitation (DWS) / Department of Environmental Affairs (DEA);

Detailed Design (Phase 1: 0 to 4 years):

- Detailed area & route ground surveying where required;
- Detailed ash dump liner and drainage design;
- Detailed design of the ash dirty dams and rehabilitation runoff dams;
- Detailed design of the ash dumps clean and dirty water perimeter canals to GN704;
- Detailed design of the initial temporary clean and dirty storm water control drainage works;
- Detailed design of the initial gravel access roads including access to the dams and fence patrol roads;
- Detailed design of the perimeter fencing;
- Works information;
- Specifications;
- Bill of quantities;
- Approved for construction drawings;
- Detailed design report; and
- Detailed design construction duration and cost estimate (priced bill of quantities).

Detailed Design (Phase 2: 4 to 65 years):

- Detailed area & route ground surveying where required.
- Detailed liner and drainage design for the future 4-year areas up to the end of the 65-year ash dump;
- The 4 yearly lining and drainage designs must be prepared as separate packages including works information, drawings, specifications and bill of quantities (priced) for each 4-year area;
- Detailed design of deferred ash dump clean and dirty water perimeter canals including culvert bridges for mobile plant;
- Detailed design of the ash dump ongoing construction clean and dirty water drainage systems including pollution control dams (PCD's), rehabilitation runoff dams (RRD's), final ash dump top surface and side slope stormwater control berms, channels, benching & takedown chutes/pipes with energy dissipaters into the clean and dirty water perimeter canals;
- Detailed design of the ongoing gravel access roads. Drainage culverts must be provided for all weather access;
- Current position to 65-year mechanised ash dump final stacking & geometric construction drawings with profiles for each conveyor position; and
- Operating & maintenance manual.

1.2 DESIGN CRITERIA

The approved design criteria applicable to this project is attached under **Appendix B** of this report.

1.3 BACKGROUND

The current Matimba ash disposal facility (ADF) has an exemption area with a planned ash storage life of 5years, until February 2022. This 5-year exemption capacity is nearing exhaustion and the next 4-year implementation phase is required as soon as possible since the standby system has reached the 5-year exemption line.

To gain time to allow the future planned first 4-year extension to be constructed, ashing on top of the existing ADF is proposed in this 2018 concept design.

Various ashing scenarios were assessed in order to optimize the utilization of both the existing and the future ash dump areas.

From the two options that were addressed in the basic design report, radial ashing and parallel ashing, the radial ashing (option 2) form the basis of this report. The options addressed are as follows:

Option 1:

Parallel Shifts on the front stack, with radial shifting on top of the front stack – Stacking on top of the current facility will be a combination of both parallel and radial stacking. This is known as the parallel ashing model.

Option 2:

Radial Shifts on both the lower and top stacks – Stacking on top of the current facility will be a combination of both parallel and radial stacking. This is referred to as the radial ashing option.

After discussion with Eskom, it has been agreed to proceed with the radial ashing option as presented under **Figure 1-2** and **Figure 1-3.**

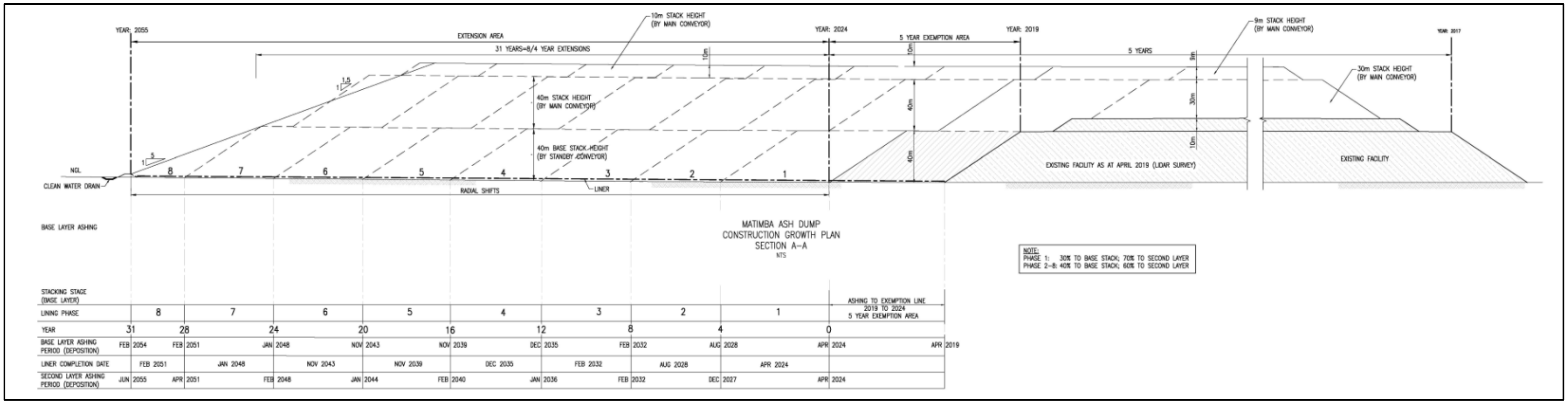


Figure 1-2: Typical ash dump layout and scope definition

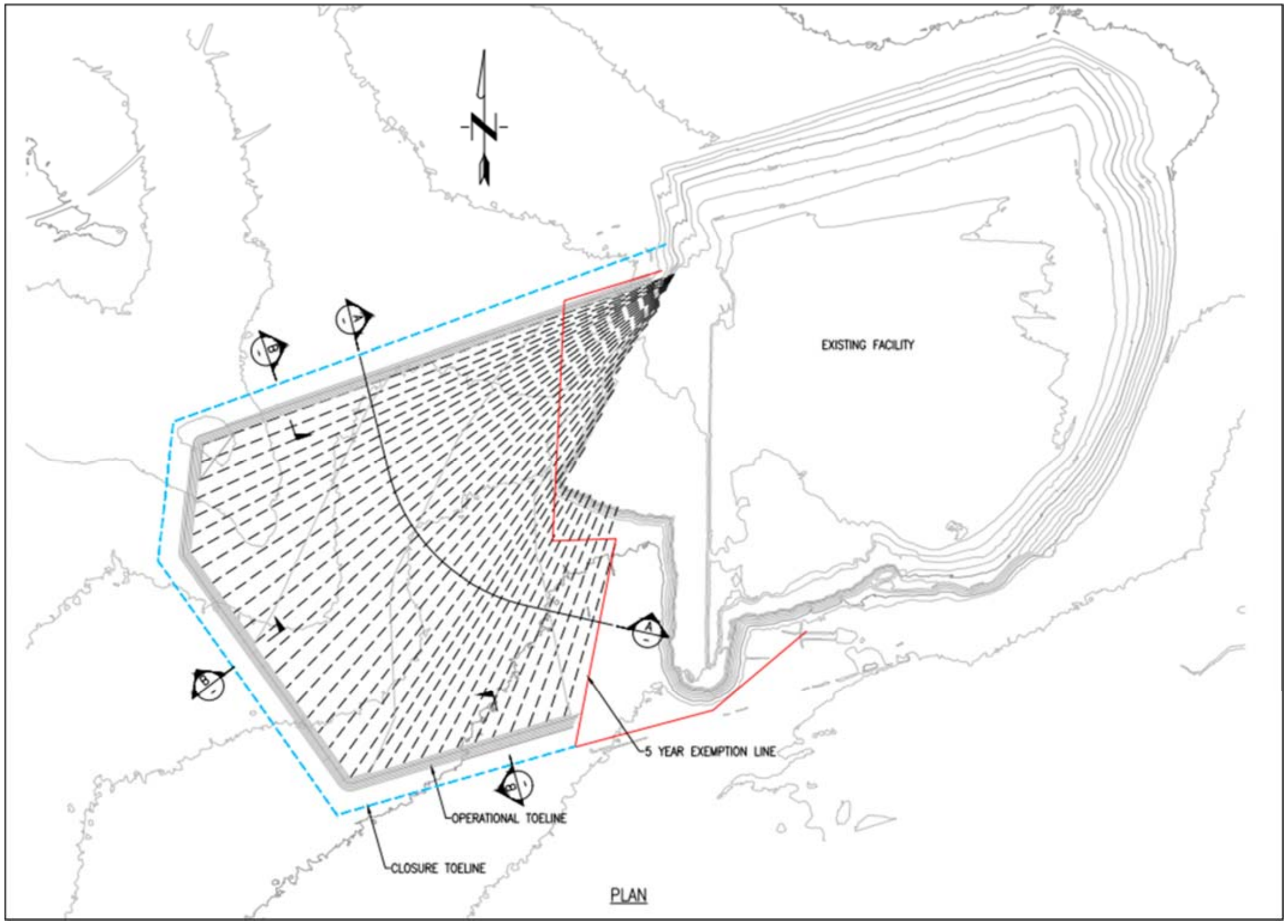


Figure 1-3: Layout Plan

2.0 AERIAL LIDAR SURVEY

2.1 AERIAL SURVEY EQUIPMENT AND SOFTWARE

Aircraft:

A Cessna 206 fixed wing aircraft was used. It was flown at a height of approximately 930 m above ground level.

Sensors used:

The airborne survey data was captured using a Leica ALS50 airborne laser scanner and a Phase One industrial calibrated aerial camera was used to capture the aerial photography.

Software:

To process the global positioning system (GPS) ground survey data, the airborne photos, digital terrain model (DTM) generation, the generation of Ortho-photos and the geoidal transformations the following software was used:

- Leica IPAS
- Novatel Explorer
- Leica ALS PP
- Microstation / uSmart
- TerraScan
- TerraModel
- Simactive

2.2 GROUND CONTROL SURVEY REPORT

A pre-marking signal construction and survey by Global Navigation Satellite System (GNSS) methodology was performed by Rasema Geomatics for the survey of the ground control points. These points consisted of a lime based white-wash Y marking of a suitable dimension to be clearly visible in the imagery.

Coordinate reference system:

Horizontal datum – Hartebeesthoek 1994, Lo27 Degrees.

Vertical datum – Mean sea level height based on the separation calculation from ellipsoidal height (WGS84) using the South African Geoidal Model of 2010 (SAGeoid2010).

The final Ground Control list is shown in **Table 2-1**.

Table 2-1: Final ground control list.

	Hart94 Lo27		
Name	**Y**	**X**	**Ortho H**
GCP01	-63,727.750	2,621,533.860	865.256
GCP02	-63,319.961	2,624,464.044	863.810
GCP03	-59,048.922	2,625,739.731	878.329
GCP04	-59,313.128	2,622,819.930	893.054
GCP05	-60,898.565	2,623,643.668	885.766

2.3 TOPOGRAPHICAL DATA COMPILATION

Aerial acquisition Date: 14 April 2019.

The raw point cloud individual flight line trajectories are matched using the position and attitude of each flight line. This method is done with the point heights based on the Ellipsoid (WGS84).

The South African Gravitational Model 2010 (SAGeoid2010) was used to calculate the separation between Ellipsoidal heights and Mean Sea Level Heights for each point. The heights were then adjusted to the mine beacon height system.

The result is a point data set with heights above mean sea level (orthometric), based on the SA-Geoid 2010 geoidal model. These points are then compared or checked against the supplied ground control survey heights as based on the mine beacon height datum and a final height adjustment was performed.

DTM ground points over the ground control comparison is shown in **Table 2-2.**

Table 2-2: DTM ground points comparison

	Hart94 Lo27				
LABEL	**Y**	**X**	**Known Z**	**DTM Z**	**DH**
GCP01	-63,727.750	2,621,533.860	865.256	865.190	-0.066
GCP02	-63,319.961	2,624,464.044	863.810	863.850	0.040
GCP03	-59,048.922	2,625,739.731	878.329	878.350	0.021
GCP04	-59,313.128	2,622,819.930	893.054	893.190	0.136
GCP05	-60,898.565	2,623,643.668	885.766	885.660	-0.106

The root mean square error is 0.085m.

2.4 CONTOUR GENERATION

The DTM was used to create a triangulated surface model. From the surface model contours at 0.5 m interval were generated utilizing TerraScan and TerraModeller software. The contour data set was used as a quality control tool to verify the overall correctness of the terrain representation and the final point classification.

2.5 AERIAL IMAGE RECORDING

The aerial images were captured with a calibrated nominally distortion free Phase One industrial 80mp digital aerial camera. In line with photogrammetric principles the photographs were taken with a 60 % forward overlap and a 30 % side overlap, this ensured that the final Ortho-photo is of a high geometric standard. This is shown in **Table 2-3.**

Table 2-3: Camera calibration

Parameter	X/Omega	Y/Phi	Z/Kappa
RMS Control	0.062	0.061	0.059
RMS Check	0.048	0.037	0.042
RMS Limits	0.100	0.100	0.100
Max Ground Residual	0.098	0.101	0.096
Residual Limits	0.100	0.100	0.100
Mean Std Dev Object	0.078	0.069	0.079
RMS Photo Position	0.002	0.004	0.016
RMS Photo Attitude	0.010	0.013	0.011
Mean Std Dev Photo Position	0.004	0.008	0.015
Mean Std Dev Photo Attitude	0.002	0.002	0.001

2.6 OUTPUT

An orthophoto image of the ash dump, with contour over-lay was produced, together with a 0.5 m contour site plan. This image is included in the drawings presented in **Appendix A.**

3.0 GEOTECHNICAL INVESTIGATIONS

The purpose of the geotechnical investigation was to determine the nature and extent of the soils and bedrock across the western open area and to provide recommendations for the foundation preparation of the ash dump.

3.1 GEOLOGY

According to the published geological map of the area (Ellisras 2326), the site is underlain by the Mogalakwena Formation of the Water Group. This formation comprises predominantly coarse-grained sandstone and in places conglomerate.

The Eenzaamheid fault that traverses east to west is located 2km north of the site, where the younger Karoo sediments are present north of the fault and which contains the coal formations. Numerous isolated north-west to south-east striking faults are located south of the investigated site but does not have any influence on the geology underling the investigated site.

According to Weinert's climatic index the site falls in an area classified as less than 5, indicating that the dominant weathering mode of rock is mechanical breakdown as opposed to chemical disintegration in areas classified as higher than 5. The mechanical breakdown of sandstone and conglomerate would typically produce coarse-grained soil horizons and limited in depth of weathering with bedrock often at shallow depths.

The previous report supplied by Eskom, "*Detailed geotechnical investigation for the proposed continuous ash disposal facility for the Matimba Power Station in Lephalale, Limpopo Province, South Africa*" prepared by Royal Haskoning in August 2013 was reviewed prior to the geotechnical investigation performed by Umbani Joint Venture for this project.

3.2 SITE DESCRIPTION

Three water reclamation dams are located south, east and north of existing ash dump. The planned extension of the ash dump covers a surface area of approximately 400 ha. The site is flat and slopes towards the south-east, with a fall in elevation from 900m above mean sea level (amsl) to 875 m amsl in the south-east corner of the open site. Most of this area is fenced off and kept intact as a nature reserve, seen to host natural vegetation and wildlife.

Vegetation comprises a typical arid bushveld with a loose sandy topsoil/ colluvium soil cover. The boundaries of the investigated area have been defined by the client, which excludes the south-western corner due to the presence of a natural non-perennial stream and drainage feature. Several smaller dams and channels were noted during fieldwork and serve as water sources for the wildlife in the reserve. Gravel roads were created running east-west and north-south dividing the site into subsequent blocks. Test pit positions were kept next to these roads in order to limit damage to sensitive flora.

A soil berm and channel were constructed towards the south-western corner of the ash dump to direct surface runoff into the southern reclamation dam. The area next to the western wall of the existing ash dam advance were cleared of vegetation and stripped of topsoil and transported soil up to bedrock or very dense material (mostly ferricrete) in order to accommodate the current ash load. Portions of this area were not accessible due to the soil berm. High voltage powerlines run along the northern and western boundaries of the site.

Bedrock outcrop of sandstone/ conglomerate with shallow soil conditions (up to 0.4 m thick) occurs within the central western and eastern portions of the site and comprises coarse grained soft to medium hard rock sandstone tending to conglomerate in places. Deeper ferruginised soils are located throughout the rest of the site.

The existing ash dump is covered by a loose sandy topsoil and overgrown by natural vegetation comprising grasses, short shrubs and trees resembling a savannah biome.

3.3 METHOD OF INVESTIGATION

The investigation comprised the excavation of forty-nine test pits by means of a Tractor Loader Backhoe (TLB) from 3 to 10 April 2019. In addition to the test pits, 12 auger holes were drilled during the same visit to assess the thickness of the topsoil cover across the existing ash dump. Test pits were excavated to maximum reach or refusal at shallower depths. Test pits and auger holes were profiled by an engineering geologist according to standard practice and the profile logs presented in **Appendix C2 and C3** respectively.

The positions of the test pits and auger holes are indicated in **Figure 3-1.**

The positions of the test pits were recorded with a hand-held GPS with an accuracy of 3 meters. The coordinates of these test pits are in WGS84 Datum, South African coordinate system (27L) as displayed on the test pit logs.

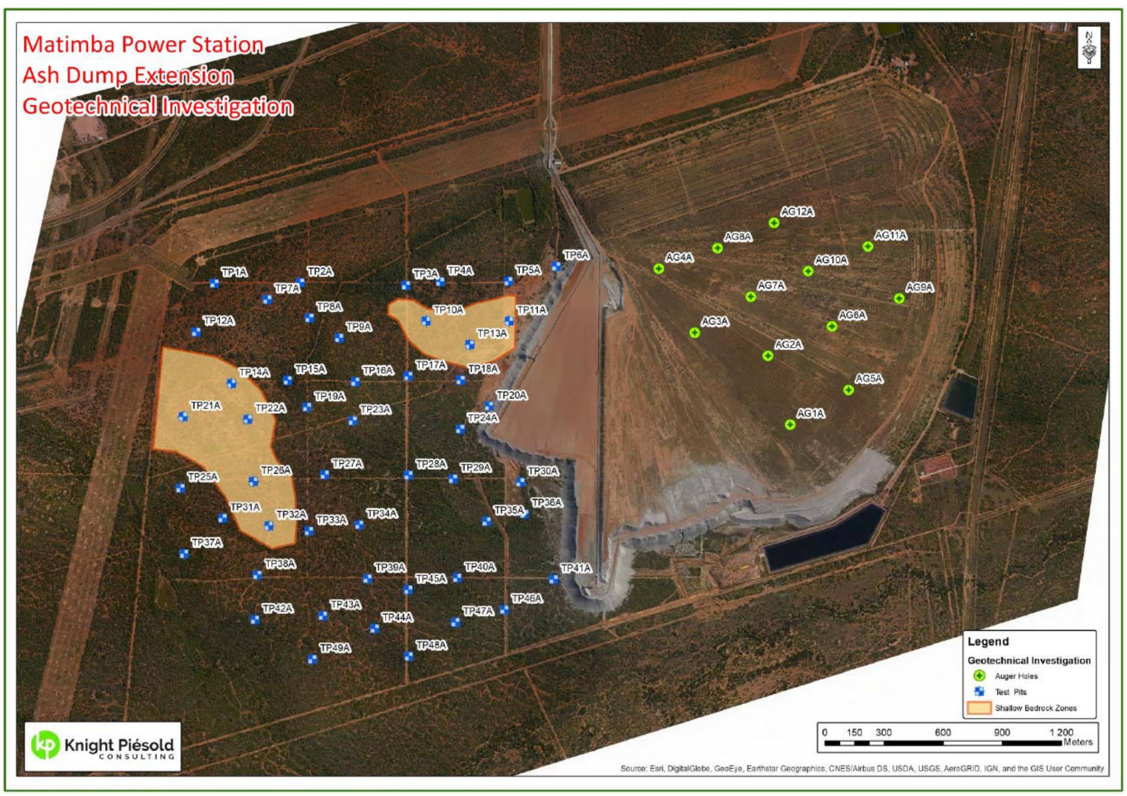


Figure 3-1: Test pit and auger hole positions

The coordinates for the test pits and auger holes are presented in **Table 3-1.**

Table 3-1: Coordinate list of test pits and auger holes

Name	X (LO27)	Y (LO27)	Name	X (LO27)	Y (LO27)	Name	X (LO27)	Y (LO27)
TP1	59,540.88	-2,623,004.84	TP22	59,710.69	-2,623,697.24	TP43	60,091.94	-2,624,694.88
TP2	59,976.57	-2,623,001.32	TP23	60,244.59	-2,623,704.32	TP44	60,355.67	-2,624,760.87
TP3	60,514.17	-2,623,013.95	TP24	60,789.97	-2,623,746.70	TP45	60,526.59	-2,624,563.00
TP4	60,687.93	-2,622,996.07	TP25	59,367.53	-2,624,046.39	TP46	61,013.29	-2,624,664.05
TP5	61,034.99	-2,622,993.53	TP26	59,739.57	-2,624,013.36	TP47	60,765.94	-2,624,728.25
TP6	61,278.54	-2,622,918.25	TP27	60,101.64	-2,623,978.63	TP48	60,528.34	-2,624,901.26
TP7	59,809.98	-2,623,086.25	TP28	60,527.48	-2,623,980.84	TP49	60,038.71	-2,624,915.97
TP8	60,023.76	-2,623,179.95	TP29	60,754.51	-2,623,999.28	AG1	62,466.10	-2,623,723.96
TP9	60,174.27	-2,623,282.03	TP30	61,095.61	-2,624,017.53	AG2	62,353.88	-2,623,371.84
TP10	60,616.28	-2,623,196.55	TP31	59,582.69	-2,624,200.00	AG3	61,980.67	-2,623,254.42
TP11	61,039.13	-2,623,196.13	TP32	59,817.58	-2,624,237.63	AG4	61,799.26	-2,622,930.15
TP12	59,448.39	-2,623,250.46	TP33	60,021.32	-2,624,264.50	AG5	62,763.01	-2,623,545.44
TP13	60,837.99	-2,623,314.82	TP34	60,277.20	-2,624,233.22	AG6	62,678.23	-2,623,222.74
TP14	59,627.51	-2,623,513.59	TP35	60,925.02	-2,624,215.99	AG7	62,265.66	-2,623,072.00
TP15	59,913.64	-2,623,496.05	TP36	61,115.67	-2,624,177.35	AG8	6,2097.23	-2,622,825.80
TP16	60,256.77	-2,623,503.55	TP37	59,386.97	-2,624,380.30	AG9	6,3019.43	-2,623,081.20
TP17	60,526.17	-2,623,475.51	TP38	59,759.44	-2,624,487.82	AG10	6,2557.17	-2,622,942.53
TP18	60,791.61	-2,623,495.56	TP39	60,320.33	-2,624,506.20	AG11	6,2860.68	-2,622,816.52
TP19	60,012.10	-2,623,634.68	TP40	60,775.96	-2,624,503.12	AG12	6,2384.88	-2,622,697.23
TP20	60,936.35	-2,623,629.42	TP41	61,263.37	-2,624,509.70			
TP21	59382.48	-2,623,683.27	TP42	59,747.50	-2,624,715.05			

Soil samples were taken from representative soil horizons and submitted to Specialised Testing Laboratories (ST Lab) in Pretoria. The laboratory testing comprised of:

- Foundation indicator tests (particle size analysis and Atterberg limits);
- Organic content tests;
- Standard proctor compaction tests;
- Shear box test on remoulded samples;
- Permeability tests on remoulded samples; and
- Consolidation tests on undisturbed samples.

Ash samples was taken from 4 locations on the ash dump, two samples from the conveyor location and 2 samples further away. The samples were submitted to Specialised Testing Laboratories (ST Lab) in Pretoria. The laboratory testing comprised of:

- Foundation indicators including specific gravity (SG);
- Standard proctor;

- Permeability tests; and
- Triaxial testing.

The laboratory results are contained in **Appendix C7.**

3.4 TEST PIT PROFILES

A summary of the soil profiles is provided in **Appendix C2** at the end of the report.

The investigated area is generally covered by transported soils and by a thin topsoil cover. Transported soils comprise colluvium and aeolian material and occurs from surface to a depth of between 0.4 m and 2.9 m. It has a pinhole voided soil structure and a loose to dense consistency with depth. The soil grading is mostly silty sand.

Ferruginised colluvium occurs occasionally and has a dense to very dense consistency also with a pinhole voided soil structure and comprises a grading of slightly clayey to silty sand and gravel.

The transported soil is underlain by residual sandstone and conglomerate towards bedrock. Strongly cemented pedogenic horizons (honeycomb to hardpan ferricrete mostly) occurs within and at the contact between the transported and residual soils.

Residual sandstone has a dense to very dense consistency and an intact to pinhole voided soil structure. The soil has a grading of gravelly silty sand to sandy gravel. Residual conglomerate is similar in appearance to residual sandstone and is clast supported sandy gravel with predominantly rounded coarse quartz gravels.

A well-developed pedogenic horizon (ferricrete with lesser calcrete) has developed within the transported and residual soils at depths of between 0.4 m and 2.6 m. The pedogenic horizon has a consistency of between very dense to soft rock strength and comprises honeycomb to hardpan ferricrete, with honeycomb being the most persistent across the site. Excavation refusal occurred at the base of most pedogenic horizons present within the investigated area.

Bedrock occurs as highly weathered very soft to soft rock sandstone and conglomerate. Excavation refusal occurred on the soft rock sandstone/ conglomerate but also on the honeycomb to hardpan ferricrete.

Two preliminary geotechnical zones were identified across the investigated area. This includes Zone A and Zone B. Zone A is characterised by shallow bedrock and areas where excavation refusal occurred at depths than less than 1.5 m. Zone B comprises deeper residual and transported soils with highly developed pedogenic soils resulting in variable refusal depths, i.e. between 1.5 m and 2.9 m. These zones will be described in detail in the final design report.

No groundwater seepage was encountered across the investigated area.

3.5 DYNAMIC PROBE SUPER HEAVY (DPSH) TEST RESULTS

The depths to penetration refusal are included in **Table C-1** under **Appendix C** next to the test pit depth. The results of the test pits indicate that the colluvium has mostly a loose consistency, however the results of the Dynamic Probing Super Heavy (DPSH) test indicate that most of the colluvium has a dense or very dense consistency. The DPSH test also encountered penetration refusal mostly at shallower depths than the bedrock depth.

The results do not correlate with the test pit logs since the colluvium and especially the residual conglomerate soil contains numerous gravel and cobbles comprising hard rock quartzite. It is assumed that penetration refusal occurred on the gravel or cobbles and caused higher density readings than logged in the test pits.

The results of the DPSH test are therefore discarded and not applicable due to the resistance of the gravel and cobble content of the soil horizons.

3.6 EXISTING ASH DUMP TOPSOIL

Twelve auger holes were drilled across the existing ash dump in order to assess the thickness and extent of the topsoil cover. A summary of the auger hole profiles is provided in **Appendix C-3** together with the auger hole profiles.

The topsoil is described as orange to dark brown, organic rich silty to gravelly sand with gravels comprising of quartz and ferricrete nodules. The topsoil is present at variable depths, ranging between 0.2 m and 0.8 m from surface, which is underlain by coal ash.

At AG1 excavation refusal was encountered on a very dense horizons at 0.4 m depth, presumably on boulders.

3.7 LABORATORY TEST RESULTS

The laboratory test results are provided in **Appendix C-5** and summarized in **Table C-3** under **Appendix C**. Most of the samples comprised colluvium and residual sandstone due their abundancy, while the remainder of samples comprised calcrete and conglomerate.

Colluvium

The colluvium, according to the results, generally comprises slightly clayey silty sand. The sand content averages 80 %, and the soil has a Grading Modulus (GM) of 1 to 1.5 (average of 1.26). The clay content varies between 3 % and 10 % (average of 5 %) and the soil is either non-plastic or has a Plasticity Index (PI) of between 3 % and 6 %. The soil has a low potential for expansiveness.

One sample that comprises calcareous colluvium (TP45) has a higher gravel content of 44 %.

The Standard Proctor Maximum Dry Density (MDD) for colluvium varies between 1,942 kg/m^3 and 2,088 kg/m^3 (average of 2,055 kg/m^3) with an Optimum Moisture Content of 7 % to 11 %. The soil compacted to 95 % of MDD has an internal friction angle of 34° with a zero cohesion. At the same compaction the soil has a coefficient of permeability (k-value) of 1,9 x 10^{-4} cm/s to 2 x 10^{-5} cm/s.

Residual Sandstone

The residual sandstone is relatively variable and comprises a silty clayey sand to silty gravelly sand. The clay content, mostly due to the reworking, varies either from low content but as high as 25 % at TP20. The PI varies from slightly plastic to between 9 % and 15 % (average of 12 %) and the GM between 1 and 1.6 (average of 1.15). The soil has a low potential for expansiveness.

Compaction tests yielded an MDD of between 1,708 kg/m^3 and 2,012 kg/m^3 (average of 1,886 kg/m^3) with an OMC of 9 % to 18 %. One shear box test on the soil at TP40 indicated an internal friction angle of 31 % with a cohesion of 8 kPa. Permeability tests on samples recompacted to 95% of MDD yielded a coefficient of permeability of between 1.1 x 10^{-7}cm/s to 3 x 10^{-8}cm/s.

Residual Conglomerate

The coarse-grained conglomerate comprises sandy gravel with very little fines content. The PI values are low and the clay content at an average of 4 %.

Compaction tests indicates an MDD of between 2,258 kg/m^3 and 2,293 kg/m^3 with an OMC of 6 % to 7 %. Permeability tests indicated similarly a low coefficient of permeability of between 3.1 x 10^{-5}cm/s and 1.3 x 10^{-6}cm/s. No shear box tests were conducted on the material.

Honeycomb Calcrete

One sample at TP7 was tested of the honeycomb calcrete. It comprises a slightly silty gravelly sand with a clay content of 3 % (slightly plastic) and has a GM of 1.86. No compaction or permeability tests were conducted on the calcrete.

Organic content tests were conducted on samples comprising mostly colluvium, while two were conducted on residual conglomerate. The results indicate that the organic content is generally low and varies between 0.6 % and 3.8 %. One sample of the calcareous colluvium indicated an organic content of 10 %. Plate 11 provides a typical view of the topsoil.

Consolidation tests were conducted on two undisturbed samples at TP2 and TP31. The results however did not indicate that consolidation will be taking place but rather compaction settlement of the soils during loading [4]. The results indicate that approximately 200 mm to 300 mm of compaction can take place for loads of up to 500 kPa. One collapse potential test at TP29 indicated a collapse at a stress of 200 kPa, which reduced the sample volume by 38 %.

Ash

The laboratory results on the ash indicated a material acting as a sandy gravely material. The particle size distribution where also in the sandy gravel range with no apparent plasticity. The material dry density was on average 800 kg/m^3 with a particle density of 2.1. Average permeability was tested as 3.11E$^{-0.6}$ m/s. The cohesion of the ash was tested as 0.0 kPa and the friction angle on average was tested as 30 degrees.

3.8 GEOTECHNICAL EVALUATION

3.8.1 FOUNDATION RECOMMENDATIONS

The current ash dump, that continuously advances towards the west, requires suitable foundations for the extension to limit excessive settlement. The ash material dumped to its optimal height is covered by topsoil to limit any erosion caused by wind of rainfall. It is thus of importance that the settlement is limited to ensure the topsoil cover is not affected. It is assumed that the topsoil material is obtained from within the foundation of the ash dump extension area.

The soil profiles are relatively consistent across the site. The weathering of the underlying sandstone and conglomerate formations are limited, and soil profiles is generally thin and comprise of a sandy nature. Colluvium is the most abundant soil type and covers the site to various depths of between 0.4 m and 3.4 m and has an average thickness of 1.2 m. The underlying residual sandstone or conglomerate is thin and comprises a dense to very dense consistency.

The soil profiles can be divided into two zones, namely soil profiles with a thickness of less than 1.5 m, and soil profiles with a thickness of more than 1.5 m and limited to 3.5 m. These zones (Zone A and B) are illustrated in **Figure 3-2** and labelled in **Table C-1** under **Appendix C**.

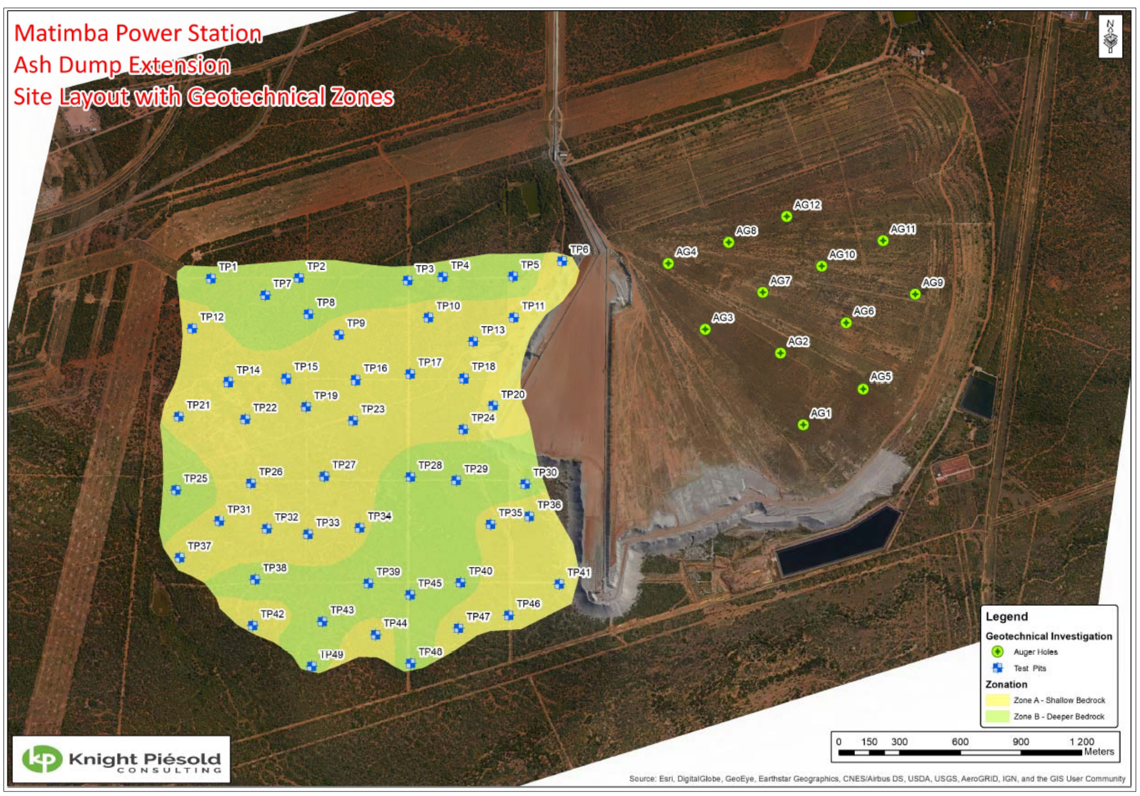


Figure 3-2: Geotechnical zones

The recommendations for the foundations of each zones is as follows:

Zone A: Shallow Bedrock Zone

- Bush clearing is mostly required for access within this zone. The upper 0.2 m rarely contains enough organic material suitable for topsoil for cover of the existing ash dump.
- Remove a maximum of 0.5 m of colluvium from surface level, which comprises the loose silty sandy soil. The loose consistency soil includes a pinhole voided soil structure and may cause excessive compaction settlements of approximately 200 mm to 300 mm for every 1 m of material thickness. The material should be removed and spoiled.
- Compact the in-situ floor of the foundation to at least 93 % of Standard Proctor density at optimum moisture content. A large 12-ton vibratory roller should be suitable.
- Settlement below the foundation is expected to be less than 100 mm for loads of up to 500 kPa.
- To level the highly undulating areas for the placement of the liner, the colluvium or residual sandstone soils can be utilized. Backfilling should be conducted by placement of layers limited to 250 mm thickness, compacted to 93 % of Standard Proctor MDD at optimum moisture content.

The following areas have very shallow bedrock and rock outcrop at surface:

- Area covered by test pits TP10, TP11 and TP13, and
- Area covered by test pits TP14, TP21, TP22, TP26 and TP32.

Excavation at these positions is not required, only removal of vegetation and in-situ compaction where rock outcrop is not visible. It is assumed that a slightly undulating floor would not cause detrimental problems for the foundation of the ash dump extension.

If stripping of outcrop rock is required a D9 ripper may be utilized to remove a maximum of 500 mm of surface rock. Any deeper excavations in rock may be classified as hard excavation.

Zone B: Deeper Bedrock Zone

The deeper bedrock zone can be divided into two parts, namely the northern and southern areas. These two areas have distinct different in situ consistencies and allowable bearing capacities.

Recommendations for each area are as follows:

Northern Area at Zone B:

- This area was covered by test pits TP1 to TP8.
- Removal of at least 1,7m of material is required since the area is covered by loose colluvium with a pinhole voided soil structure to least to 2 m depths.
- Compact the in-situ floor at least to least 93 % of Standard Proctor density at optimum moisture content with the same roller.
- Settlement below the foundation is expected to be less than 100 mm for loads of up to 500 kPa.
- Sidewalls of any excavation slopes should be battered at least to 1:2 (V:H) to ensure safe slopes.
- If backfilling is required can the same apply as discussed for Zone A.

Southern Area at Zone B:

- This area has thick soils profiles but the soil consistencies from depths of between 0.5 m and 1 m is generally medium dense to dense and occasionally very dense towards bedrock.
- The foundation preparations for this southern area can follow the same recommendations as recommended for Zone A, viz. the removal of 0.5 m of colluvium and in situ compaction as specified above.

3.8.2 CONSTRUCTION MATERIALS

Ash dump topsoil:

The results of the organic content tests indicated relatively low contents from 0.6 % to 3.8 %, while one sample tested at 10 %. The required organic content of topsoil is generally between 12 % and 18 %, indicating that the topsoil, or upper colluvium material is generally poor in organic content.

The recommendations to increase the organic content to utilize as topsoil on the existing ash dump are as follows:

- Mix the topsoil with fertiliser before placement on the ash dump. The amount of fertilizer required should be recommended by a professional agronomist.
- Fertilize the soil by hydroseeding it.
- Use the vegetation removed during the bush clearing to decompose and form organic rich content suitable to mix with the soil.

In-situ materials from foundation:

Two materials were mainly tested for reuse of construction materials, namely the upper colluvium covering the site and the lower residual sandstone.

The colluvium may be utilised for embankment construction materials since the material has relatively high strength characteristics (internal friction angle of 34°). However, the clay content of the soil is low, and the material comprises a low coefficient of permeability (less than $1x10^{-5}$ cm/s). The colluvium is also suitable as general or bulk fill above the foundations for preparation of the liner.

The colluvium material was not tested for road or platform construction material; however, it is assumed that according to the grading and Atterberg limits the material is of poor quality.

The residual sandstone soil has slightly lower strength characteristics but contains suitable clay content. This residual soil appears to be more suitable for the construction of berms or embankments since it has a coefficient of permeability of $1x10^{-8}$ cm/s. The residual sandstone is also suitable for backfilling on foundations below the liner.

It is anticipated that the pedogenic soil, honeycomb or hardpan calcrete, is a suitable material for roads or platform construction. This material was difficult to obtain for laboratory testing since excavation refusal was encountered on it. It is widely known that the calcrete can produce materials, classified according to COLTO [5], of G5 to G6 quality and suitable for road and platform construction.

The poor grading and sub-rounded to rounded gravel content of the residual conglomerate makes the material not suitable for construction.

4.0 SLOPE STABILITY

A slope stability assessment was done on the proposed design of the Matimba Power Station. ash dump extension. The aim was to determine the minimum factor of safety under operational conditions and for long term conditions after closure.

4.1 STABILITY ANALYSIS

The slope stability assessment was performed using the Rocscience programme: Slide Version 8. An initial slope stability assessment was conducted using published material properties for Matimba ash dump together with information that was obtained from a stability assessment obtained from Jones and Wagener Report No: JW171/03/8939, October 2003.

Various options regarding the outside slope and lift heights have been investigated. Each option was also evaluated with and without a smooth HDPE liner, and with an underlying GCL. These results were used as a starting point for the design to evaluate the best slope configuration regarding lift heights. Typically, the results returned lower factor of safeties for the slopes with the HDPE liner system.

Analysis on the current proposed design focused on the influence of the barrier system and the friction interfaces between the different barrier system components. As well as the loading associated with the stacker (stacker load estimated at 90 kPa).

4.1.1 MATERIAL PROPERTIES

Table 4-1 summarise the material properties that were previously applied to Matimba ash dump by Jones and Wagener (October 2003) as well as material properties based on the laboratory ash tests done during the design phase. Liner interface analysis done by Jones and Wagener was deemed sufficient as it was in line with expectations from the suppliers. It also summarises the average friction interfaces between the different materials that was observed on similar projects and dams. In order to remain on the conservative side and to make sure that the stability effects of the proposed liner is taken into consideration, the published liner interface with the lowest friction angle was used to model a 'weak' layer representing the liner and a possible slip surface. Further to this it was assumed that the foundation preparation would be similar to the existing ash dam and that the foundation material will have a some firmness, but to ensure a conservative analysis approach and to take into account the possible honeycombing effect that was observed in some test pits, the friction angle was reduced to around 12 degrees to simulate a layer that may be weak therefore accounting for potentially weaker material under the liner. A conservative seismic ground acceleration of 0.1 g was used based on the Seismic Hazard Map of South Africa.

Table 4-1: Summary of material properties

Material	Unit weight (kN/m^3)	Cohesion (kPa)	Friction Angle (degree)
Ash	8	0	30
Foundation soil	16.5	12	12
Foundation rock	20	20	35
Barrier system	8	0	6

Interface between barrier materials per Jones and Wagener			
Ash / Protection layer	-	0	30
Protection layer / HDPE	-	15	32
HDPE / GCL - Peak	-	0	25
HDPE / GCL – Residual 1	-	25	3
HDPE / GCL – Residual 2	-	0	9

4.1.2 ANALYSIS AND RESULTS

The analyses were conducted using previously published material information on both Matimba and similar ash dump projects in combination with the results from the laboratory testing. The results from the liner shear interface testing must be verified with the values used for the analysis and the analysis should be updated accordingly. The quality and properties of the liner used for construction of the liner system must be compared to the sample used to do the liner shear interface testing so that the materials used for construction have similar properties to the material that was used for during the shear interface tests, alternatively it is recommended that shear interface testing be done on the material to be used for the liner construction for consistency and to meet the design specifications.

Various analyses were done and include the following:

Different slope configurations; typically, the ash will be stacked at angle of repose and then reshaped at a later date. Jones and Wagener (October 2003) found that the slopes had a combined outer slope configuration with the base flow at 10 degrees, the top edge at an over steepened angle of 40 degrees and that the overall slope was on average 30 degrees.

A Class C barrier system was identified as being required for this facility. Traditionally this system incorporates a layer of clay, but as the site does not have the required clay to construct with, a material of similar performance needs to be used. Analyses focus on a GCL (Geosynthetic clay liner).

The analysis for the configuration where the slope is stacked at angle of repose of 30 degrees are shown in **Figure 4-1**. The first image is for the slope without any loading, the second image incorporate the stacker loading and the third image include a horizontal seismic acceleration of 0.1 g.

The analysis for the configuration where the slope is stacked at the reshaped 1V:5H slope is shown in **Figure 4-2**. The first image is for the slope without any loading, the second image include a horizontal seismic acceleration of 0.1 g.

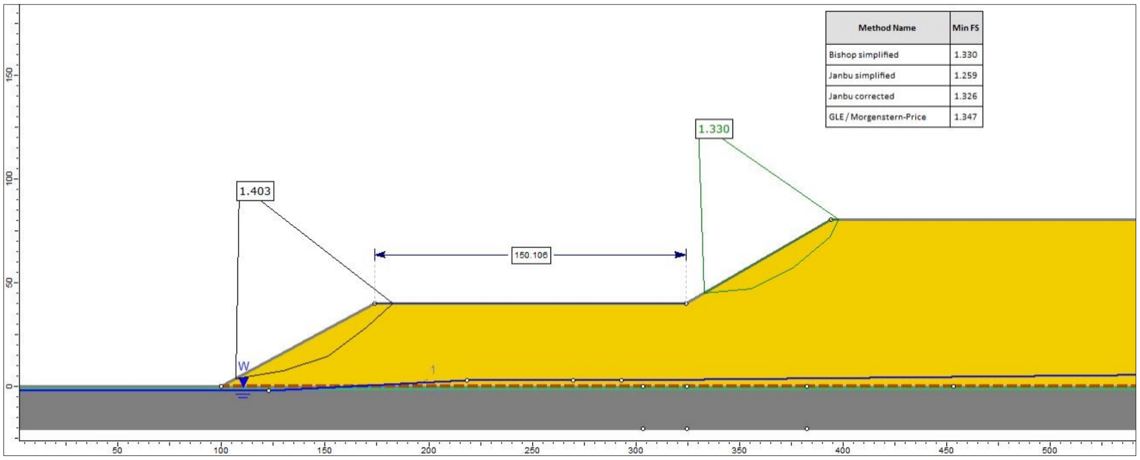
Various methods were used and are shown in the images but the Bishop Simplified method was the most representative with the Janbu method being very conservative and the Morgenstern Price method being slightly under conservative.

Table 4-2 follows with a summary of the results.

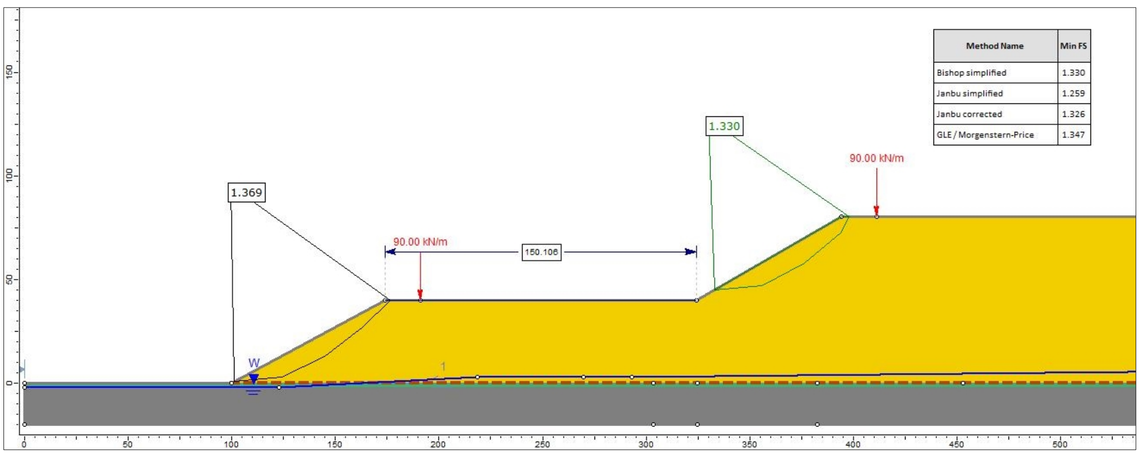
Table 4-2: Summary of Slope stability FOS

Slope	Lower stack slope			Upper stack slope		
	Static, Basic	Static with load	Seismic with load	Static, Basic	Static with load	Seismic with load
Slope at angle of repose (30 degrees)	1.40	1.37	1.18	1.33	1.33	1.09
Slope 1V:5H (as 1 slope)	3.17	-	2.08	-	-	-

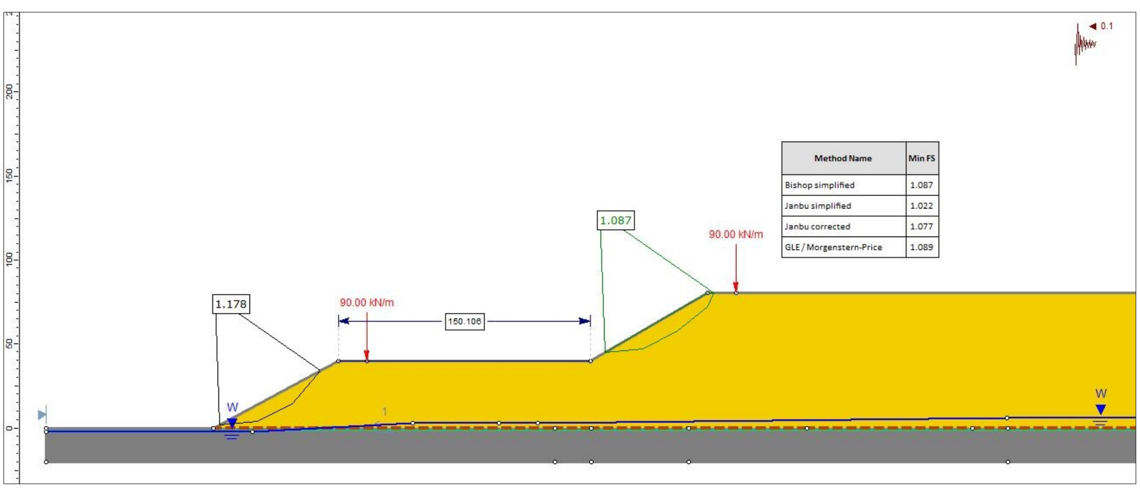
These results are all within acceptable norms, namely FOS 1.3 for the operating phase, 1.5 for closure and 1.1 for seismic loading.



1) Slope without any loading

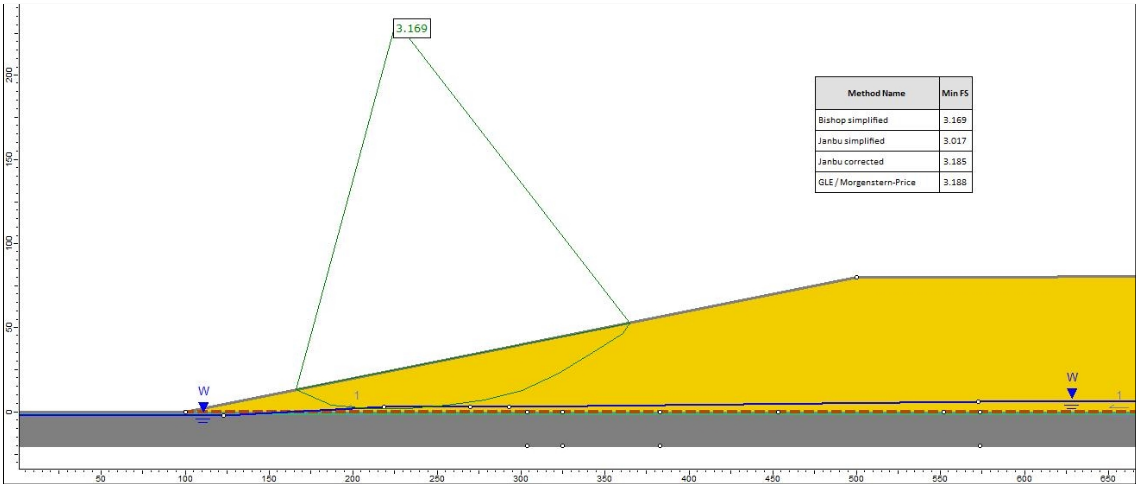


2) Slope with stacker loading

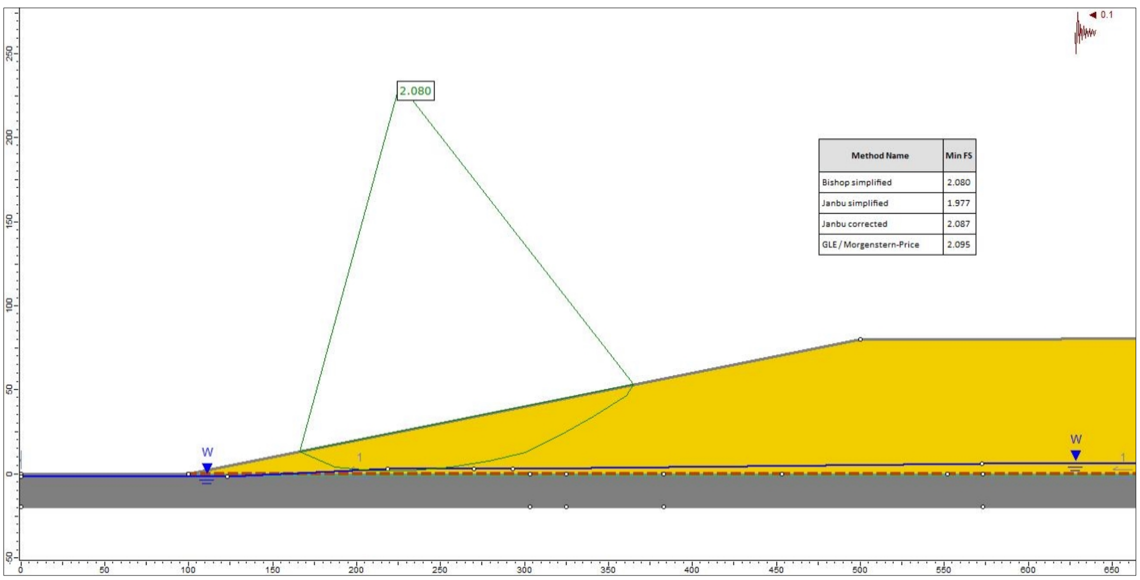


3) Slope with stacker and seismic loading

Figure 4-1: Typical slope stability results at angle of repose of 30%



1) Slope without any loading



2) Slope with seismic loading

Figure 4-2: Typical slope stability results at a slope of 1V:5H (Design reshaped)

4.2 CONCLUSION

This FOS is above the desired value of 1.5 for long term closure and above the minimum FOS of 1.3 for the operational or stacked slope. With the rehabilitation plan in place and with proper operational practices this FOS will be sufficient over a short-term operational period and should increase to the desired 1.5 once the slopes have been reshaped and rehabilitated at a 1V to 5H final slope.

A physical sample of the liner was tested using the liner shear interface tests and the results compares well. The results are attached to this report under Annexure I.

5.0 LINER DESIGN

5.1 ASH DUMP

The ash dump was classed as a type 3 waste which according to legislation requires a class C barrier system. The geotechnical investigation confirmed the lack of suitable clay material for the barrier system, therefore the barrier system was designed to incorporate a GCL.

5.2 POLLUTION CONTROL DAMS

The ash dump was classed as a type 3 waste which according to legislation requires a class C barrier system. It follows that the pollution control dams (PCD) will require a class C barrier system. The geotechnical investigation confirmed the lack of suitable clay material for the barrier system, therefore the barrier system was designed to incorporate a GCL.

5.3 NATURE OF ASH

The ash at Matimba will be stacked at a temperature (up to 45 ^{o}C), and the liner system layers, especially the HDPE liner, need to be able to perform satisfactory under these temperatures. According to a study done by Jeffares & Green in 2015 (Geotechnical Assessment and Thermal Investigation, Phase 2 Report No. 3145, Jeffares & Green (Pty) Ltd, 5 June 2015) the ash can reach temperatures of 43 ^{o}C.

The geochemistry of the ash (see Report Waste classification, Jefferson Green, 2013) indicates high concentrations of calcium and sulphate that can leach from the ash. Based on the waste classification a class C liner was proposed and accepted to comply with legislation.

5.4 LEGISLATION

National Norms and standards for the Assessment of Waste for Landfill Disposal, regulation 636 prescribes as a minimum a Class C barrier system for a Type 3 waste. The suggested configuration of this barrier system, as published in the Government Gazette, 23 Augusts 2013, No.36784, Regulation 636, is presented in **Figure 5-1**.

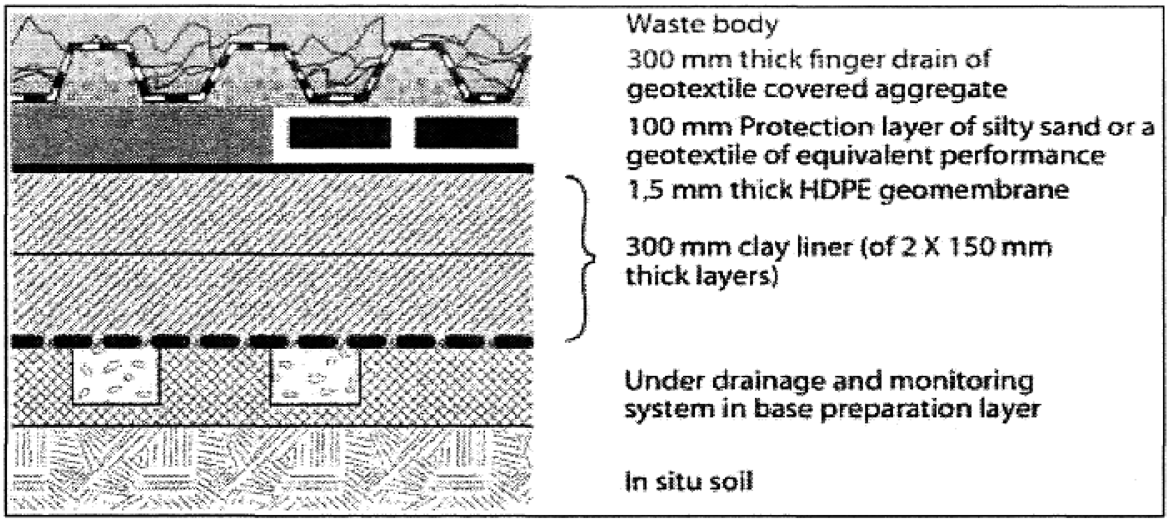


Figure 5-1: Class C Barrier System (Regulation 636)

The ash at Matimba will be dry stacked and the only water that realistically will reach the barrier system during the stacking process will be storm runoff and infiltration. No ground-water table was encountered during the site geotechnical investigation The site drainage will be achieved by a series of toe drains along the perimeter starter wall and solution trenches that will follow the progression of lining installation and capture and drain away the runoff and storm water from the barrier system. For this reason, the 300mm finger drain system, above the liner as in **Figure 5-1** above is omitted from this design. The proposed 300 mm gravel protection layer above the liner system will however provide some degree of moisture absorption.

5.5 PROPOSED BARRIER SYSTEM

Based on legislation, the prescribed barrier system can be adjusted as long as the performance can be proved to be similar or better than the prescribed barrier system. As previously mentioned, the finger drain system is replaced by the toe drain and solution trench system. The site in general have no clay that can be used for the proposed clay layers and therefore it will be replaced by a GCL of similar performance below the 2 mm HDPE liner. The general configuration will therefore be as shown in **Figure 5-2.**

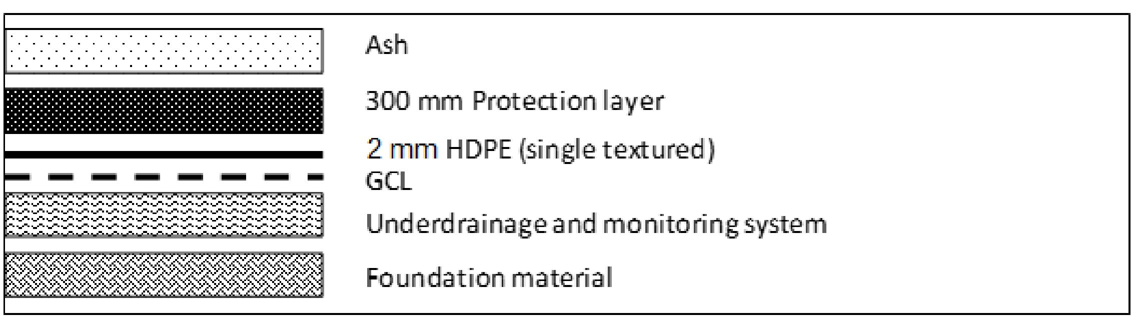


Figure 5-2: Proposed barrier configuration

5.5.1 PROTECTION LAYER

Considerations for the protection layer above the HDPE liner was determined as 3.6 mm sized gravel, sourced commercially if the material on site proof to be inadequate or insufficient. The protection layer will form a ballast layer on the HDPE liner preventing it from moving excessively during the stacking process and will also provide a barrier for dissipation of heat from the deposited ash. It will also aid in the protection of the exposed liners from the elements before the ash stacking process commences.

Coarse ash was also considered as a possible protection layer, but quantity, quality and availability of the material is questionable, and this material was rejected as a possible protection layer.

5.5.2 HDPE GEOMEMBRANE

A 2.0 mm single textured HDPE liner will be installed according to the specifications of GRI GM13, the texturing should be rolled and not blown therefor flat died manufacturing as opposed to blown film with an asperity height of no less than 0.4 mm (embossed friction layer). The liner will be installed with the smooth side upwards, forming an interface with the protection layer in order to allow some movement to reduce the possibility of the liner tearing under shear during the stacking process. The textured under-side will enable greater stability when it forms an interface with the GCL.

The high ash temperatures are a consideration when looking at the life of the HDPE liner. From the paper published in the Journal of Hazardous, Toxic, and Radioactive waste, ASCE, January 2014, (Service life of HDPE Geomembranes subjected to elevated temperatures, Jafari, Stark, and Rowe) it can be expected that the liner will have a service life of approximately 200 years for temperatures around 30 degrees Celsius. If the temperature increases to around 40 degrees Celsius this life of liner is reduced to about 100 years. The 100 year life of liner will be sufficient as the life of the dam is far less and once the stacking has stopped and the dam is rehabilitated the latent heat potential is reduced considerably.

5.5.3 GEOSYNTHETIC CLAY LINER (GCL)

As a replacement for the standard Class C clay layer, a GCL is proposed as it is currently the industry standard and proved to have a performance similar or better than the suggested clay layer. Alternatives for this material layer was investigated and deemed inferior to the GCL performance.

Test done by Jones and Wagener on a different ash dam indicated that the long-term permeability of the GCL may be affected by ash leachate. Eskom provided KP with results done by SmecTech Research Consulting (Victoria, Australia) that indicated that the current ash dam leachate is compatible with Eccabond K bentonite provided that the leachate does not increase in acidity.

For the Matimba ash dam we proceed with the GCL in combination with a 2 mm HDPE, on the basis that Matimba is a dry dump, and that it is extremely unlikely for a hydraulic head to develop above the HDPE from saturation of the ash. Furthermore, the development of leachate from the ash deposits will be minimal. It is thus likely that any pin-hole leakage of leachate through the HDPE will be minimal and will only impact on tiny, random zones of the GCL.

5.5.4 UNDERDRAINAGE AND MONITORING SYSTEM

A drainage pipe system consisting of 160 mm diameter slotted HDPE pipes will be installed below the GCL layer in the prepared foundation layer with a spacing of no more than 200 m. This layer should catch any seepage that may pass through the barrier system and report it to the dirty water solution trench. No groundwater was picked up during the site investigation that may cause a problem but in the event that ground water should increase to the point of reaching the barrier system, any build-up of pore pressures will be reduced by the underdrainage, thereby safeguarding the integrity of the barrier system.

5.6 INSTALLATION

The barrier system will be installed in accordance with SANS 1526 (2015), in approximate 4-year phases based on the progression of the radial stacking. This will reduce the exposure time of the barrier system to the elements that may compromise the integrity of the barrier system.

5.7 SUMMARY OF DWS REQUIREMENTS

The following points highlight the DWS requirements:

- The final design report and drawings will reflect the following points;
- Waste classification indicated Class C barrier system (refer to **Figure 5-1** and **Figure 5-2**) Type 3 waste may only be disposed of at a Class C landfill designed in accordance with section 3 (1) and (2) of this standard, or, subject to section 3 (4) of this standard, may be disposed of at a landfill site designed in accordance with the requirements for a GLB+ landfill as specified in the Minimum Requirements for Waste Disposal by Landfill (NEMWA R635, DWAF, 2013).
- Geotechnical site investigation findings can be found in **Appendix C**, dominant material on site include colluvium, residual sandstone, residual conglomerate and honeycombed calcrete, no phreatic surface was identified in any of the test pits at the time of the investigation.
- Design criteria, Alternative elements of proven equivalent performance were considered. As there is no clay locally available near the site, it is recommended that the two clay layers are replaced with a Geosynthetic Clay Liner (GCL) (Envirofix X1000 or similar).
- A toe drain above the liner at the toe wall to catch any storm seepage and convey it to the dirty water system
- A leakage detection drainage system beneath the liner to catch any leakage that may come through the expected 2.5 - 5 pinholes/ha as suggested by Giroud & Bonaparte (2001). This layer will also intercept ground water in the unlikely event that it reaches the barrier system. For the design it was assumed that 2.5 pin holes per hectare will occur (see **Figure 5-3** and **Figure 5-4**) as no HDPE liner is leak proof.
- Standards as per regulation 636 indicates a 300 mm clay liner, as the site have no available clay a GCL will provide the equivalent performance. For a 300 mm compacted clay liner the expected permeability is 10^{-9} m/s, the equivalent GCL performance requirement is around 4.6×10^{-11} m/s (values based on published Kaytech technical notes).
- Using Bernoulli's equation, the leakage through a single hole in an HDPE liner is estimated at 8.4×10^{-5} m/s. Thus, allowing for the suggested 2.5 holes per hectare, the total leakage can be expected to be 0.76 m^3/hr/ha

- For quality purposes the provider should be able to prove that both the GCL and HDPE liner adhere to the design specifications.
- Predicted service life based on 40 degrees heat of ash is around 100 years – this exceeds the expected life of the dump usage as it is further expected that the ash will cool down over time and reduce the heat impact on the liner.
- A minimum design factor of safety over the short term should not be less than 1.3 and over the long term 1.5. Current stability analyses for short term indicate factors of safety between 1.3 and 1.4.

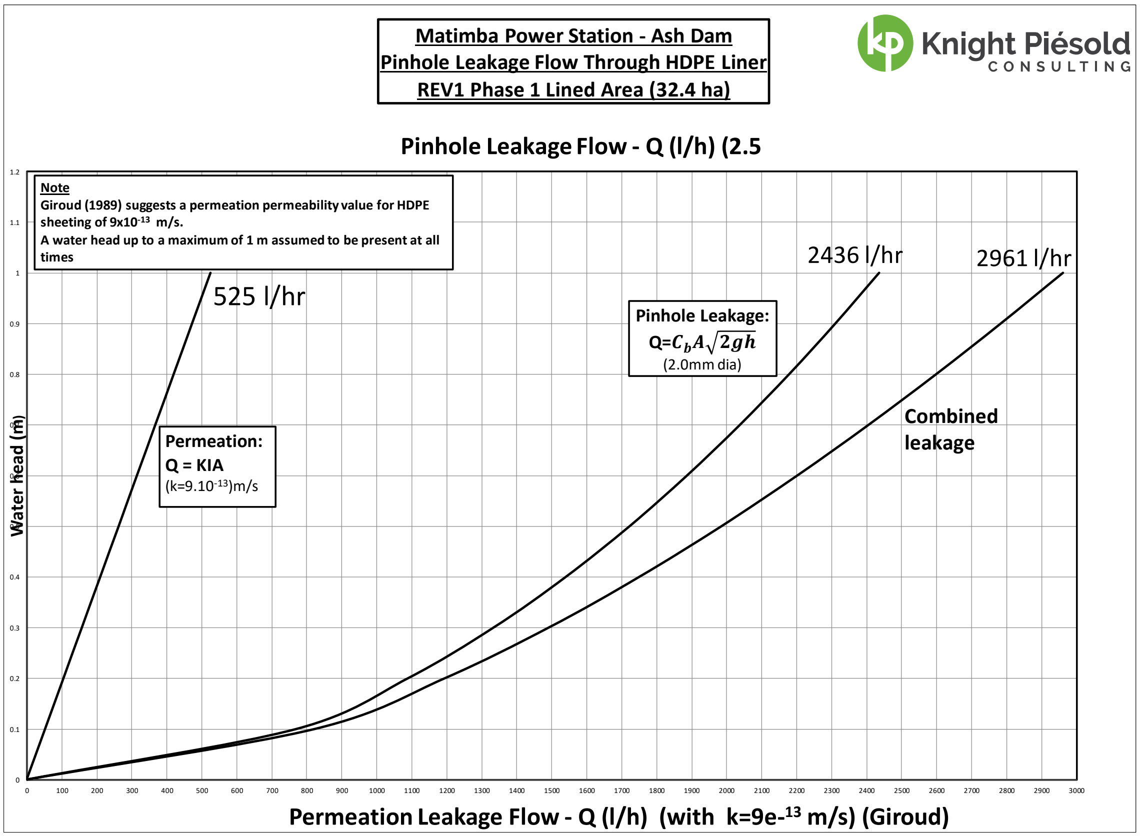


Figure 5-3: Pinhole leakage flow through HDPE liner for phase 1

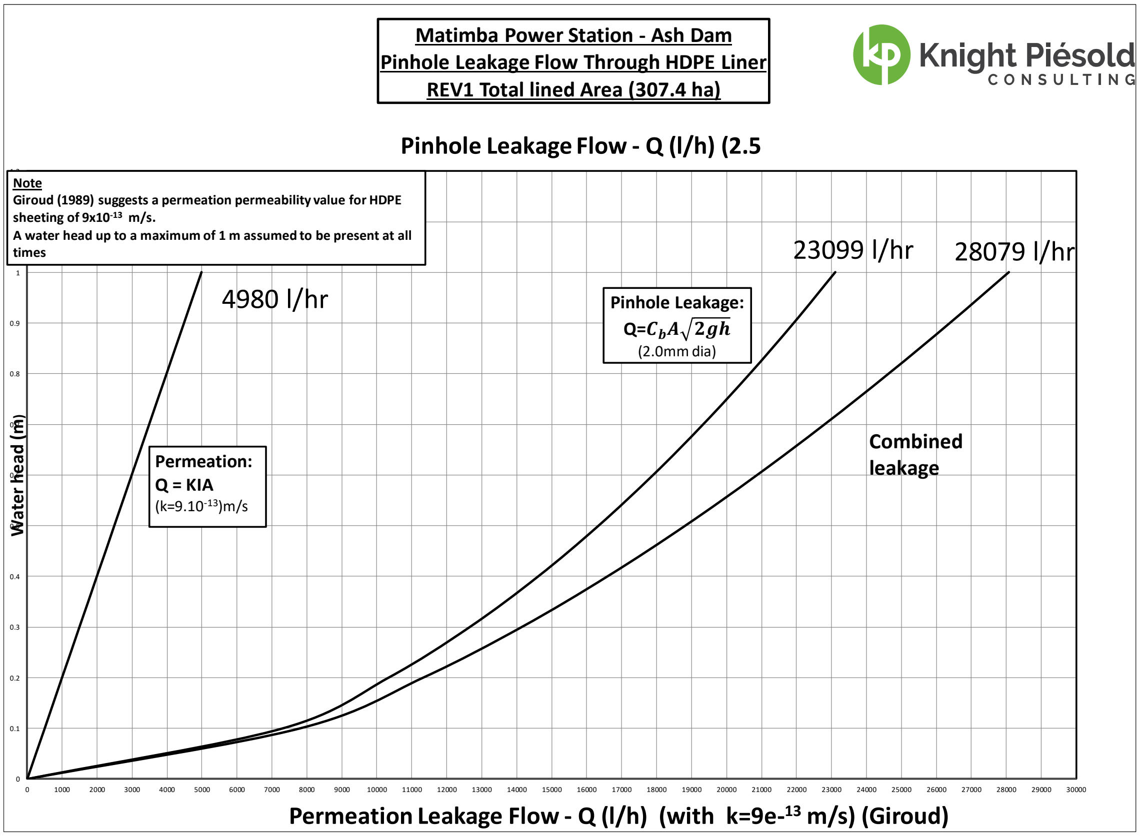


Figure 5-4: Pinhole leakage flow through HDPE liner for the total lined area

5.8 POLLUTION CONTROL DAM LINERS

The pollution control dams will have the same liner configuration as the ash dump. An underdrainage leak detection system will also be included.

5.9 CONCLUSION

The barrier design was based on the current legislation for a type 3 waste and the corresponding Class C barrier system. Variations in design was dictated by the availability materials at or close to site as well as by the nature of the ash itself. As this a ash dump designed to be operated as a dry stack, the only leakage issues that may arise is from storm water and in the unlikely instance where the ground water level (non was detected during the geotechnical investigation) may increase to reach the barrier system.

6.0 GEOMETRIC DESIGN

6.1 DEPOSITION AREAS

Matimba Power Station was commissioned from 1987 to 1991 and is expected to operate until 2055. The Ash Dump must provide storage capacity for all the ash produced at Matimba Power Station up to 2055. This detailed design caters for a remaining station life of 36 years from 2019 to 2055. The total storage capacity volume required for this 36-year period is 230 265 000 m^3. The area available for ashing activities is approximately 400ha of which 290 ha is utilized as part of this design.

The following areas have been defined as the available areas where ash deposition is to occur on Matimba Ash Dump:

- Northern Exemption Area
- Southern Exemption Area
- Lower Radial Front Stack Area
- Existing Facility Ramp Up Area – (parallel deposition) including upper Front Stack and Back Stack
- Existing Facility Area– (radial deposition) including upper Front Stack and Back Stack. Deposition will continue on the top of the Lower Radial Front Stack area.

Figure 6-1 and **Figure 6-2 i**ndicates the deposition areas as described above.

The deposition of ash onto the Ash Dump Facility occurs in nine (9) phases. The phases are defined in **Table 6-1** with the deposition rates per phase for the Main and Standby Systems defined in **Table 6-2.**

Table 6-2**.** The Deposition will occur on the lower front stack and on the existing facility simultaneously.

Table 6-1: Ashing Phase Definition

Phase	Period (Years)	Volume ashed (m^3)	Ashing area
Phase 0	2019 – 2024	31,981,250	Northern exemption, southern exemption, existing facility ramp up
Phase 1	2024 – 2028	25,585,000	Lower radial front stack, existing facility
Phase 2	2028 – 2032	25,585,000	Lower radial front stack, existing facility
Phase 3	2032 – 2036	25,585,000	Lower radial front stack, existing facility
Phase 4	2036 – 2040	25,585,000	Lower radial front stack, existing facility
Phase 5	2040 – 2044	25,585,000	Lower radial front stack, existing facility
Phase 6	2044 – 2048	25,585,000	Lower radial front stack, existing facility
Phase 7	2048 – 2052	25,585,000	Lower radial front stack, existing facility
Phase 8	2052 - 2055	19,188,750	Lower radial front stack, existing facility
Total		230,265,000	

Table 6-2: Phase deposition rates

Phase	Standby System Deposition Rate	Main System Deposition Rate
Phase 0	Variable*	Variable*
Phase 1	30%	70%
Phase 2	40%	60%
Phase 3	40%	60%
Phase 4	40%	60%
Phase 5	40%	60%
Phase 6	40%	60%
Phase 7	40%	60%
Phase 8	40%	60%

Phase 0 includes variable deposition rates in the defined areas for ashing in order to cater for the relocation of the Main System onto the existing facility as well as the relocation of the Standby System from the Northern exemption area to the Southern exemption area. Please refer to the detailed growth development plan in **Appendix E.*

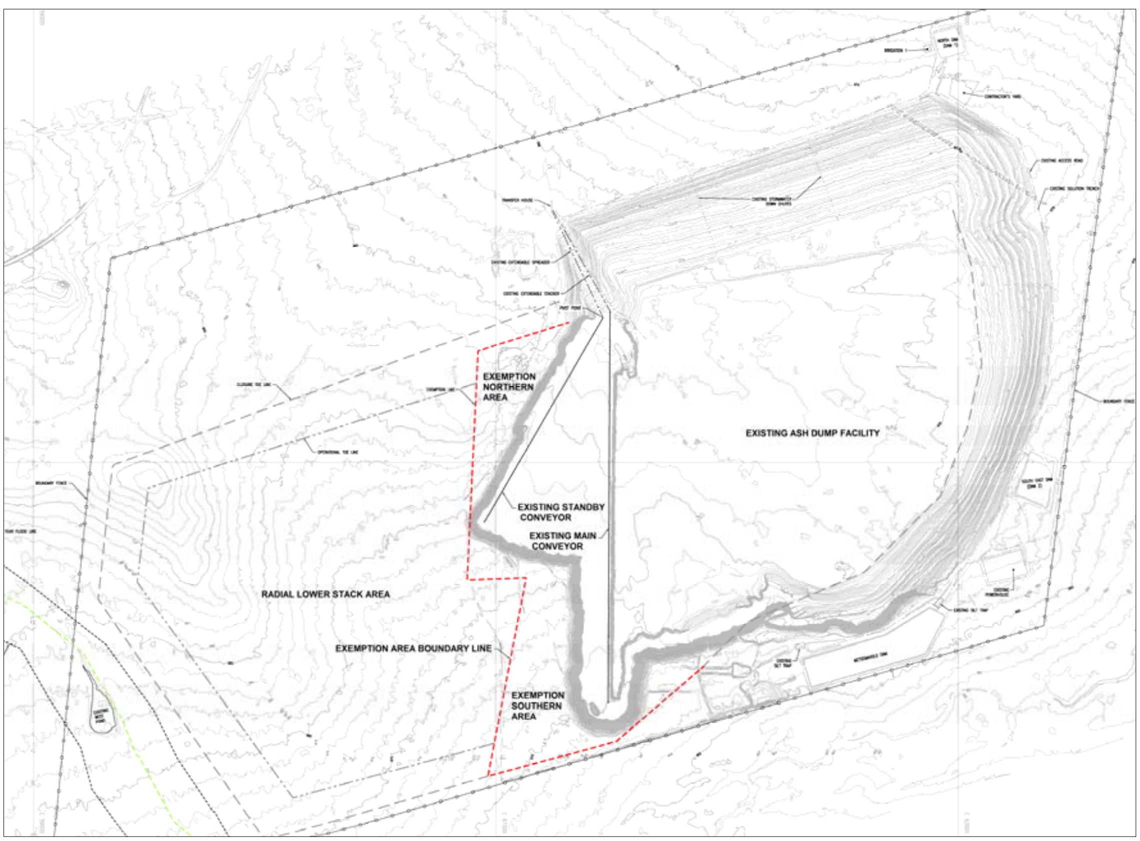


Figure 6-1: Northern & Southern Exemption Area and Radial Lower Stack

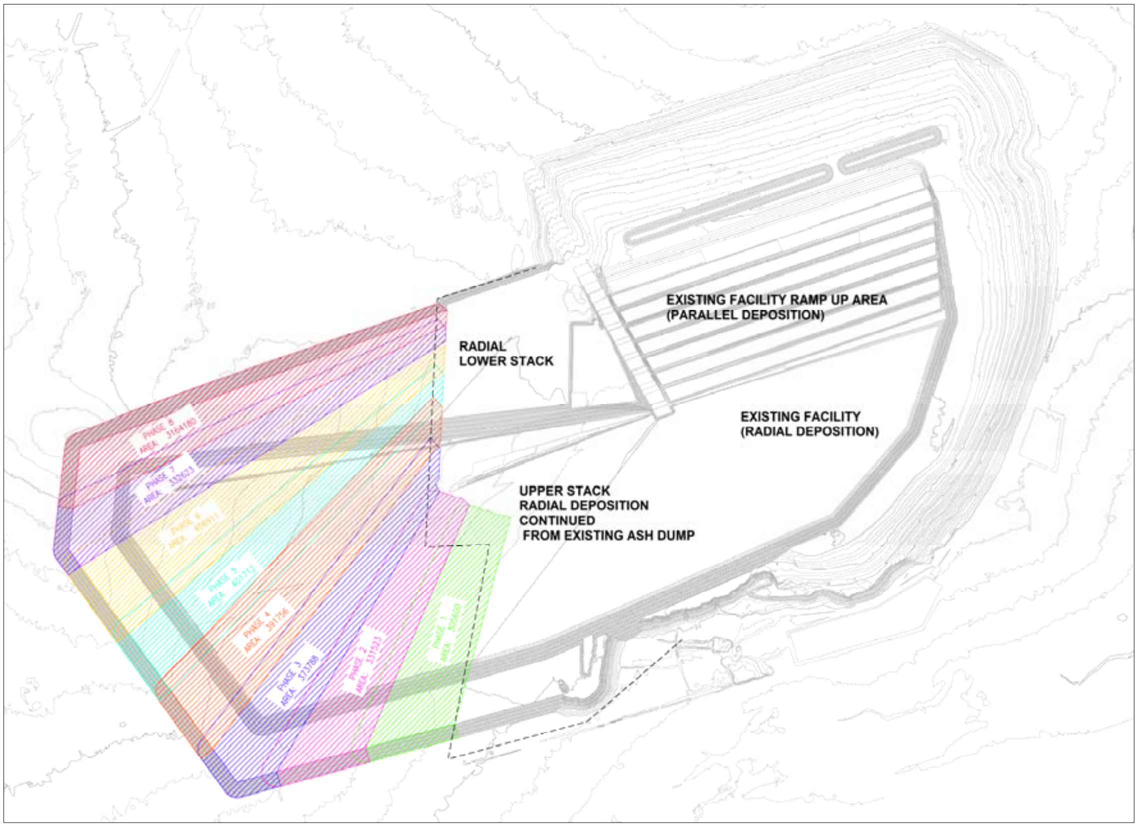


Figure 6-2: Deposition areas – Ramp up and upper stacks

6.2 CAPACITY ASSESSMENT

The storage capacity for each of the areas listed in Section 6.1 is illustrated in **Table 6-3.**

Table 6-3: Defined ashing area storage capacity

Defined Ashing Area	Volume (m^3)
Northern Exemption Area	3,782,237
Southern Exemption Area	8,046,774
Lower Radial Front Stack Area	77,952,797
Existing Facility Ramp Up Area	21,200,049
Existing Facility Area	128,043,956
Total	239,025,813

6.3 CONVEYOR SETUP

In order to commence with ashing on the existing facility, the main conveyor system is required to be moved from its current position to the top of the existing ash dump while the standby conveyor deposits in the north side of the exemption area. Ash will need to be deposited to create a ramp for the main system to climb so that the main conveyor system can begin ashing on the with main stacker on the existing facility in a parallel manner.

The standby conveyor will be utilised in order to deposit the ash in the front lower stacking area.

Once in position, the main conveyor will initially deposit on the existing facility building up a ramp in a parallel manner, until it reaches a height of 30m high. At the end of the parallel conveyor movements the main extendible will be moved and the ramp dozed to a 1:10 slope, the conveyor is then to be moved back in place as a final position. The main conveyor will then begin to deposit in a radial manner on the existing facility, creating both a 40m high front stack and a 9m high back stack during the piggybacking and a 10m high back stack after the piggy-backing has been completed. This deposition will continue radially onto lower stack created by the standby system, in order to create the upper frontstack and back stack on the ash dump extension. The facility has been designed in a manner that that it does not exceed the 90m height restriction. The elevation of the facility has been designed not to exceed the 90m Ash dump height restriction. The height of the facility must at all times be monitored in order to not exceed this height restriction. The existing positions of the conveyors are indicated in Figure 6-3.

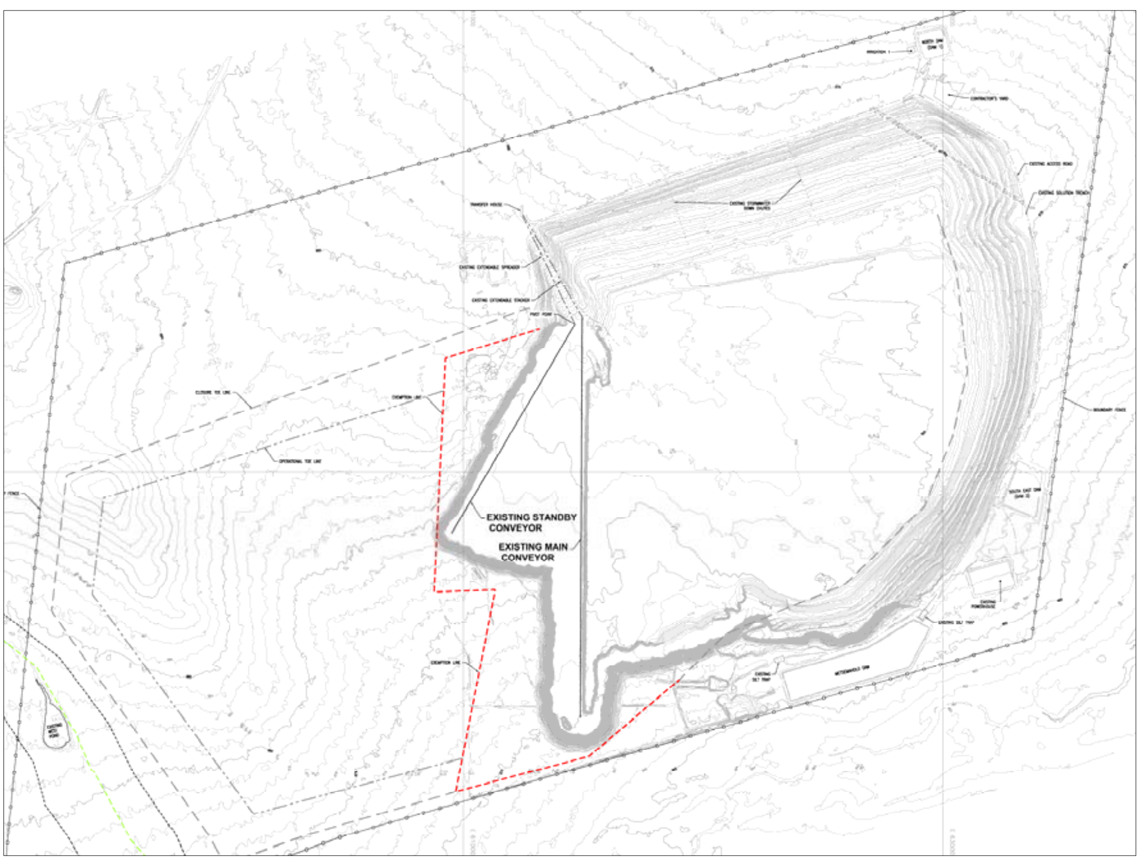


Figure 6-3: Conveyor positions

Each phase is broken down into stages as per the conveyor shifts for that phase. The drawings that refer to these stages are titled as "Layout Plan and typical sections" for each phase and can be located in **Appendix A**. An example of the conveyor positions for the ramp-up section is illustrated as below in **Figure 6-4**. The stages for the ramp up section are representative of the conveyor position I.e. Stage 5 in **Figure 6-4** indicates the conveyor would be at position 5 on the ramp-up section. **Figure 6-4** is referenced from drawing 301-00825/01-117.

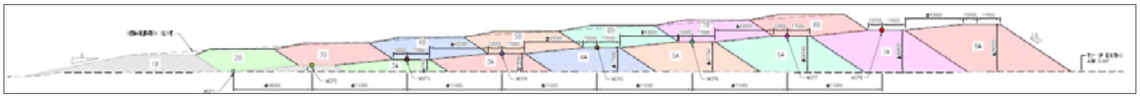


Figure 6-4: Stages for the conveyor positions on the ramp up section

Figure 6-5 below indicates the typical section of the deposition on the lower stack.

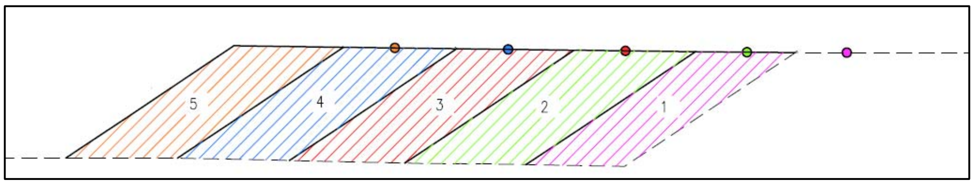


Figure 6-5: Stages for the lower ash dump extension

6.4 GROWTH DEVELOPMENT PLAN

This section defines the growth development plan for the Ash Dump Facility for the remaining 36-year life of Matimba Power Station for the period 2019 - 2055.

It must be noted for reference purposes that the movement of the main conveyor system should occur during September 2019.

6.4.1 GROWTH DEVELOPMENT PLAN STRATEGY

The ash deposition over the remaining 36-year life of the Ash Dump Facility is split into nine (9) phases i.e. phase 0 to phase 8. The growth development plan is based on the most recent Lidar survey taken on 14 April 2019 and does not consider the subsequent ashing activities on the Ash Dump Facility post survey.

6.4.1.1 PHASE 0

This phase involves deposition in the northern exemption area, southern exemption area and the existing facility ramp up area (piggyback). The deposition strategy for this phase has been defined as follows:

- Standby system deposition in the northern exemption Area at a 70 % deposition rate for an approximate period of five (5) months, combined with the main system deposition in the additional back stack and 1:20 ramp area at a 30 % deposition rate for an approximate period of five (5) months

- Standby system deposition in the northern exemption Area at a 100% deposition rate for an approximate period of 3.7 months. The main system must be relocated to the existing facility ramp up area position 1 and must be commissioned within this period.
- Main system deposition in the existing facility ramp up area at a 100% deposition rate for an approximate period of 2.5 months. The standby system must be relocated to the southern exemption area within this period.
- Main system deposition in the existing facility ramp up area and existing facility area at a 70 % deposition rate for an approximate period of 47 months, combined with the standby system deposition in the southern exemption area at a deposition rate of 30 % for an approximate period of 47 months until the exemption line is reached.
- At this point phase 0 will be completed with the main system located at position seven (MCP7) and the standby system located at the exemption line.
- Phase 1 is then initiated.

6.4.1.2 PHASE 1

This phase involves deposition in the lower radial front stack area (lined area) and existing facility area. A four-year lined area is required to be constructed for this phase. Construction of this phase must be completed and commissioned before the end of phase 0 to ensure that continuous ashing can take place on the facility. The deposition strategy for this phase has been defined as follows:

- Main system deposition on the existing facility area at a 70 % deposition rate for the duration of phase 1 (4-years).
- Standby system deposition on the lower radial front stack area (lined area) at a 30 % deposition rate for the duration of phase 1 (4-years).
- At the end of phase 1 the main system will be located at position 23 (MCP23) and the standby system will be located at position nine (SCP9)

6.4.1.3 PHASE 2 TO PHASE 8

These phases involve deposition in the lower radial front stack area (lined area) and on the upper existing facility area. Each phase will require a 4-year lined area to be constructed ahead of time as the Ash Dump advances. The ash deposition splits for phase 2 to phase 8 have been calculated and set to ensure that there is sufficient space between the main system and the standby system which ensures that these two (2) systems do not intersect each other. The deposition strategy for these phases have been defined as follows:

- Main System deposition on the existing facility area at a 60% deposition rate for the duration of each phase (4-years)
- Standby system deposition on the lower radial front stack area (lined area) at a 40% deposition rate for the duration of each phase (4-years)

6.4.1.4 SUMMARY

Drawings 301-00825/01-200 to 301-00825/01-820 in **Appendix A** indicates the phases for the development of the ash dump facility for the 4-year and 64-year Ash dump development. The various phases have been summarized in the tables below which indicate the starting stage and start date for each phase as well as the end stage and end date for each phase.

The detailed growth development plan is included in **Appendix E.**

Table 6-4: Phase 0 Summary

	Start Stage	Conveyor Position	Start Date	End Stage	Conveyor Position	End Date
Main System	MSBS + Ramp Up	-	April 2019	7B	MCP7	April 2024
Standby System	1A (North)	SCP1N	April 2019	9A (South)	SCP9S	April 2024

Table 6-5: Phase 1 Summary

	Start Stage	Conveyor Position	Start Date	End Stage	Conveyor Position	End Date
Main System	8A	MCP8	April 2024	23B	MCP23	December 2027
Standby System	1A	SCP1	April 2024	9A	SCP9	August 2028

Table 6-6: Phase 2 Summary

	Start Stage	Conveyor Position	Start Date	End Stage	Conveyor Position	End Date
Main System	24A	MCP24	December 2027	42B	MCP42	February 2032
Standby System	10A	SCP10	August 2028	18A	SCP18	February 2032

Table 6-7: Phase 3 Summary

	Start Stage	Conveyor Position	Start Date	End Stage	Conveyor Position	End Date
Main System	43A	MCP43	February 2032	55B	MCP55	January 2036
Standby System	19A	SCP19	February 2032	26A	SCP26	December 2035

Table 6-8: Phase 4 Summary

	Start Stage	Conveyor Position	Start Date	End Stage	Conveyor Position	End Date
Main System	56A	MCP56	January 2036	66A	MCP66	February 2040
Standby System	27A	SCP27	December 2035	33A	SCP33	November 2039

Table 6-9: Phase 5 Summary

	Start Stage	Conveyor Position	Start Date	End Stage	Conveyor Position	End Date
Main System	66B	MCP66	February 2040	71A	MCP71	January 2044
Standby System	34A	SCP34	November 2039	40A	SCP40	November 2043

Table 6-10: Phase 6 Summary

	Start Stage	Conveyor Position	Start Date	End Stage	Conveyor Position	End Date
Main System	71B	MCP71	January 2044	76A	MCP76	February 2048
Standby System	41A	SCP41	December 2044	48A	SCP48	January 2048

Table 6-11: Phase 7 Summary

	Start Stage	Conveyor Position	Start Date	End Stage	Conveyor Position	End Date
Main System	76B	MCP76	February 2048	80A	MCP80	April 2051
Standby System	49A	SCP49	January 2048	54A	SCP54	February 2051

Table 6-12: Phase 8 Summary

	Start Stage	Conveyor Position	Start Date	End Stage	Conveyor Position	End Date
Main System	80B	MCP80	April 2051	85B	MCP85	June 2055
Standby System	55A	SCP55	February 2051	60A	SCP60	February 2054

6.4.2 GROWTH DEVELOPMENT CURVES

6.4.2.1 PHASE 0 GROWTH DEVELOPMENT CURVES

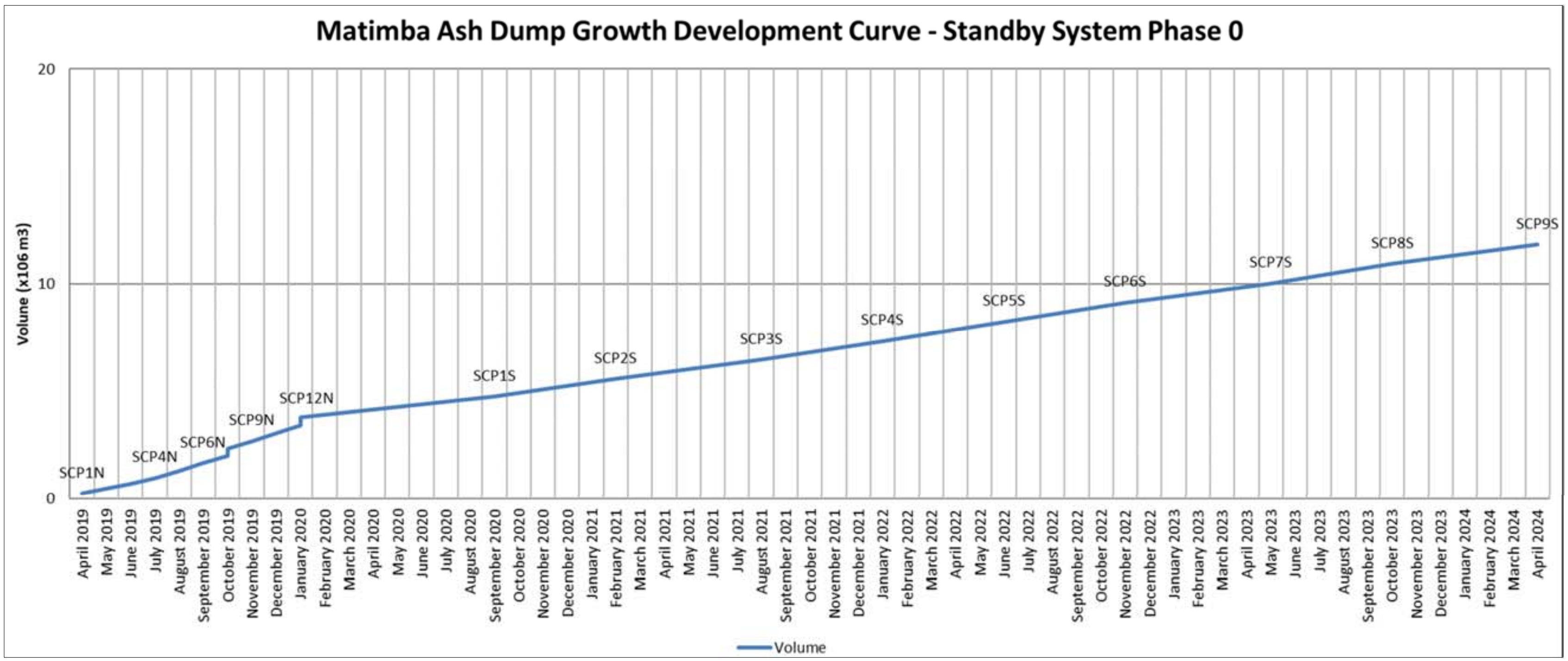


Figure 6-6: Matimba Ash Dump Growth Development Curve – Standby System Phase 0

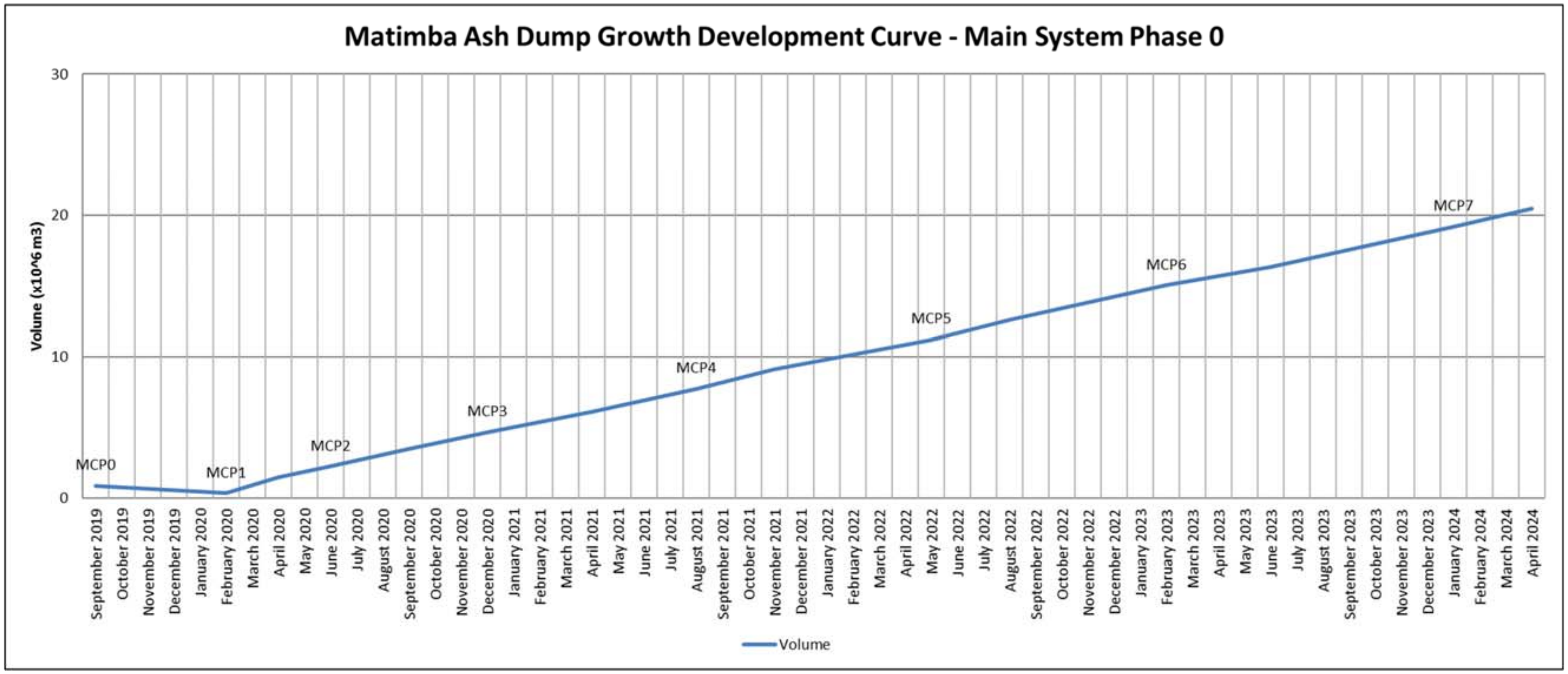


Figure 6-7: Matimba Ash Dump Growth Development Curve – Main System Phase 0

6.4.2.2 PHASE 1 (4-YEAR) GROWTH DEVELOPMENT CURVES

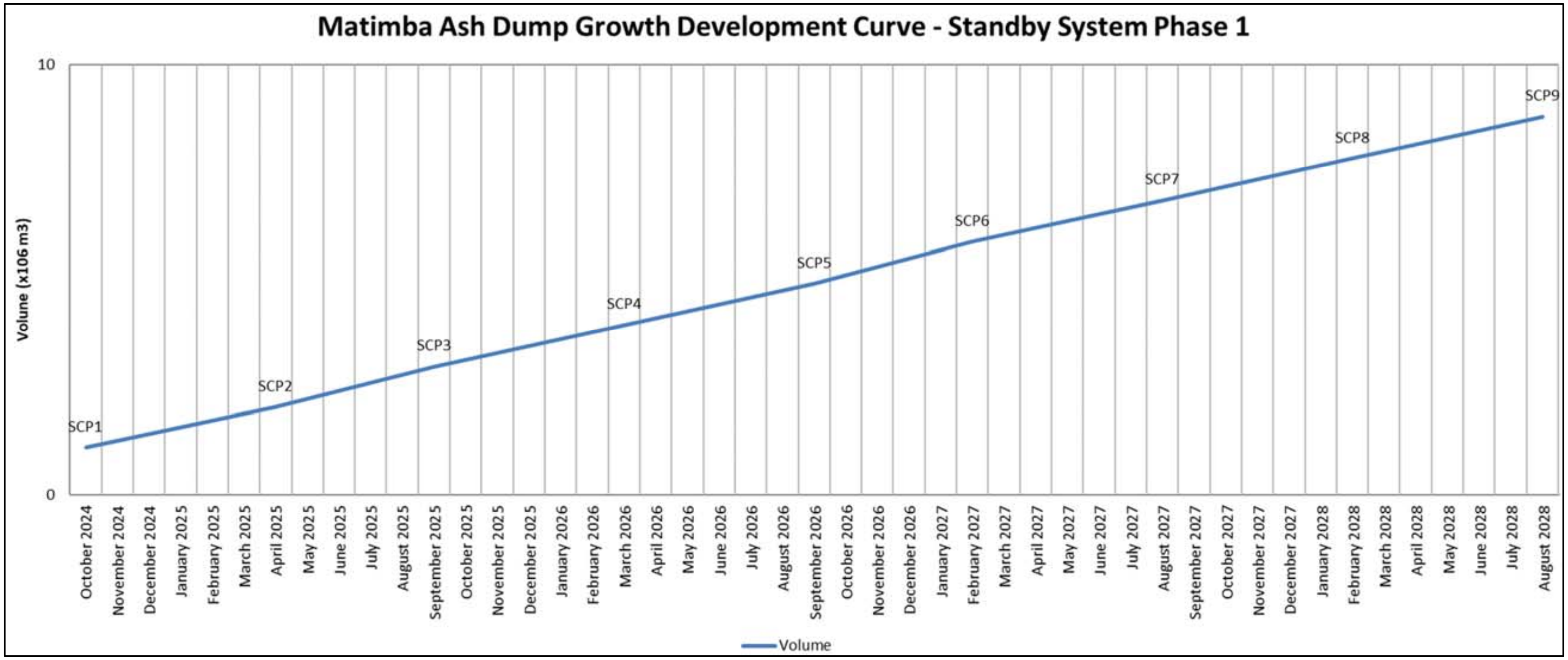


Figure 6-8: Matimba Ash Dump Growth Development Curve – Standby System Phase 1

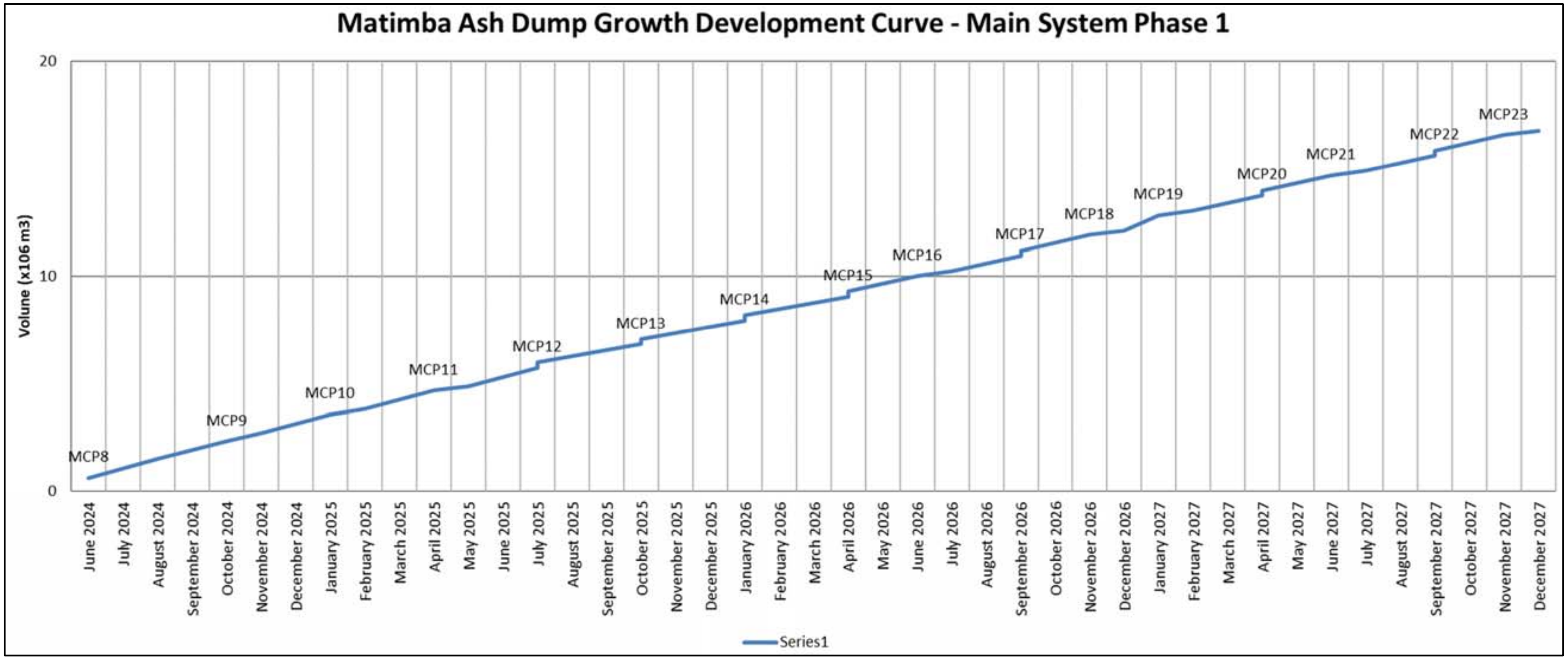


Figure 6-9: Matimba Ash Dump Growth Development Curve – Main System Phase 1

6.4.2.3 60-YEAR GROWTH DEVELOPMENT CURVES

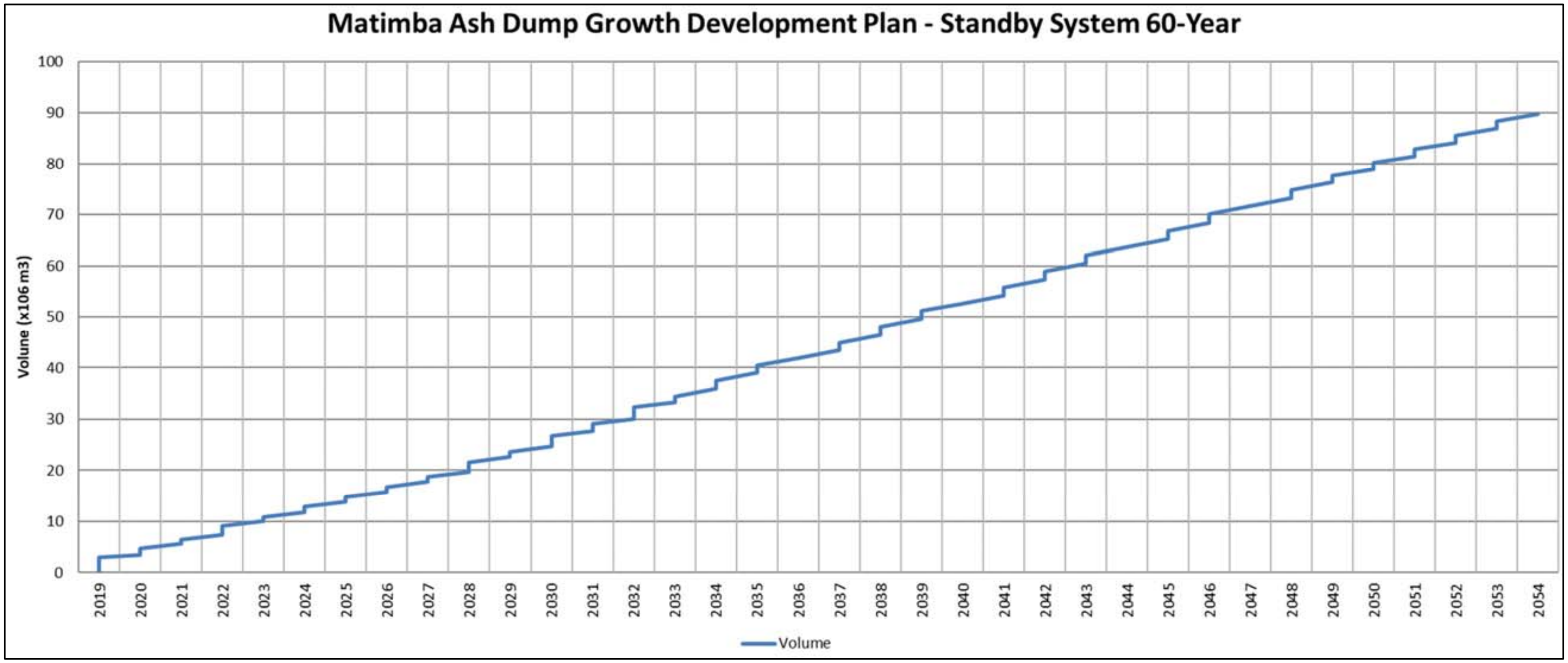


Figure 6-10: Matimba Ash Dump Growth Development Curve – Standby System 60-Year

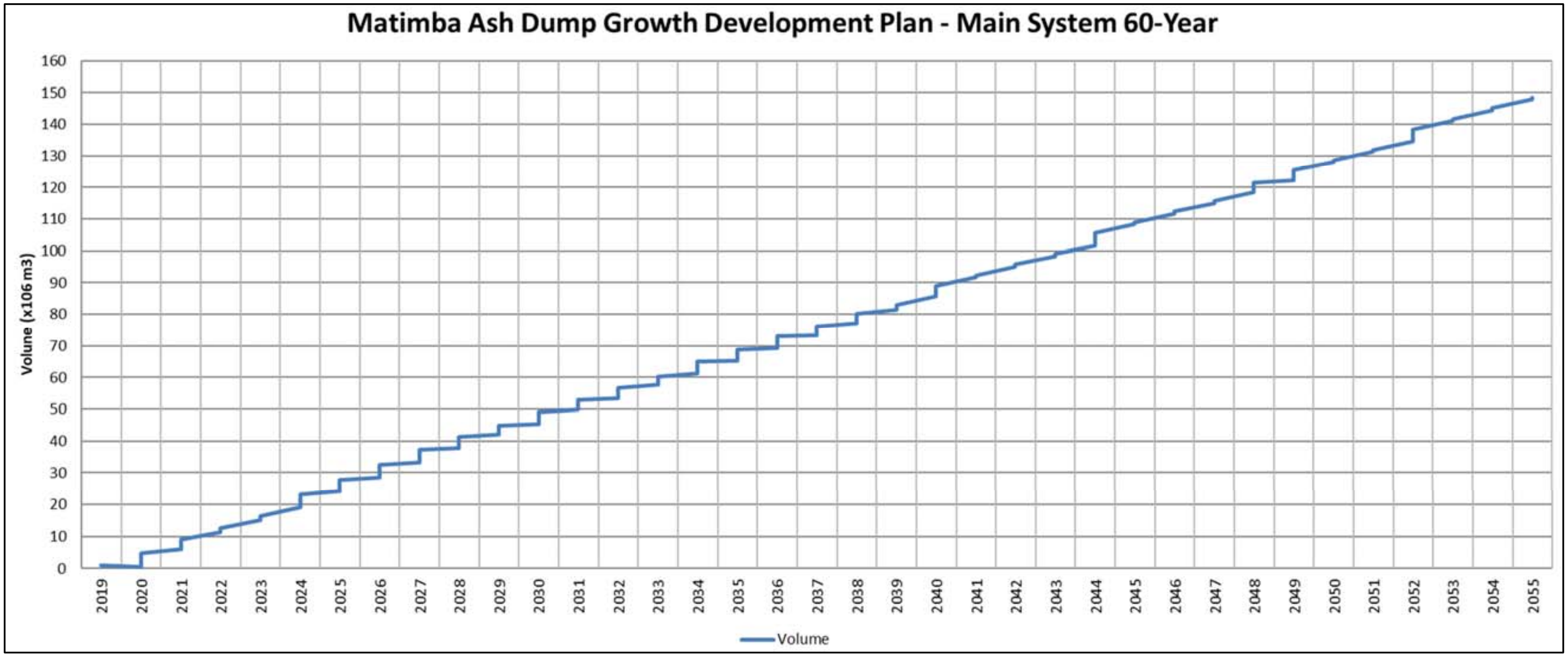


Figure 6-11: Matimba Ash Dump Growth Development Curve – Main System 60-Year

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Section 6.1 indicates the procurement & construction dates for the ash dump facility. The Construction works should be complete by March 2024 to allow to begin ashing in April 2024. The complete Ash dump development and construction dates for all the phases are indicated in Section 6.4.

6.5 CONVEYOR PROFILES

For each phase, conveyor profiles have been illustrated and are visible in Appendix A.

In summary the following conveyor profiles have been draughted for the Ash dump facility:

- The first position
    - Main conveyor – (Drawing Number: 301-00825/01-117)
    - Standby conveyor - (Drawing Number: 301-00825/01-116)
    - Profiles: Drawing Number: 301-00825/01-163 and 301-00825/01-164
- The main conveyor at the top of the ramp up area on the existing facility (Drawing Number: 301-00825/01-162))
- The longest conveyor length
    - Main conveyor - (Drawing Number: 301-00825/01-813)
    - Standby conveyor - (Drawing Number: 301-00825/01-714)
    - Profiles: 301-00825/01-814 to 301-00825/01-816 and 301-00825/01-716
- Final Position (60 year)
    - Main conveyor - (Drawing Number: 301-00825/01-813)
    - Standby conveyor - (Drawing Number: 301-00825/01-813)
    - Profiles: 301-00825/01-814 to 301-00825/01-816

6.6 COST OF CONVEYORS

Table 6-13 summarises the cost of the conveyors for the ash dump extension.

Table 6-13: Conveyor costs

Standby System	
Shiftable Conveyor	
Maximum Length Required	2,441 m
Current Length Available	1,000 m
Conveyor Length to be Purchased	1,441 m
Cost/m	R65,000
Total Cost of New Conveyors	**R 93,665,000**
Extendable Conveyor	
Maximum Length	518 m
Current Length Available	518 m
Conveyor Length to be Purchased	0 m
Cost/m	R50,000
Total Cost of New Conveyors	**R 0**
Main System	
Shiftable Conveyor	
Maximum Length	2,158 m
Current Length Available	1,700 m
Conveyor Length to be Purchased	458 m
Cost/m	R65,000
Total Cost of New Conveyors	**R29,770,000**
Extendable Conveyor	
Maximum Length	1,216 m
Current Length Available	518 m
Conveyor Length to be Purchased	698 m
Cost/m	R50,000
Total Cost of New Conveyors	**R34,900,000**
Overall Total Cost	**R158,355,000**

7.0 WATER BALANCE

The water balance was developed to assist in the optimisation of the irrigation for dust suppression as well as to size the proposed storm water dams (specifically for the ash dump extension phases). The Water Balance investigation included components to be modelled such as water sources, water usage and losses to the system; this information was supplied by the personnel on site, this included:

- Documentation of operational philosophies;
- Documentation of User Requirements and Assumptions of relevant operations;
- Linkages and routes between components.

This study was undertaken with adherence to the relevant South African Best Practice Guidelines and Acts. The Water Balance update will be undertaken according to the Department of Water and Sanitation; DWS (previously Department of Water Affairs; DWA) Guidelines; Best Practice Guidelines (BPG) G2: Water and Salt Balances.

GN 704 and Regulation 77 of the National Water Act (Act 36 of 1988) stipulate the requirement in respect of use of water for mining and related activities aimed at the protection of water resources. This guideline stipulates that spillage of water from any storm water dam is not allowed except during extreme flood events that are, on average, exceeded no more than once in 50 years. This criterion was used for the sizing of the storm water dams.

The study commenced with a desktop assessment of the area of interest and included identification of existing data and literature pertaining to the area. Visio-based water Process Flow Diagrams (PFDs) were generated for the following scenarios (shown in **Figure 7-1** to **Figure 7-3**):

- Current existing Matimba Ash Dump Facility;
- 5-Year exemption period;
- Final Matimba Ash Dump Facility Footprint (full extension footprint).

It must be noted that the PFDs are not a water balance diagram. The PFD illustrates the connectivity of the various dams, water uses and demands as well as the areas of the catchments generating the storm water runoff. The final water balances will be based on the PFDs but will include volumes per month.

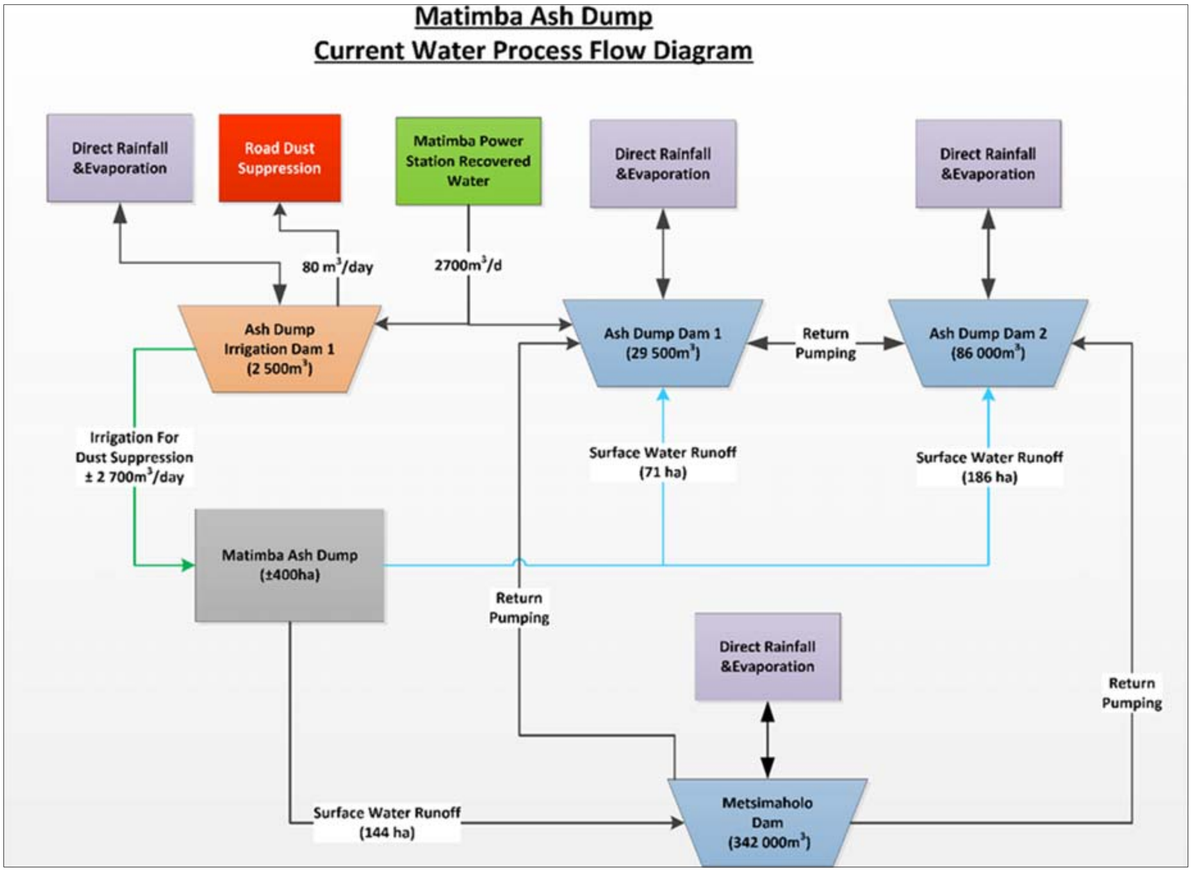

Figure 7-1: Current existing Matimba Ash Dump Facility PFD

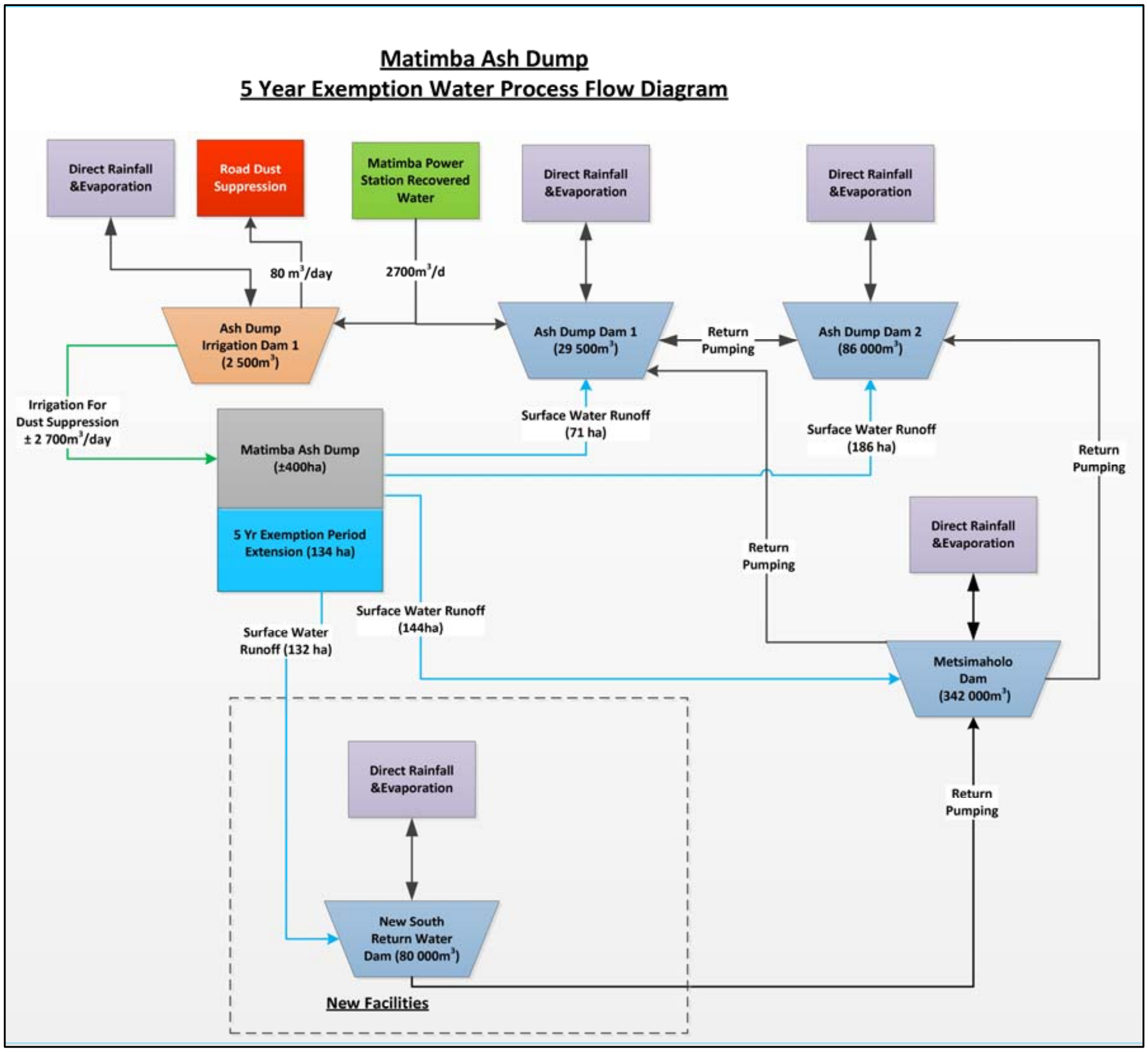

Figure 7-2: 5-Year exemption period PFD

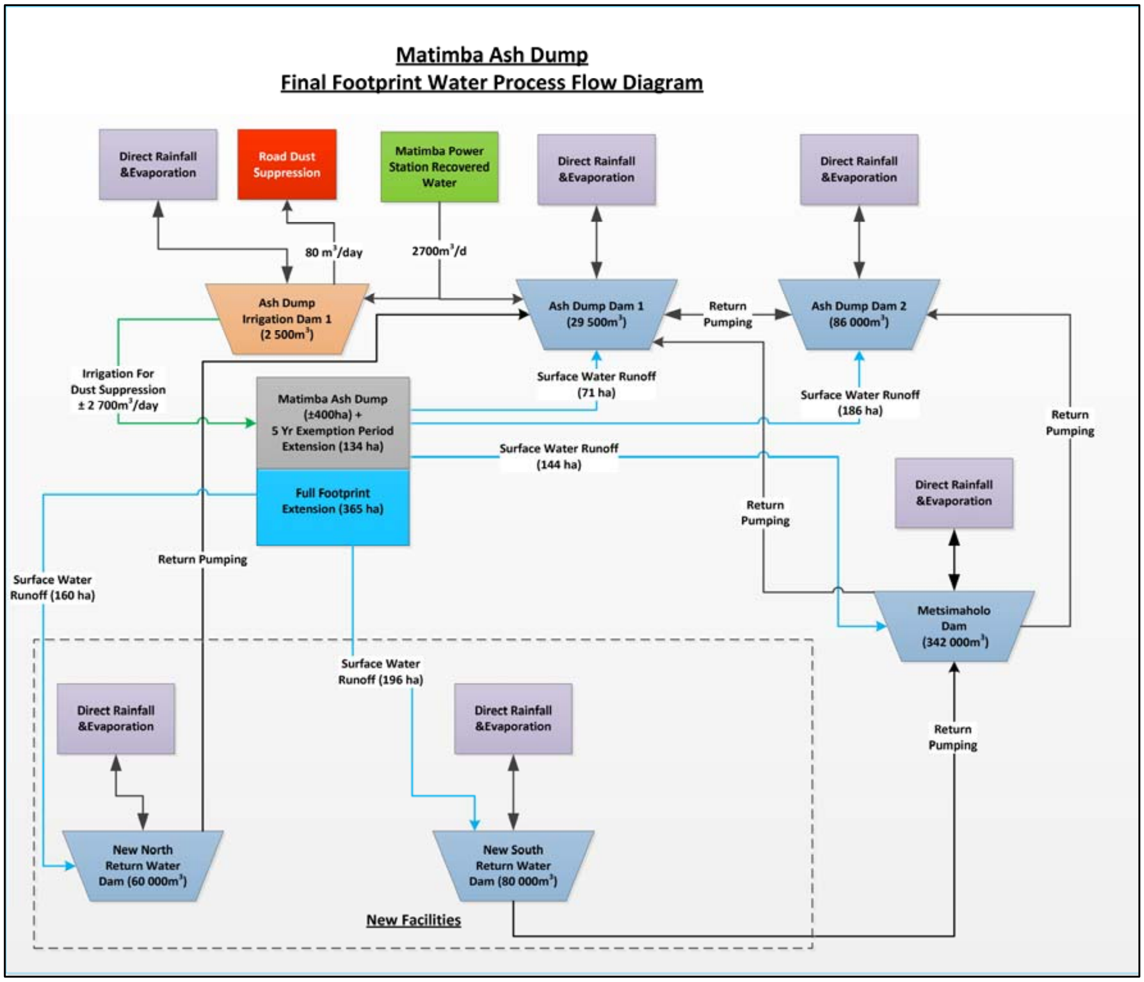

Figure 7-3: Final Matimba Ash Dump Facility (full extension footprint)

7.1 HYDROLOGY

The Matimba Ash Dump falls within the A42J quaternary catchment, the Mean Annual Precipitation (MAP) was found to be 428 mm. The Daily Rainfall Data Extraction Utility (ICFR, 2012) indicates a MAP for the nearest reliable rainfall station (providing good quality rain data) located at the Lephalale (Ellisras) Police Station of 455mm (SAWS 0674400_W).

The Daily Rainfall Data Extraction Utility was used to patch rainfall data for this station and create an extended daily rainfall record that covers at least fifty (50) years (see **Figure 7-4**) and is distributed as shown in **Figure 7-5**. This synthetic record indicates a MAP of 444 mm, which correlates well to the shorter record. The programme is widely used and accepted within the hydrology profession.

The Symons Pan, Mean Annual Evaporation (MAE) of 1949mm, reported in WR2012 (WRC, 2012), and the average monthly distribution thereof, was accepted (**Figure 7-6**).

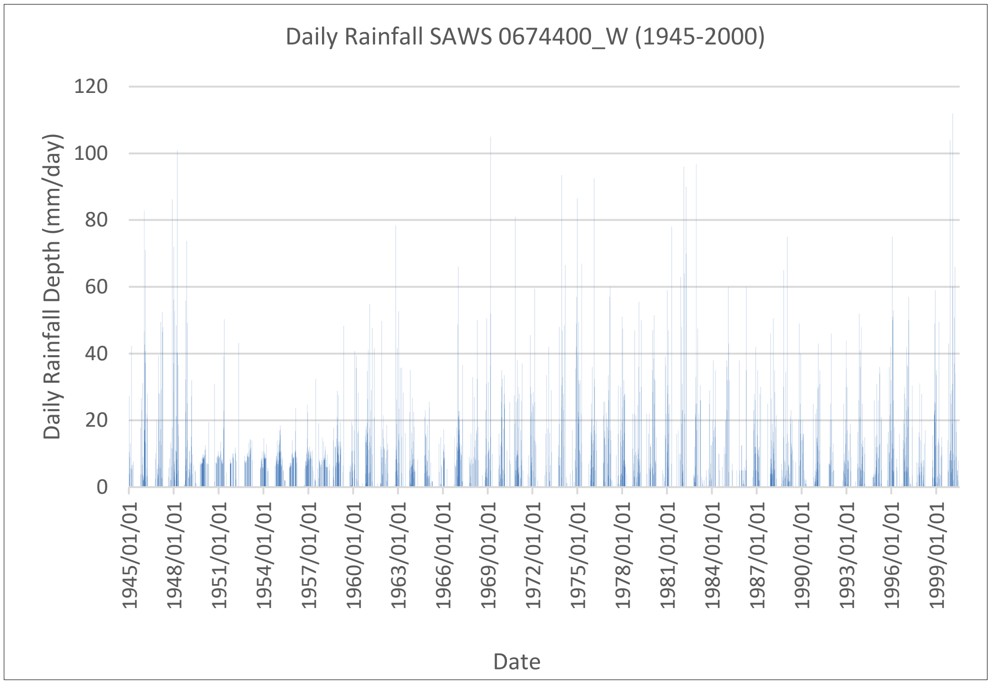

Figure 7-4: Daily rainfall depths

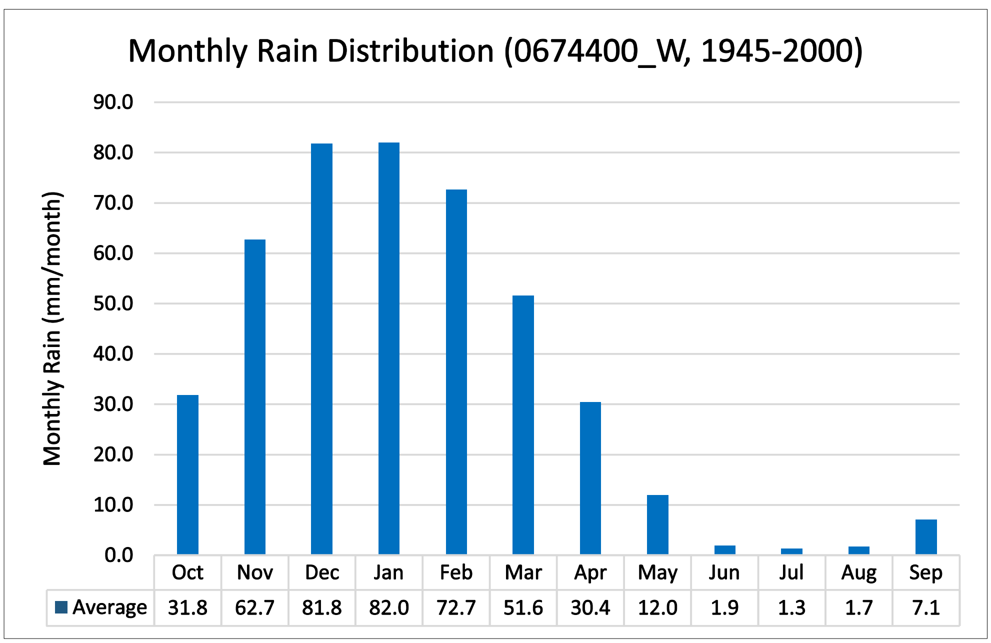

Figure 7-5: Average monthly rainfall distribution

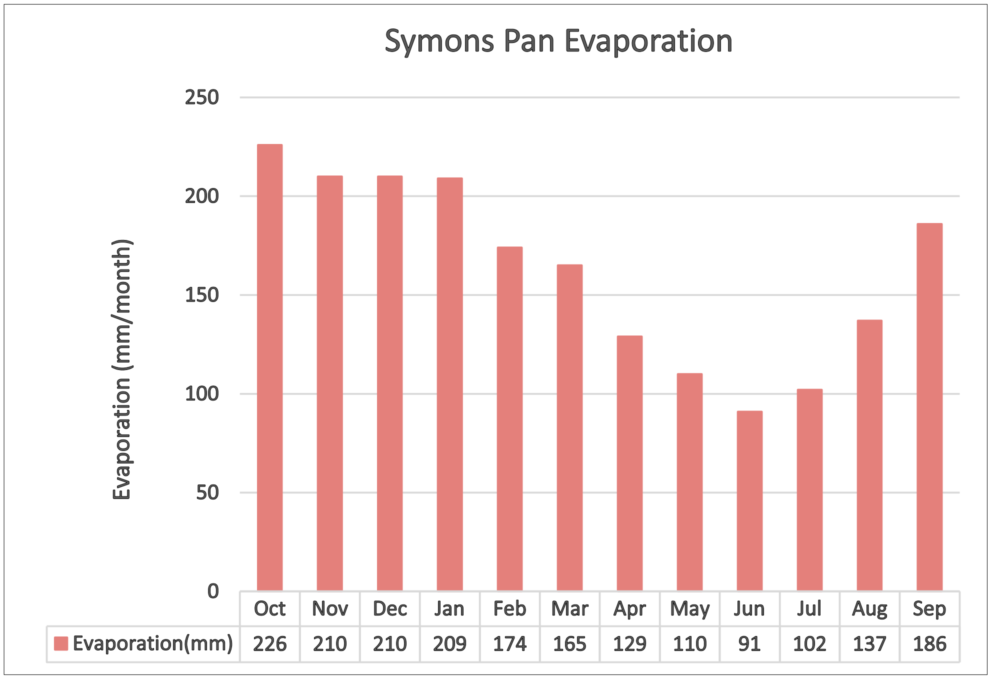

Figure 7-6: Monthly evaporation distribution (Symons Pan)

7.2 EXISTING DAM DATA

The capacities and areas of the various impoundments are indicated in **Table 7-1**. The capacities were obtained from the Dams Design Information Report (LBCE, 2011) and the Hydrological Assessment Update (RHDHV, 2016).

Table 7-1: Existing Dam capacities

Dam	Maximum Capacity (m^3)	Maximum Area (m^2)
Ash Dam 1	29,500	14,400
Ash Dam 2	86,000	28,600
Metsimaholo Dam	342,000	88,600
Ash Dump Irrigation Dam	2,500	1,225

7.3 PUMP DATA

The following section describes the existing pumping system on site:

7.3.1 EXISTING IRRIGATION DAM PUMP STATION

A total of five pumps are located in the pump station as shown in **Figure 7-7**. The water is received from Ash Dump Dam 1 via a 600 mm pipeline. The water is then pumped to the irrigation sprinklers as follows:

- Two WKLn 80/3 pumps with a pumping capacity of 35 l/s (115 m Head and via a 150 mm diameter pipeline);
- Two WKLn 65/3 pumps with a pumping capacity of 17.5 l/s (115 m Head and via a 100 mm diameter pipeline);
- One WKLn 50/4 pump with a pumping capacity of 8.75 l/s (115 m Head and via an 80 mm diameter pipeline).

Figure 7-7: Photograph of the Irrigation Pond Pump Station

7.3.2 METSIMAHOLO PUMP STATION

A single pump system is located at Metsimaholo Dam as shown in **Figure 7-8**. The system is comprised of a suction pump (T3A3S-B with a pumping capacity of 22 l/s and a 18m head), concrete lined sump (with a gate valve) and a 150 mm diameter pipeline which transfers the water to Ash Dump Dam 2.

Figure 7-8: Metsimaholo Pump Station

7.3.3 ASH DUMP DAM 2 PUMP STATION

A single pump system is located at Ash Dump Dam 2 as shown in **Figure 7-9**. The system is comprised of a suction pump (T3A3S-B with a pumping capacity of 22 l/s and a 18m head), concrete lined sump (with a gate valve) and a 150 mm diameter pipeline which transfers the water to Ash Dump Dam 1.

Figure 7-9: Ash Dump Dam 2 Pump Station

7.3.4 ASSESSMENT OF IRRIGATION POND PUMPING CAPACITY

The Irrigation Pond Pump Station capacity was assessed to determine if there is sufficient capacity to deliver the irrigation water to the proposed Ash Dump extension. The proposed future elevations were obtained and are shown in **Table 7-2** below.

Table 7-2: Proposed future crest elevations

Description	Elevation (MAMSL)	Static Head (m)
Max Projected Crest Elevation of the Ash Dump (Max Ramp section height)	960	88.5
Maximum Elevation of 1st 4 Years	915	43.5
Maximum Elevation of 2nd 4 Years	920	48.5
Maximum Elevation at Base Layer	930	58.5
Maximum Elevation at Closure	980	108.5

The friction losses were calculated based on the current irrigation pipe size of 280 mm diameter and pipeline length of approximately 3,500 m and a projected length of 5,000 m. The calculated friction loss head(Hazen-Williams) is shown in **Table 7-3**.

Table 7-3: Pipe friction loss head

Pump Flow Options (l/s)	Pipeline Friction Head (m)	
	3500 m Pipeline	5000 m Pipeline
35	3.5	5
17.5	0.9	1.3
8.75	0.3	0.5

A sprinkler driving head of 28m was assumed. The total required pumping head for the proposed Ash Dump extension was calculated and compared (shown in **Table 7-4**) to the current available pumping head of 115m as described in Section 7.3.1 above. The analysis shows that the current pumping system is sufficient for all elevations except Max Projected Crest Elevation of the Ash Dump (Max Ramp section height) and the final closure phase. It is proposed that for the Max Projected Crest Elevation of the Ash Dump (Max Ramp section height) elevation the existing irrigation pump station capacity be increased for the additional 6.5 m head and for the final closure elevation a new pump station be constructed at the new South Return Water Dam.

Table 7-4: Total required pumping head

Description	Elevation (MAMSL)	Static Head (m)	Sprinkler Head (m)	Maximum Friction Head (m)	Total required Pumping Head (m)
Max Projected Crest Elevation of the Ash Dump (Max Ramp section height)	960	88.5	28	5	**121.5**
Maximum Elevation of 1st 4 Years	915	43.5	28	5	76.5
Maximum Elevation of 2nd 4 Years	920	48.5	28	5	81.5
Maximum Elevation at Base Layer	930	58.5	28	5	91.5
Maximum Elevation at Closure	980	108.5	28	5	**141.5**

7.3.5 PROPOSED DUST SUPPRESSION AND IRRIGATION SYSTEM

7.3.5.1 DUST SUPPRESSION REQUIREMENTS

The following information was extracted from the water balance as described in Section 7.
For dust suppression the areas per phase are shown in **Table 7-5** below:

Table 7-5: Dust Suppression areas per phase (lower stack) (excluding dust control soil layer) area

Phase	Dust Suppression Area Data (m^2)
Phase 1	139,385
Phase 2	150,754
Phase 3	169,837
Phase 4	177,401
Phase 5	181,752
Phase 6	205,931
Phase 7	151,863
Phase 8	144,901
Maximum	205,931

Note: The Phase areas assume that both the top surface (excluding the dust control soil layer area) and the advancing slope face will be sprinkled for dust suppression. Distribution pipe sizes are also based on this assumption.

The worst-case scenario for the dust suppression area requirements in phase 6 which yields 205,931 m^2. This is further proposed to be done in 2 shifts, each of 4 hours.

7.3.5.2 DUST SUPPRESSION SPRINKLER SELECTION

Currently Eskom's fleet sprinkler is the VRYSA86 Sprinkler. This sprinkler is used at Matimba and it is therefore recommended to maintain the sprinkler for standardisation. It also will aid maintenance and operations staff due to familiarity.

Based on the VRYSA86 Sprinkler or equivalent, the specifications are shown in **Table 7-6** below:

Table 7-6: Sprinkler specifications

Pressure (Bar)	Flow Rate (l/h)	Coverage (D) (Øm)
2.8	1.998	36.8
3.15	2.134	37.4
3.5	2.240	38
4.2	2.470	39.2
4.55	2.588	39.8
5.6	2.860	42.6

Number of sprinklers are selected based on the following formula, the calculated number of sprinklers per phase is shown in **Table 7-7**:

$$Number\ of\ sprinklers = \frac{(Area)}{(D/2\sqrt{2})^2}$$

Table 7-7: Required number of sprinklers per phase (lower stack)

Phase	No. of sprinklers Required
Phase 1	823
Phase 2	891
Phase 3	1,003
Phase 4	1,048
Phase 5	1,074
Phase 6	1,217
Phase 7	897
Phase 8	856

Using sprinkler D= 36.8m and worst-case dust suppression as 205,931 m^2. Results in 1217 sprinklers in total being required.

7.3.5.3 DUST SUPPRESSION SPRINKLER LAYOUT CONFIGURATION

A configuration layout for dust suppression is required such that the facility has the ability to create zones that can easily be operated without the operator having to move sprinklers. Each zone is to be resprayed after 7 days. It is recommended that 2x4 and a 4x2 configuration is used as shown in **Figure 7-10**: Sprinkler configuration**.** Sprinklers are attached via 25mm hose pipe of not more than 30m in length that is attached by means of a saddle to quick couple irrigation piping. It is envisaged that the quick couple piping is 70mm. T- Pieces attach the quick couple piping to the ring main.

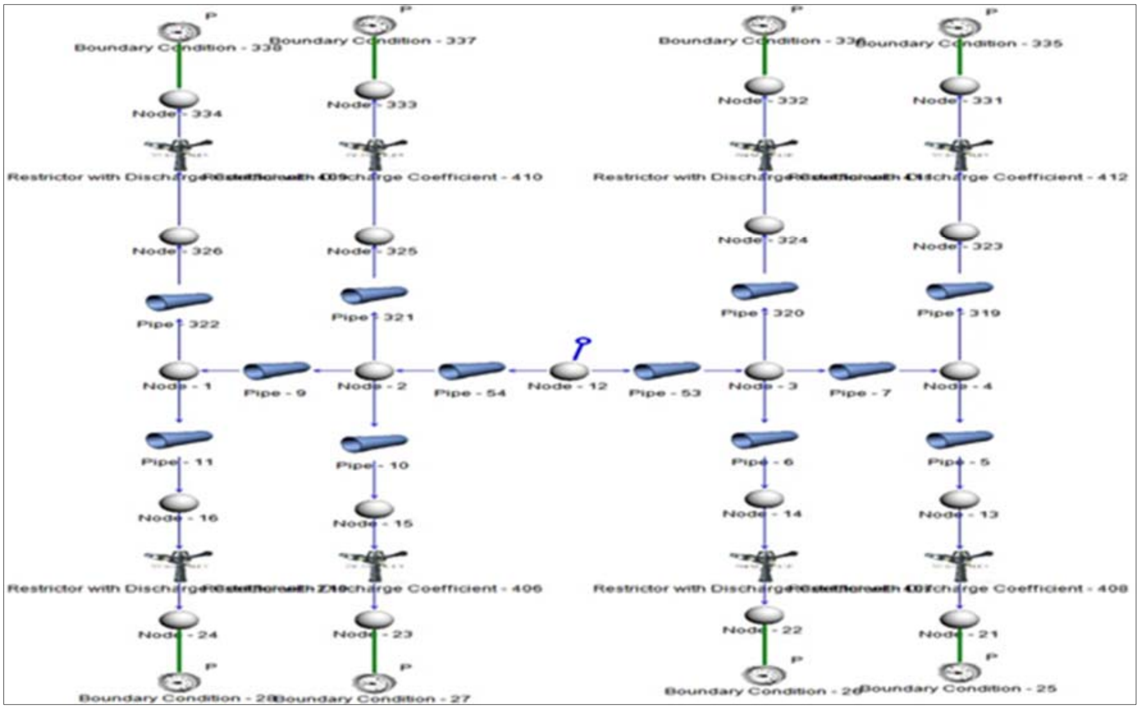

Figure 7-10: Sprinkler configuration

7.3.5.4 DUST SUPPRESSION RING MAIN CRITERIA

To dust suppress the top and advancing faces of each phase, a total daily flow of 12* 110 m^3/hr will be required. This must be delivered in two half-day periods so as to limit the feed pipe size to 280mm diameter. The pipe pressure rating required for dust suppression is a minimum PN20.

7.3.5.5 PUMP SELECTION

A centrifugal multi-stage pump (KSB WKLn 80) was selected to be installed in the existing irrigation pumphouse to provide the required additional head for the final top dump elevation, with the following information

Number of Pumps	2
Number of Pumps on Stand-by	1
Flow Per Pump	110 m^3/hr
Head	150 m
NPSHr	2.5 m
Impeller Diameter	210 mm
Speed	2900 rpm
Stages	3
Efficiency	72.6%
Power	62 kW
Motor Size	75 kW
Drive	Variable Speed Drive

See **Appendix H** for Pump Curves.

A duplex strainer is to be fitted to ensure water does not contain particles that may cause blockage. Envisaged Pipe size is a nominal size of 280mm; the route of the pipeline is to be determined by the contactor for optimal efficiency.

Valves utilised are:

- Butterfly Valves for isolation
- Non – Return Valves for Pump Protection
- Air Release Valves
- Ball/Gate/Butterfly Valves for zoning

7.3.5.6 IRRIGATION REQUIREMENTS (TOP STACK CLOSURE CONDITION)

Note: The Phase areas for irrigation of the final dump closure surface assume that both the top surface and the dressed external perimeter slopes will be sprinkled for irrigation after topsoiling/grassing. Once the vegetation has been placed irrigation will commence, it is assumed that the irrigation will stop and move to the next phase once the rehabilitation is complete. Distribution pipe sizes are also based on this assumption.

Table 7-8: Irrigation areas per phase

Phase	Irrigation Area Data (m^2)
Phase 1	426,561
Phase 2	371,273
Phase 3	286,620
Phase 4	295,963
Phase 5	295,593
Phase 6	312,576
Phase 7	237,559
Phase 8	297,816
Maximum	426,561

The worst-case scenario for the irrigation area requirements in phase 1 which gives 426,561 m^2.

7.3.5.7 IRRIGATION SPRINKLER SELECTION

Currently Eskom's fleet sprinkler is the VRYSA86 Sprinkler. This sprinkler is used at Matimba and it is therefore recommended to maintain the sprinkler for standardisation. It also will aid maintenance and operations staff due to familiarity.

Based on the VRYSA86 Sprinkler or equivalent the specifications are shown in **Table 7-9** below:

Table 7-9: Sprinkler specifications

Pressure (Bar)	Flow Rate (l/h)	Coverage (D) (Øm)
2.8	1.998	36.8
3.15	2.134	37.4
3.5	2.240	38
4.2	2.470	39.2
4.55	2.588	39.8
5.6	2.860	42.6

Number of sprinklers are selected based on the following formula, the calculated number of sprinklers per phase is shown in **Table 7-10**:

$$Number\ of\ sprinklers = \frac{(Area)}{(D/2\sqrt{2})^2}$$

Table 7-10: Required number of sprinklers per phase (Final top stack)

Phase	No. of sprinklers required
Phase 1	2520
Phase 2	2193
Phase 3	1693
Phase 4	1748
Phase 5	1746
Phase 6	1847
Phase 7	1403
Phase 8	1759

Using D = 36.8m and worst-case dust suppression as 426,561 m^2. Results in 2520 sprinklers in total being required.

7.3.5.8 IRRIGATION SPRINKLER LAYOUT

A configuration layout for irrigation is required such that the facility has the ability to create zones that can easily be operated without the operator having to move sprinklers. Each zone is to be resprayed after 7 days. It is recommended that 2x4 and a 4x2 configuration is used as shown in **Figure 7-11.** Sprinklers are attached via 25 mm hose pipe of not more than 30m in length that is attached by means of a saddle to quick couple irrigation piping. It is envisaged that the quick couple piping is 70 mm. T-Pieces attach the quick couple piping to the ring main.

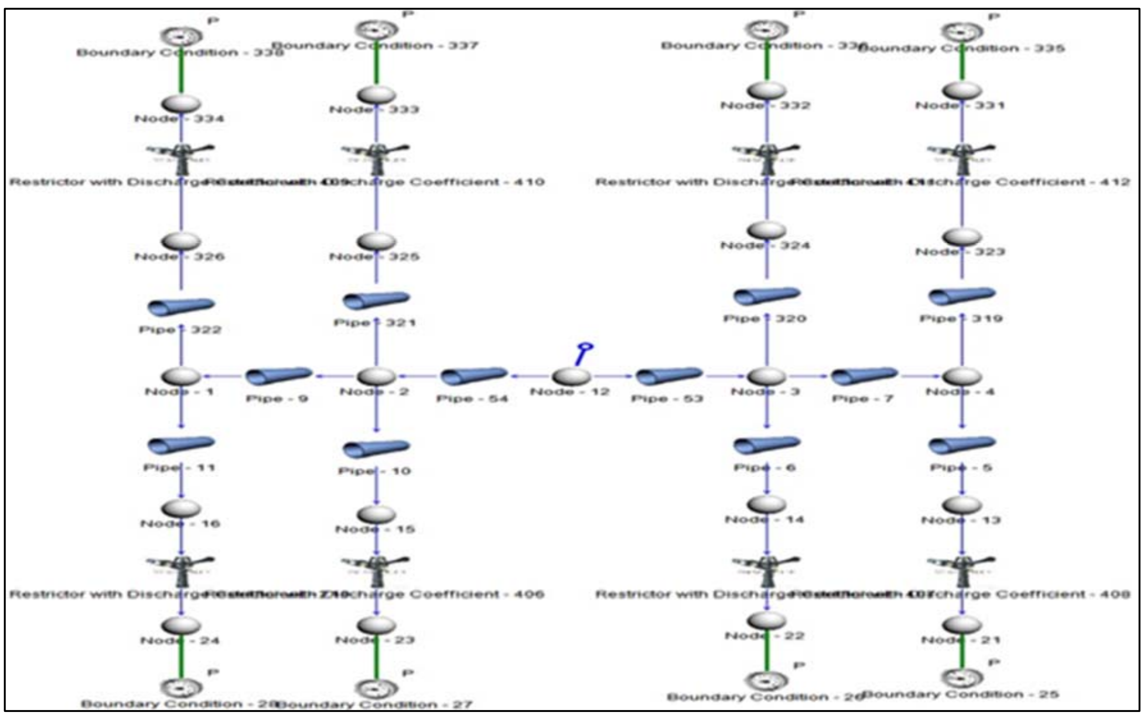

Figure 7-11: Sprinkler configuration

7.3.5.9 IRRIGATION RING MAIN SELECTION

The ring main needs to be sized to deliver a flow of 110 m^3/hr. This must be delivered in two half-day periods so as to limit the feed pipe size to 280mm diameter. The pipe pressure rating required for dust suppression is a minimum PN20.

7.3.5.10 COMBINED DUST SUPPRESSION AND IRRIGATION REQUIREMENTS

The dust suppression sprinklers for the lower stack will be utilised for the first 32 years of the ash dump extension. Only thereafter will the increased number of sprinklers be required for irrigation of the top-soiled/grassed surfaces of the final top stack.

It is therefore proposed that only the sprinklers required for dust control of the bottom stack, i.e. as per table 7.7 above, be purchased at this stage.

The top stack closure irrigation sprinkler requirement should then be re-assessed after completion of the bottom stack in 32 years time

7.3.6 OPERATIONAL PROCEDURES

The following sections describe the operational procedures for the various scenarios:

7.3.6.1 EXISTING MATIMBA ASH DUMP FACILITY SCENARIO

The operational procedures and pumping data for the current existing Matimba Ash Dump Facility scenario (PFD shown in **Figure 7-1**):

Irrigation for Dust Suppression and Irrigation:

- The Client provided measured records of daily flow rates for water recovered from the Matimba Power Station and irrigation on the Ash Dump Facility. Daily recovery rates record covers the period from 1st January 2018 until 12th May 2019. Irrigation usage was only measured between June 2015 and October 2015;
- Calculated monthly averages of daily recovery rates and irrigation on the Ash Dump Facility were used as input into the water balance model;
- Based on measured average volumes, a constant rate of 2,700 m^3/d was assumed for recovered water pumped into Ash Dump Dam 1, and
- Based on the measured record of irrigation for dust suppression (rehabilitation requirements are included in these flows) flow rates, a constant average daily flow rate for irrigation was calculated to be 400 m^3/d.
- Irrigation rates on the existing current ADF have been optimised to cover a total of 29 ha (information from the Client).
- Assumed application/irrigation rates on the ADF that depended on rainfall and irrigation to apply a maximum of 12 mm/day (information from Hydrological Assessment Report, RHDHV 2016).

This application rate allowed for a maximum irrigation rate of 3,480 m^3/d during dry days only if 29 ha is irrigated.

Ash Dump Dam 1 and Ash Dump Irrigation Dam:

- Monthly evaporation data from the surface of this existing dam was taken from the WR2012 database (WRC, 2012);
- Surface runoff from the ADF was taken as a function of daily rainfall input and monthly evaporation rates;
- Maximum road dust suppression rates on gravel roads to the ADF have been assumed at 80 m^3/d;
- Pumping from the Ash Dam 1 to Ash Dam 2 is initiated if the dam has more than 70% of water storage and the Ash Dam 2 can still accept water;
- A maximum pumping capacity to the Ash Dump Irrigation Dam (Irrigation Dam) was provided at 10 000 m^3/d. The actual pump rates will depend on water availability in the dam;
- Daily measured pump rates showed however that irrigation pump rates generally do not exceed 3 000 m^3/d and are on average varying between ~500 m^3/d and ~1,000 m^3/d. A more realistic maximum daily irrigation rate was assumed at 3,600 m^3/d. This is equal to a maximum unit irrigation rate of 9 mm/day applied over 40ha of exposed ash on the ash dump (measured surface area). There are 5 pumps located at the irrigation pond as described in Section 7.3.1, and

Ash Dump Dam 2:

- Pumping from existing Ash Dam 2 to existing Ash Dam 1 is initiated if Ash Dam 1 is less than 30% full and the Ash Dam 2 still has water;
- Pumping from the Ash Dam 2 to Metsimaholo Dam is initiated if Ash Dam 2 is more than 70 % full, the Metsimaholo Dam has the capacity and the Ash Dam 2 still has water, and
- No data was available on pump capacities or measured pump rates. A maximum daily pump rate was therefore assumed at 3,000 m^3/d.

Existing Metsimaholo Dam:

- Monthly evaporation data from the surface of this infrastructure were taken from the WR2012 database (WRC, 2012);
- Surface runoff from the ADF was taken as a function of daily rainfall input and monthly evaporation rates;
- Pumping from the Metsimaholo Dam to Ash Dam 2 is initiated if the Metsimaholo Dam has greater than 70% storage volume, and Ash Dam 2 has less storage volume than 70%, and
- No data was available on pump capacities or measured pump rates. A maximum daily pump rate was therefore assumed at 8,000 m^3/d.

7.3.6.2 FIVE-YEAR EXEMPTION PERIOD

The operational procedures and pumping data for the 5-year exemption period scenario (PFD shown in **Figure 7-2**) is described below. A new Return Water Dams (RWD) is proposed for this scenario, the RWD will be located to the north and south of the 5-year exemption period footprint.

Irrigation for Dust Suppression:

The dust suppression for irrigation requirements during the 5-year exemption period will be sourced from the existing Irrigation Dam Pump Station. The surface water runoff generated from the existing footprint will be captured and conveyed to the existing dams.

Ash Dump Existing Dam 1 and Existing Ash Dump Irrigation Dam:

The operational procedures and pumping data will remain the same as in the current existing scenario.

Ash Dump Existing Dam 2:

The operational procedures and pumping data will remain the same as in the current existing scenario.

Metsimaholo Dam:

The operational procedures and pumping data will remain the same as in the current existing scenario.

New South Return Water Dam:

- The Return Water Dam was sized as per GN704 i.e. the dam is not allowed to spill except during extreme flood events that are, on average, exceeded no more than once in 50 years. The RWD will receive runoff from the ash dump extension. The dams were modelled taking into account the dust suppression for irrigation demand as described above. The required dam capacity to meet this demand was found to be 80 000 m^3 (a 15% allowance for silting has been included in the capacity). Details regarding the sizing and design are described in Section 8.0.
- Pumping from the South Return Water Dam to Metsimaholo Dam is initiated if the dam has greater than 80% storage volume.
- One KSB 100-080-200-(or equal approved,) 126 m^3/hr, 12m head, pump, delivering into a 140OD PE100, PN6.3 butt welded HDPE pipe, will be required to meet the pumping requirements to Metsimaholo Dam (see 7.6.3 below for details) One pump arranged as per the existing Ash Dump 2 Pumpstation (Fig 7-9), will be located at the new pump/sump facility adjacent to the South RWD, the other kept in store.

7.3.6.3 FINAL MATIMBA ASH DUMP FACILITY FOOTPRINT PERIOD

The operational procedures and pumping data for the final Matimba Ash Dump Facility footprint (PFD shown in **Figure 7-3 (Section 7.0)**) is described below.

Water for Irrigation and Dust Suppression:

The dust suppression and irrigation requirements will be sourced from existing Ash Dump Irrigation Dam. The surface water runoff generated from the footprint will be captured and conveyed to the holding dams. Water for dust suppression and irrigation will then be pumped from the Holding Dams via a pipeline to the existing Ash Dump Irrigation Dam. The constant average daily flow rate for irrigation and dust suppression was calculated to be 5,600 m^3/d.

Existing Ash Dump Dam 1 and Ash Dump Irrigation Dam:

The operational procedures and pumping data will remain the same as in the current existing scenario.

Existing Ash Dump Dam 2:

The operational procedures and pumping data will remain the same as in the current existing scenario.

Existing Metsimaholo Dam:

The operational procedures and pumping data will remain the same as in the current existing scenario.

New North Return Water Dam:

- The required dam capacity is 60 000 m^3 for the New North Return Water Dam as described above.
- Pumping from the North Return Water Dam to Ash Dam 1 to is initiated if the dam has greater than 70% storage volume, and Ash Dam 1 has less storage volume than 70%.

One KSB Etanorm 100-080-200 (or equal approved)- 126 m^3/hr, 20 m head pump at 1800 rpm, 212mm dia impeller pump, arranged as per the existing Ash Dump 2 Pumpstation (Fig 7-9), delivering into a 180OD, PN100,PN6.3 HDPE pipe 2300m long, will be required to meet the pumping requirements to the existing Ash Dump Dam1.(A second similar pump will be delivered to store)(see Appendix H for pump curves)

New South Return Water Dam

- The required dam capacity is 80,000 m^3 for the New South Return Water Dam as described above.
- Pumping from the South Return Water Dam to Metsimaholo Dam to is initiated if the dam has greater than 80% storage volume, and Ash Dam 1 has less storage volume than 50%.
- One KSB Etanorm 100-080-160 (or equal approved)- 126 m^3/hr, 12 m head pump at 1800 rpm, 174mm dia impeller pump, arranged as per the existing Ash Dump 2 Pumpstation (Fig 7-9), delivering into a 140mm OD HDPE PE100 PN6.3 pipeline 300m long, routed at ground level around the existing spoil dump between the two dams, will be required to meet the pumping requirements to Metsimaholo Dam.(A second similar pump will be delivered to store)
- (See Appendix H for pump curves)

7.3.7 WATER BALANCE RESULTS

The above data and operational requirements will be used to develop a daily Excel based water balance for the abovementioned scenarios. The following water balance results are presented below:

- Average monthly water balance;
- Average wet season water balance; and
- Average dry season water balance.

7.3.7.1 AVERAGE MONTHLY WATER BALANCE

The average monthly water balances are shown below for the current existing system scenario (**Figure 7-12**), the 5-year exemption period scenario (**Figure 7-13**) and the final footprint scenario (**Figure 7-14**).

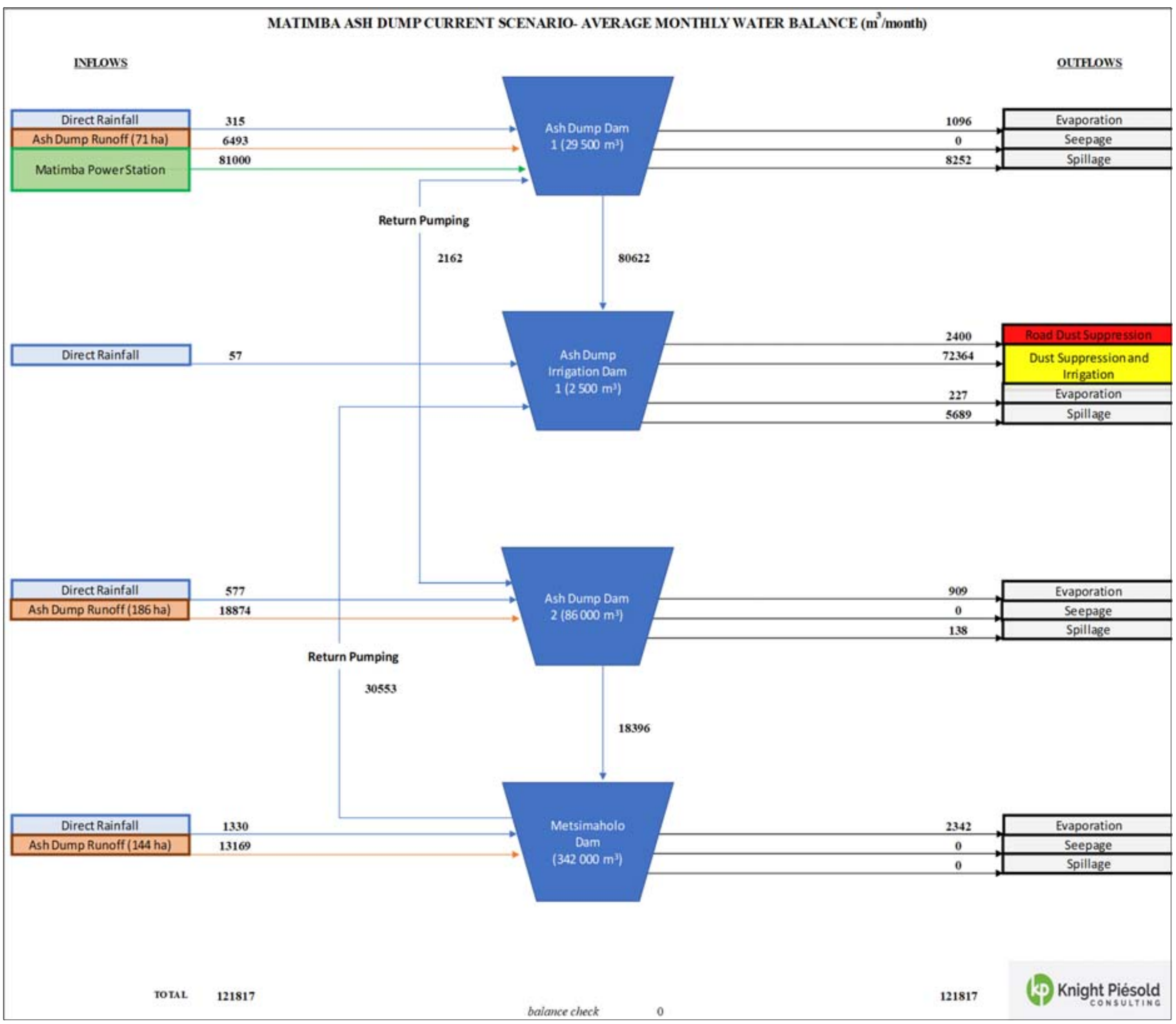

Figure 7-12: The average monthly water balance for the current existing system scenario

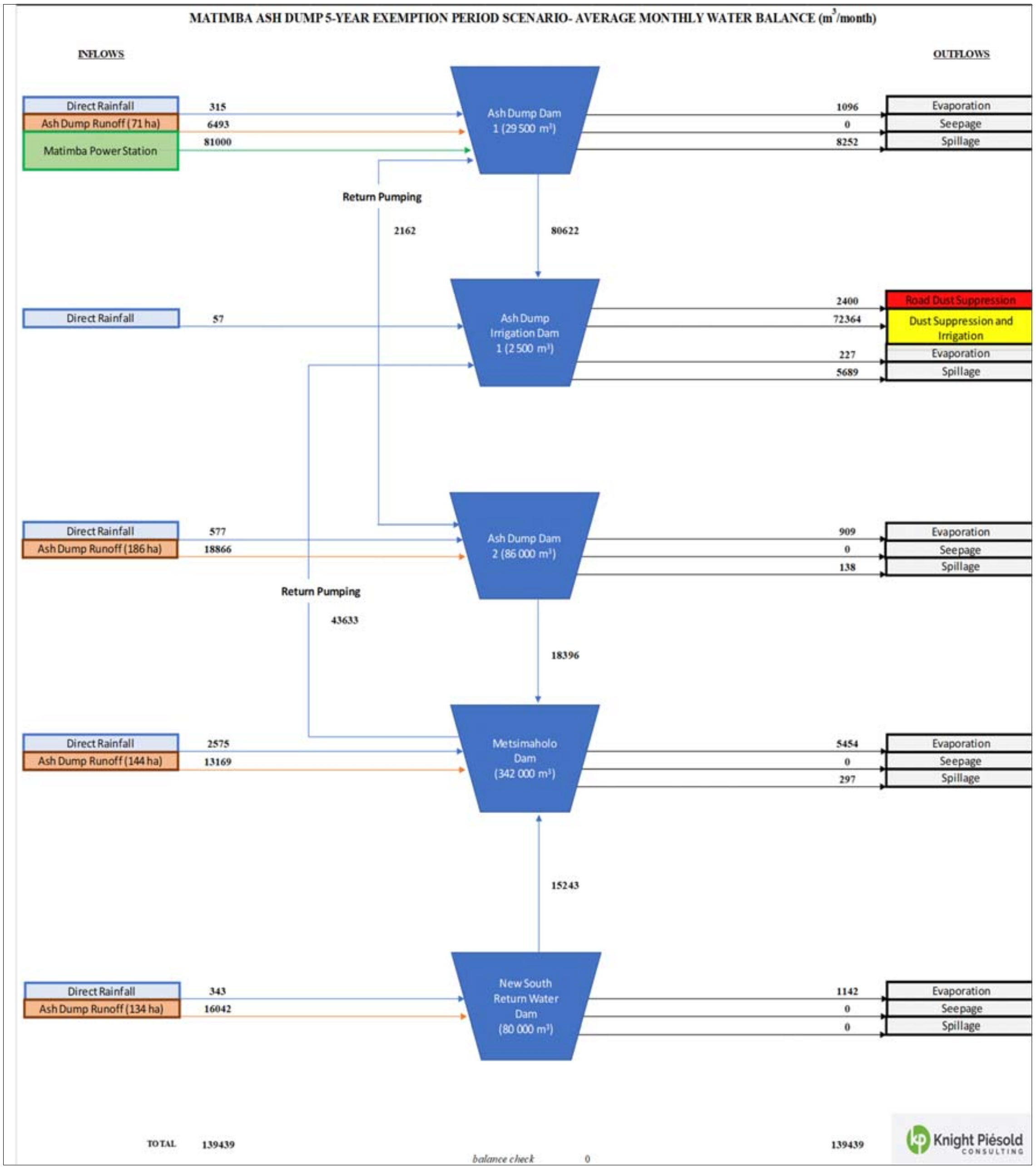

Figure 7-13: The average monthly water balance for the 5-year exemption period scenario

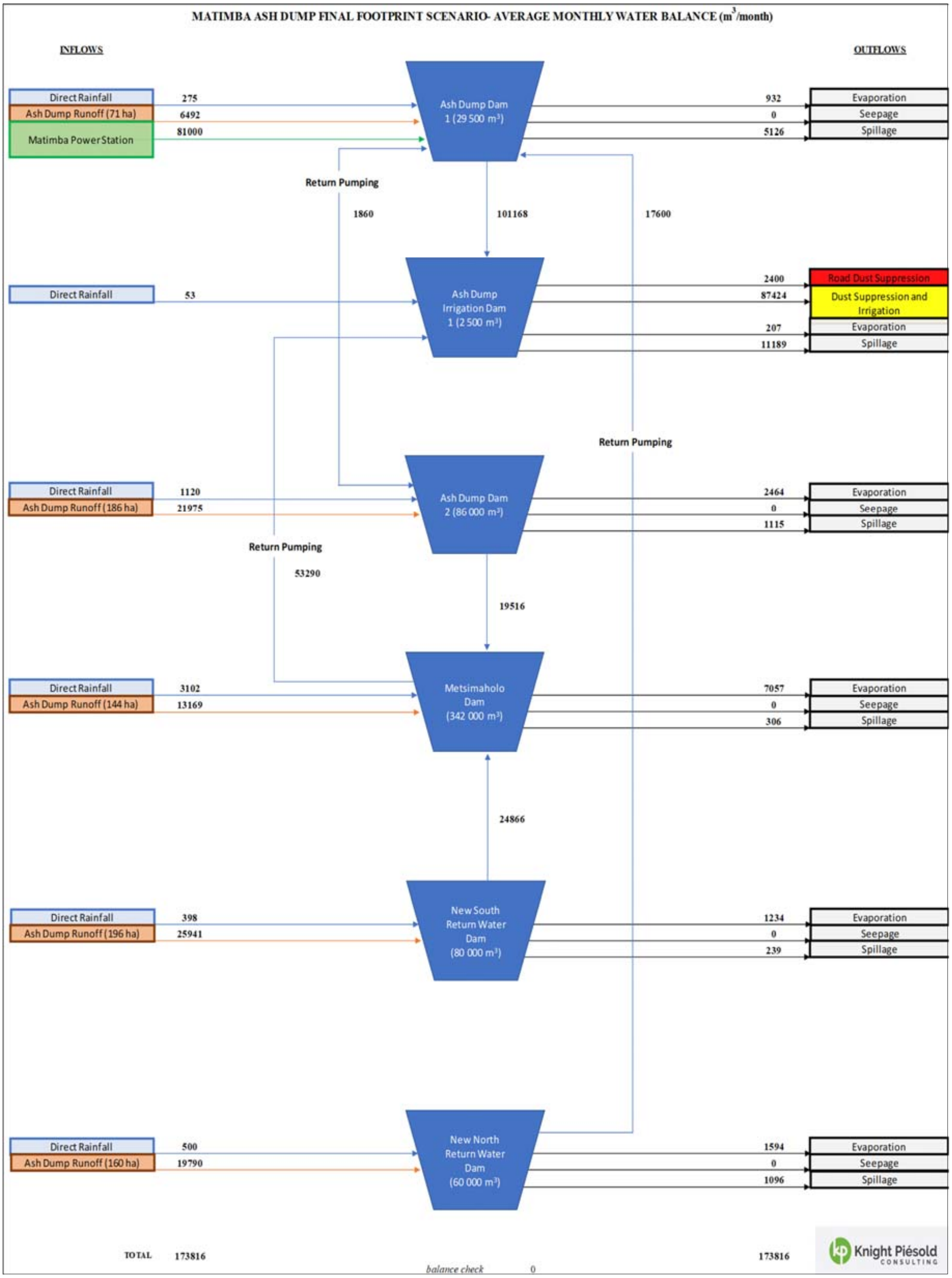

Figure 7-14: The average monthly water balance for the final footprint scenario

7.3.7.2 AVERAGE WET SEASON WATER BALANCE

The average monthly wet season water balances are shown below for the current existing system scenario (**Figure 7-15**), the 5-year exemption period scenario (**Figure 7-16**) and the final footprint scenario (**Figure 7-17**).

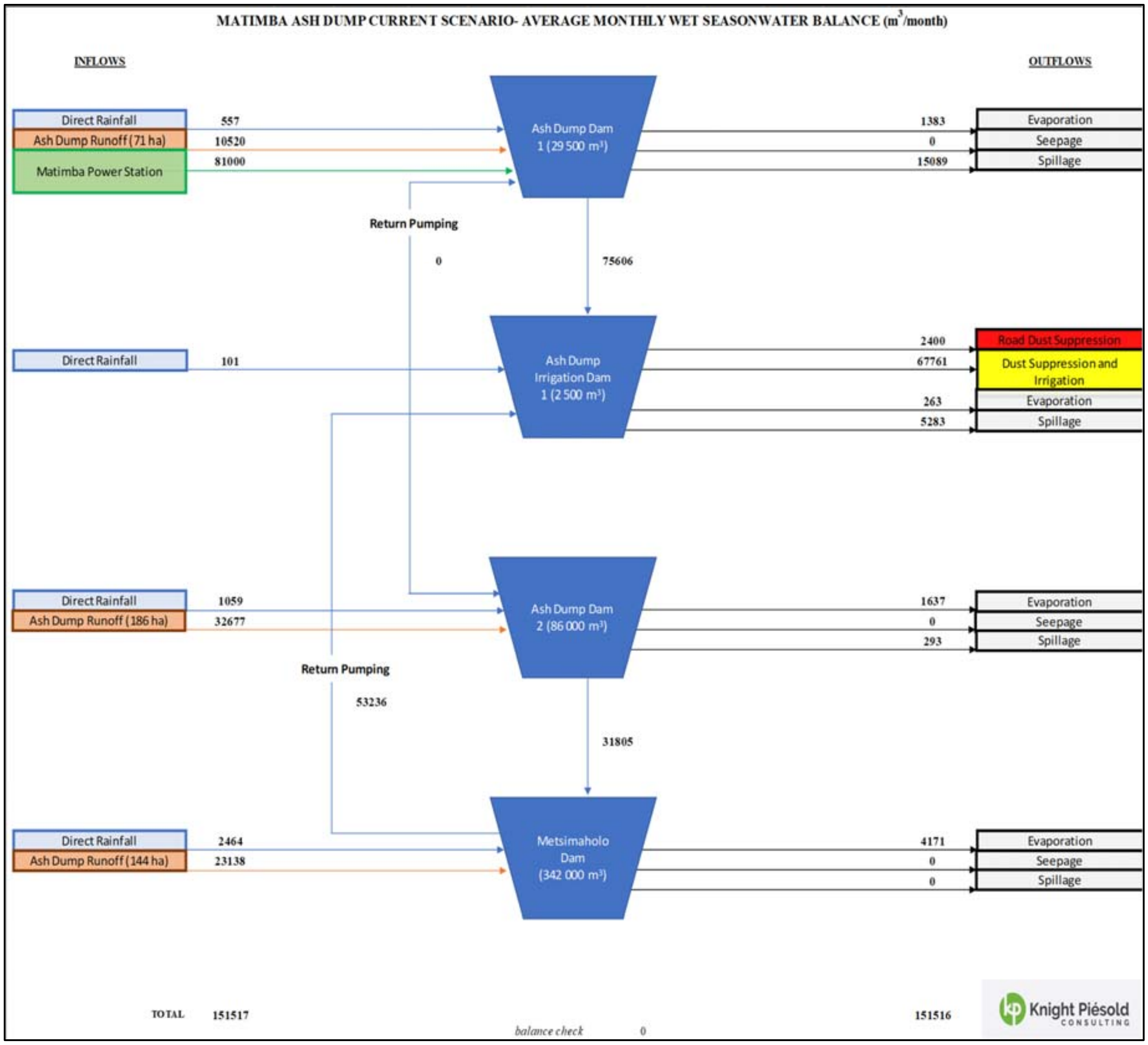

Figure 7-15: The average monthly wet season water balance for the current existing system scenario

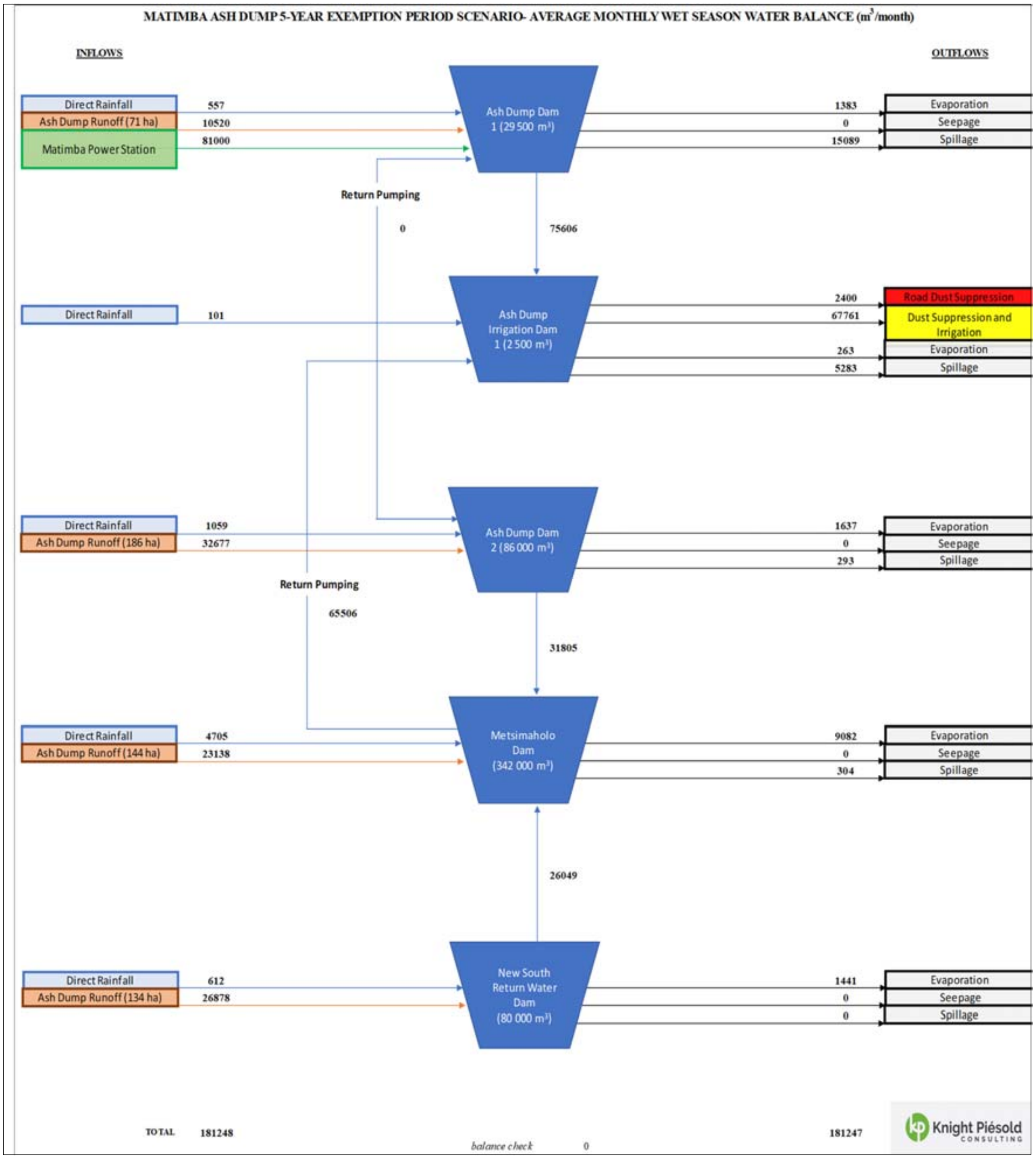

Figure 7-16: The average monthly wet season water balance for the 5-year exemption period scenario

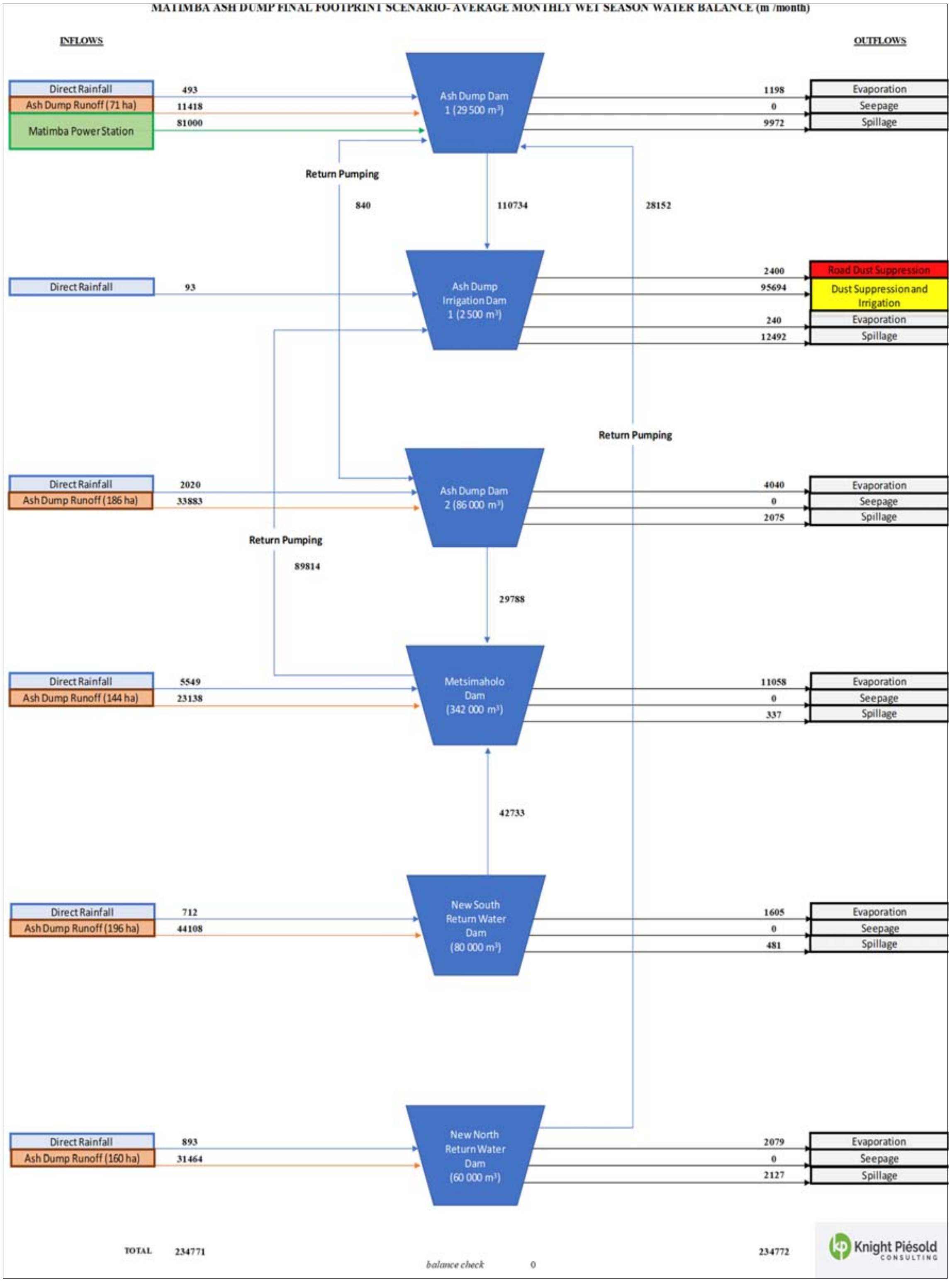

Figure 7-17: The average monthly wet season water balance for the final footprint scenario

7.3.7.3 AVERAGE DRY SEASON WATER BALANCE

The average monthly dry season water balances are shown below for the current existing system scenario (**Figure 7-18**), the 5-year exemption period scenario **(Figure 7-19**) and the final footprint scenario (**Figure 7-20**).

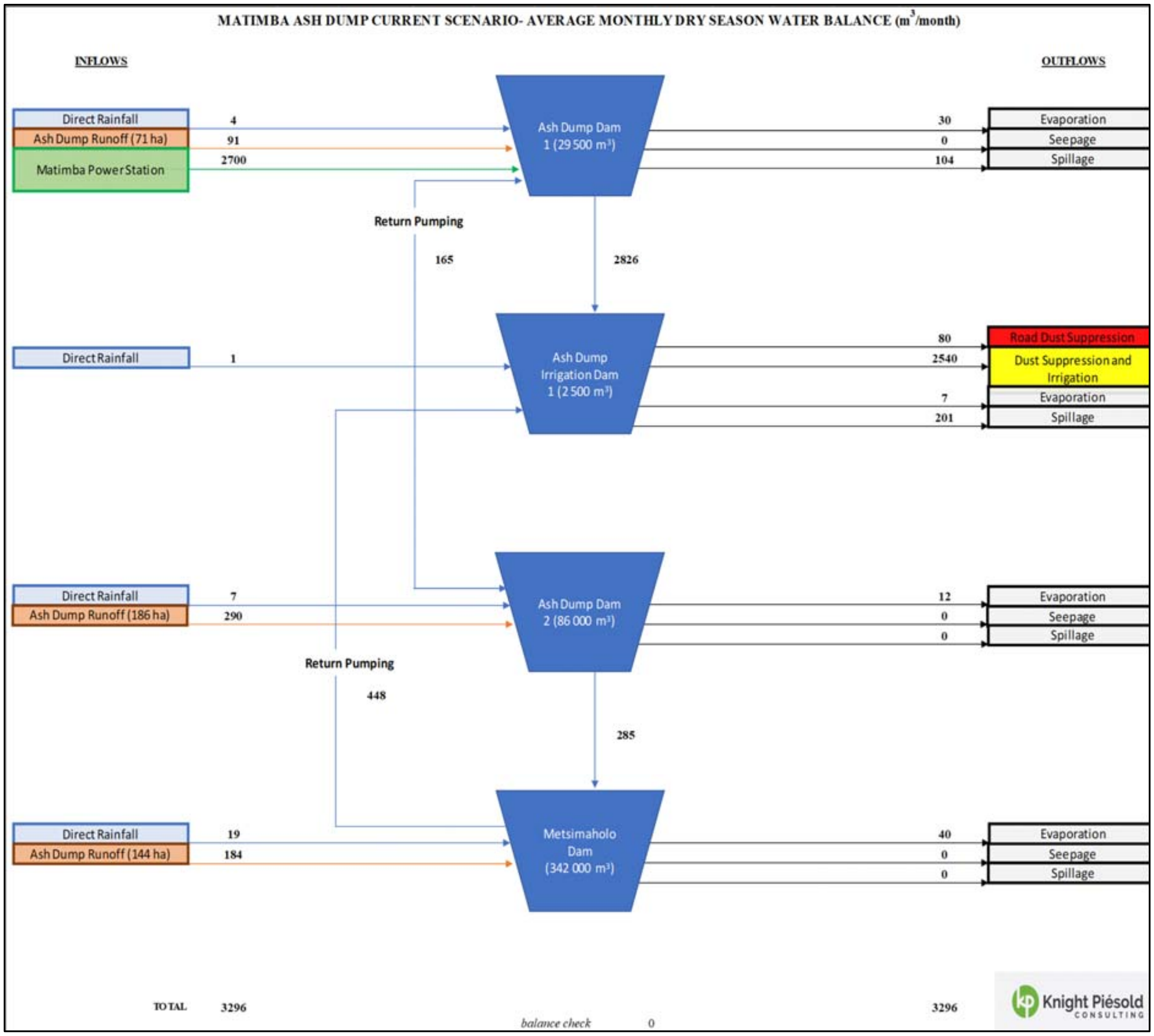

Figure 7-18: The average monthly dry season water balance for the current existing system scenario

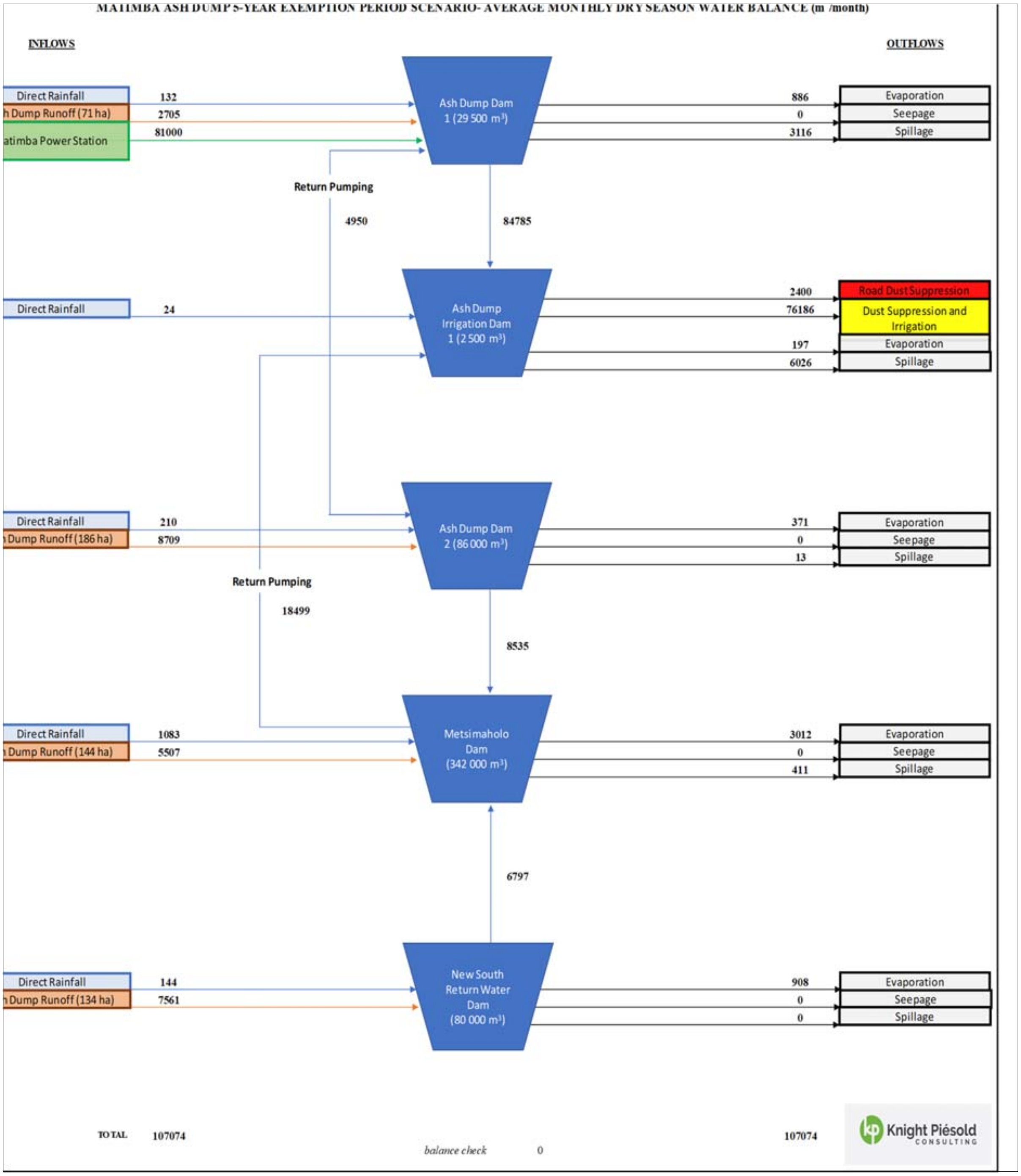

Figure 7-19: The average monthly dry season water balance for the 5-year exemption period scenario

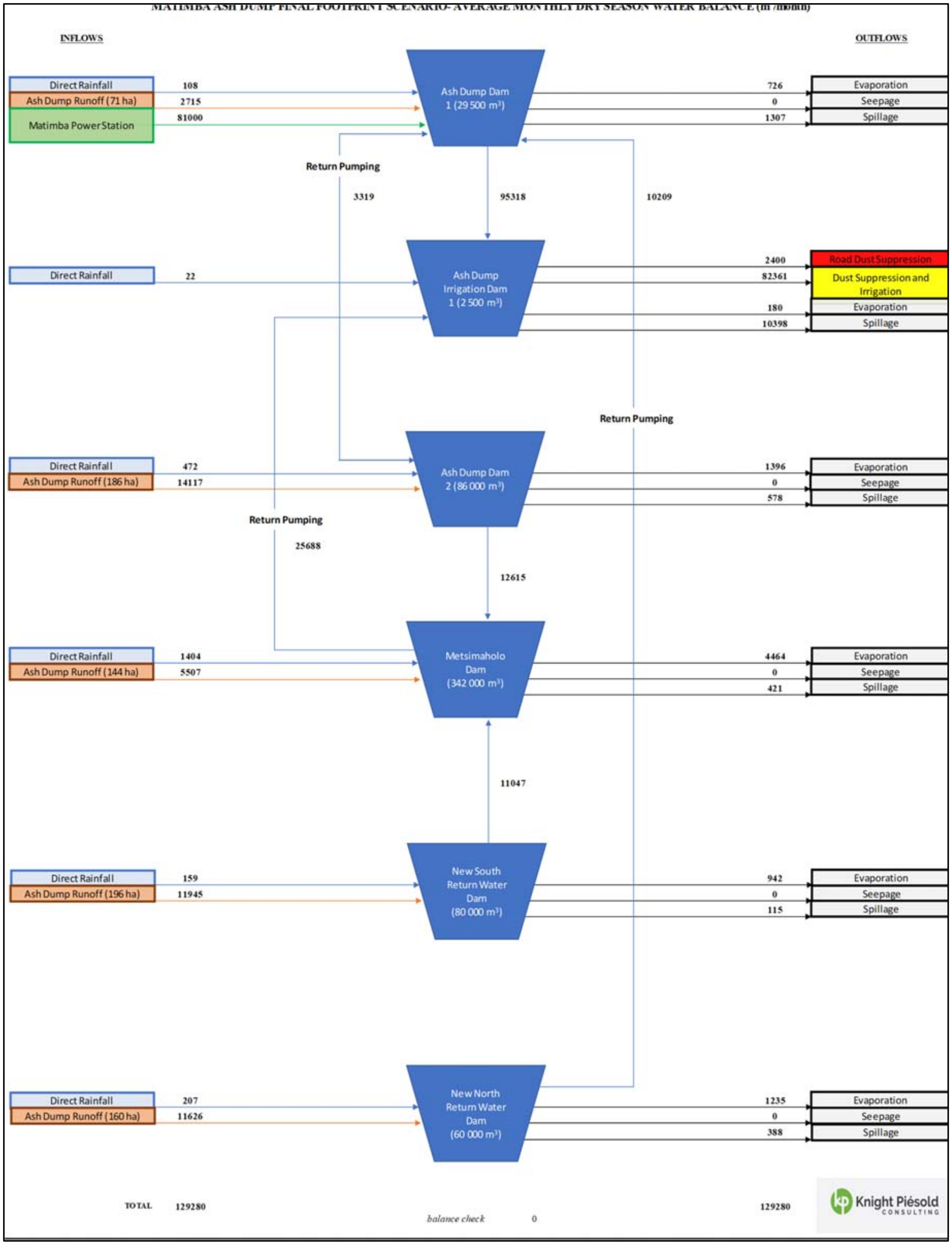

Figure 7-20: The average monthly dry season water balance for the final footprint scenario

7.3.8 CLEAN AND DIRTY WATER SEPARATION

The existing storm water dams capture all surface water runoff from the Ash Dump Facility as well as the Matimba Ash Dump recovery water. The system is considered dirty, even though a large portion of the Ash Dump is rehabilitated and should be considered clean.

A strategy will be developed to convert the dirty water dams to clean water dams as the rehabilitation is completed. The conversion strategy will initially include Ash Dam 2, as this dam will only receive water from the rehabilitated areas. Ash Dam 1 receives the Matimba Power Station recovery water which is considered dirty water and as this water will continue to be pumped from the station this dam will be required to be maintained as a dirty water dam. Metsimaholo Dam, New North and South RWDs could also be included in the conversion strategy once the rehabilitation is completed in the corresponding catchments. Section 10 discusses the conversion of the dirty water to clean water dams.

8.0 HOLDING DAMS

The following Sections describe the sizing of the proposed new North and South RWDs. The new return water dams will capture the surface water runoff from the ash dump extension footprint.

8.1 DAM SIZING

A daily timestep water balance was developed for the RWDs sizing using Goldsim simulation software. The dam sizing water balance is dynamic and depends on many variables including rainfall, evaporation. The dams were sized for the 2% Average Exceedance Probability (AEP) (1 in 50-year return period). The AEP is defined as the probability that a given event (a spill in this case) over a given duration will be exceeded in any one year. This was considered as the minimum acceptable design criteria, in accordance to Government Notice 704 (GN 704) of 1999 on the minimum requirements for mine waste. The Goldsim model basically calculates a balance between the inputs and outputs of the system taking into account the amount of storage left in the system. To represent the water management system, the following elements have been included in the model:

- Rainfall element
- Catchment Runoff element (Surface water runoff)
- Dam element.

The above elements were used to build up the water balance and are described in more detail below. The operating rule and connectivity govern how the water streams produced from the different elements are linked together. The connectivity and operating rules (as mentioned above) are programmed into the Goldsim model. The time step of the model is dependent on the objective of the model. A daily time step model allows for a more accurate determination of the dam sizes and pump/pipeline capacities. A daily time step was used for this model as it accounts for the seasonal variations of both rainfall and evaporation. The model was run iteratively (by changing the proposed dam capacity) until all the relevant sizing criterion and operational philosophies were met.

Surface runoff is considered to be the runoff from pervious catchment areas. A single soil layer model, together with the SCS runoff equation (Schulze, 1995), is used to model the soil moisture budget and therefore calculate the surface water runoff. A single layer model is adequate for typical catchment areas. These areas are generally small and do not contain a defined watercourse or channel which intercepts the groundwater table. The catchments will produce runoff with the recharge or percolation from the catchment reporting to the groundwater system.

The SCS equation (Schulze, 1995) is used to determine the runoff depth from the excess water that spills from the interception storage. The difference between the excess water and the runoff infiltrates into the soil layer. The moisture in the soil layer is budgeted for by adding in the infiltration and subtracting the evaporation and the percolation from the moisture in the soil store.

The dam element is a storage element, with inflows and outflows. The inflows to the Dam element are:

- Runoff from a catchment area, which would be calculated using the catchment element.
- Rainfall falling directly on the surface of the dam. This is calculated as the daily rainfall depth multiplied by the water surface area of the dam.

The outputs from a typical dam are:

- Evaporation from the surface of the dam, which is calculated as the product of the evaporation depth for water body times the water surface area
- Seepage from the dam floor if unlined, which is calculated as the product of a seepage rate and the water surface area in the dam. The seepage is assumed to leave the water system (not applicable at Matimba as the dams are lined)
- Pumping from the dam, which is governed by the operating rules. The pumping outputs in this case is the water re-use on site in the form of irrigation and dust suppression.
- Spillage occurs when the dam is full and overflows.

A 55-year rainfall record (SAWS 0674400_W) was used to model the RWDs as described in Section 7.1 The dam was sized iteratively to meet the GN 704 guidelines as well the required water re-use. The analysis determined that a dam capacity of 60,000 m^3 for the North Return Water Dam and 80 000 m^3 for the South Return Water Dam, is sufficient to meet the spill criteria, with a maximum pumping rate of 2,000 m^3/d and 6,000 m^3/d from each of the dams respectively. **Figure 8-1** below describes shows the daily runoff volumes reporting to the North and South RWDs (catchment areas of 162 ha and 229 ha). **Figure 8-2** and **Figure 8-3** shows the simulated dam water levels over the 55-year period for both the RWDs.

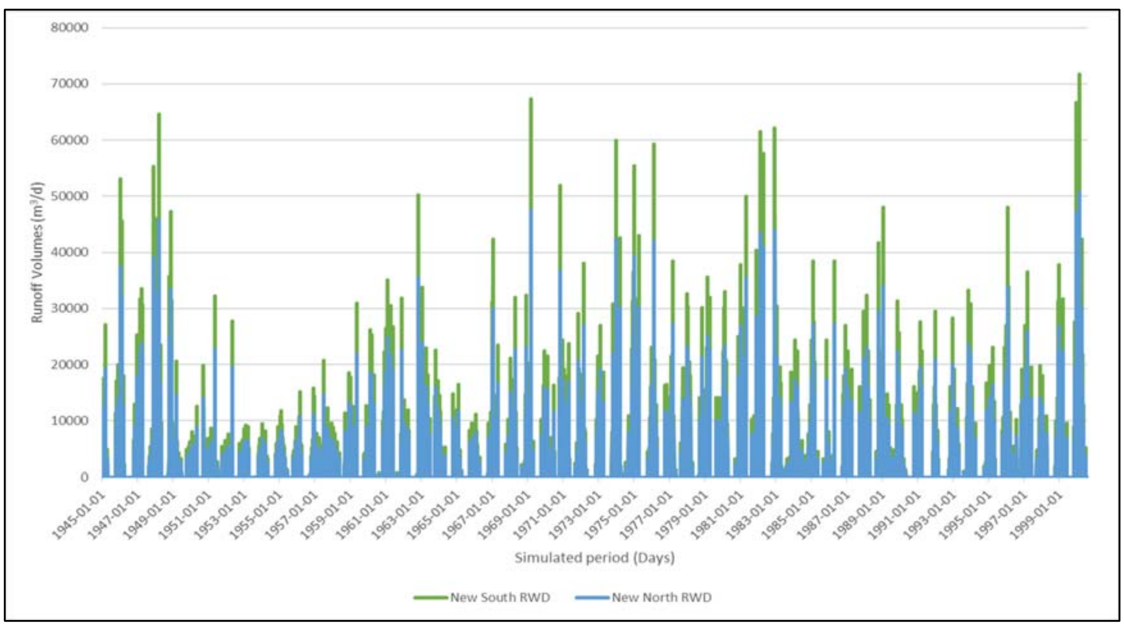

Figure 8-1: Simulated runoff volumes to North and South RWDs

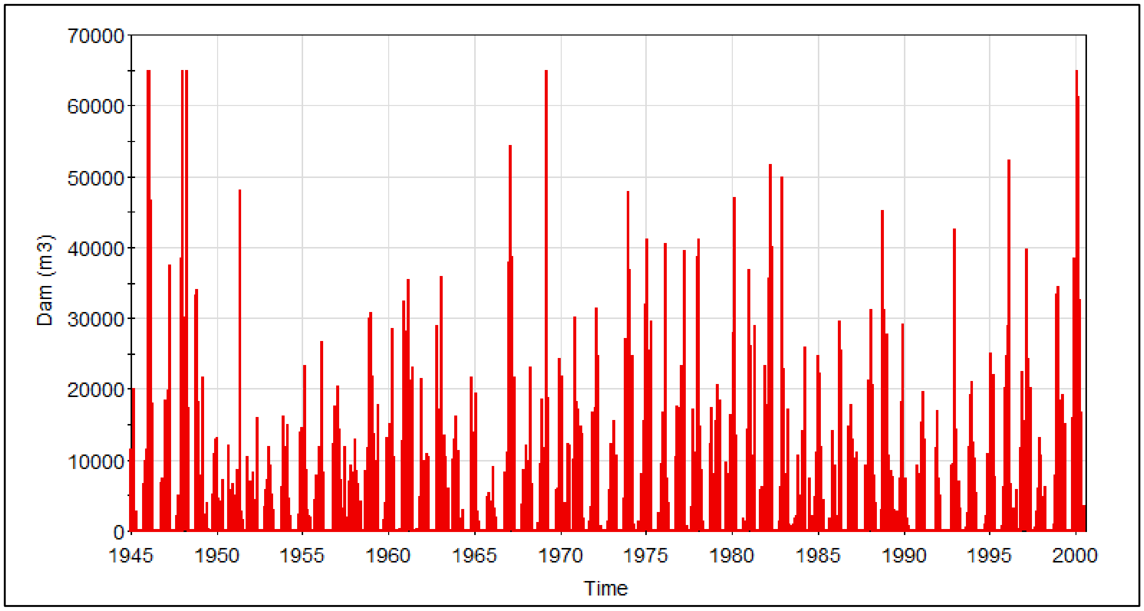

Figure 8-2: Simulated North RWD water levels over the 55-year period

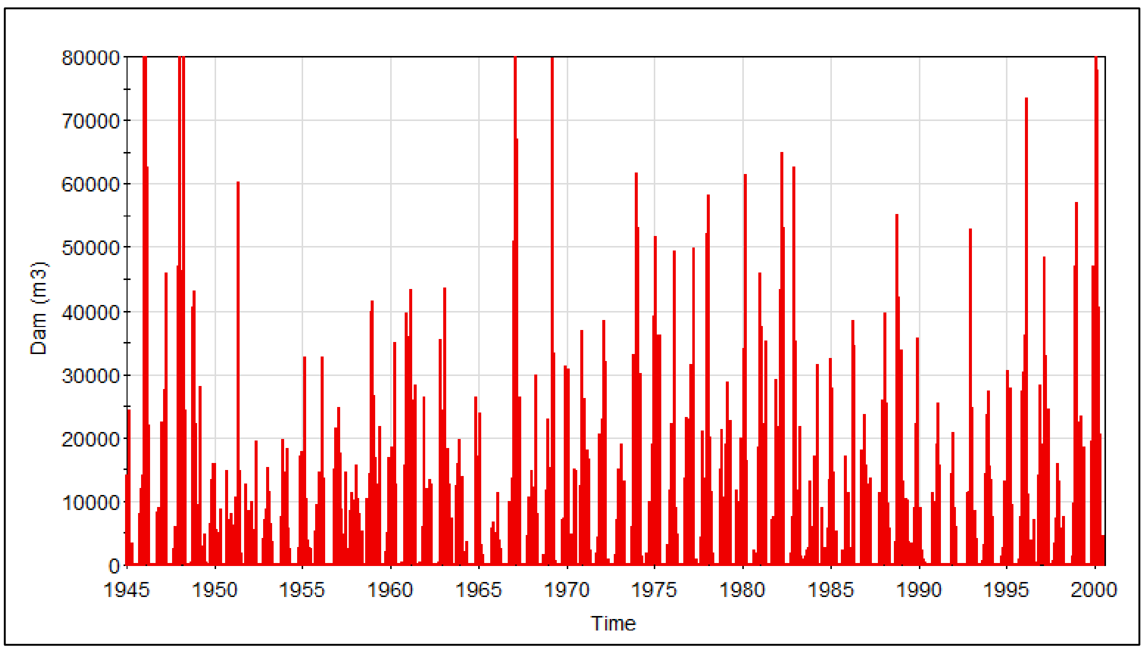

Figure 8-3: Simulated South RWD water levels over the 55-year period

8.2 DAM SAFETY REGULATIONS

The Dam Safety Regulations were promulgated in 2009 to promote the safety of dams within South Africa. According to the National Water Act, the Minister of Water Affairs has the power to control the design, construction, operation, alteration or abandonment of a "dam with a safety risk".

The proposed RWDs capacities were found to be 60 000 m^3 and 80 000m^3, with a max depth below ground of 5 m. The Dam Safety Office of the DWS requires dams to be licensed when a safety risk is present; this occurs when the maximum wall height is greater than 5 m and with a storage capacity of more than 50 000 m^3. In this case the maximum wall height is less than 5m (dam is below NGL), and the capacities are more than 50 000 m^3. Therefore, no licensing is required. Thus, the RWDs are not considered dams with a safety risk.

In order to develop design criteria for the spillway and freeboard requirements, we have adopted the classification system in the dam safety regulations for dams with a safety risk. Although this RWDs are not classified as a dam with a safety risk, the regulations have been used as the Hazard Rating is an important consideration. The RWDs are dirty water dams and as such there is a potentially significant impact to the environment. Therefore based on the size classification in **Table 8-1** from the Guidelines on Safety in relation to floods (SANCOLD, 1991), the SWD is classified as a small dam.

Table 8-1: Size classification (SANCOLD, 1991)

Size class	Maximum wall height (m)
Small	More than 5 and less than 12
Medium	Equal to or more than 12 but less than 30
Large	Equal to or more than 30

The dam safety regulation defines the classification of a dam's hazard rating based on a separate consideration of the potential loss of life, economic loss and the impact on resource quality. The factor giving the highest rating being decisive.

Table 8-2: Dam hazard rating (SANCOLD, 1991)

Hazard rating	Potential loss of life	Potential economic loss	Potential impact on resource quality
Low	None	Minimal	Low
Significant	Not more than 10	Significant	Significant
High	More than 10	Great	Severe

For the RWDs, the potential loss of life is low, the potential economic loss is minimal and the potential impact on the resource quality is significant. Therefore, the hazard rating of the dam as a result of failure is considered to be significant.

Based on the hazard classification (significant) and the size classification (small), both the SWDs are category II dams.

Table 8-3: Hazard Rating (SANCOLD, 1991)

Size class	Low	Significant	High
Small	I	II	II
Medium	II	II	III
Large	III	III	III

8.3 SPILLWAY DESIGN

In order to ensure that the spillways for the new North and South Water dams have an acceptable level of performance, the Recommended Design Discharge (RDD) and the Safety Evaluation Discharge (SED) flowing over the spillways were examined, for the design and extreme floods respectively.

The RDD for dams with a safety risk according to the Guidelines on Safety in Relation to Floods (SANCOLD, 1991) are given in **Table 8-4**.

Table 8-4: Recommended Design Discharges (SANCOLD, 1991)

Dam size class	Hazard Rating		
	Low	Significant	High
Small	$0.5Q_{50} - Q_{50}$	Q_{100}	Q_{100}
Medium	Q_{100}	Q_{100}	Q_{200}
Large	Q_{200}	Q_{200}	Q_{200}

The SED for dams with a safety risk and with catchment sizes within the transition zone according to TR 137 (Kovacs, 1989) are given in **Table 8-5**. The floods are calculated using the Regional Maximum Flood (RMF) Method.

Table 8-5: Recommended safety evaluation floods (SANCOLD, 1991)

Dam size class	Hazard Rating		
	Low	Significant	High
Small	$RMF_{-\Delta}$	$RMF_{-\Delta}$	RMF
Medium	$RMF_{-\Delta}$	RMF	$RMF_{+\Delta}$
Large	RMF	$RMF_{+\Delta}$	$RMF_{+\Delta}$

From the tables above, the RDD and SED are Q_{100} and $RMF_{-\Delta}$ respectively. The SWD spillway shall be constructed of concrete to form a mild channel. Discharge over the spillways will be controlled by a critical section at the outfall of the channel spillway. The dams were assumed to be full during the sizing of the spillways.

The spillway geometry and surcharge rise of the RWDs above FSL for the RDD and the SED are given in **Table 8-6**.

Table 8-6: SWD spillway characteristics

Structure	Spillway Length (m)	RDD (m^3/s)	Water Level rise (m)	SED (m^3/s)	Water Level rise (m)
North RWD Spillway	20	7.32	0.15	70	0.3
South RWD Spillway	20	13.67	0.15	92	0.4

8.4 FREEBOARD CALCULATIONS

The South Water Dam (SWD) and the NWD are classified as small category II dams. The total freeboard was determined according to the Interim Guidelines on Freeboard for Dams (1990) and is equivalent to the RDD routed through the proposed spillway plus the wave run up and the wave setup.

GN 704 also stipulates that the minimum freeboard of dams that form part of a dirty water system shall be 0.8m above FSL. Consequently, the SWD spillway was designed for a total freeboard of 1.0 m taking into account the wave run up and the wave setup (a conservative approach was adopted, and an additional 0.19 m freeboard was added). The magnitudes of the freeboard components are summarised in **Table 8-7**.

Table 8-7: SWD freeboard components

Structure	Spillway Length (m)	Wave run – up (m)	Wave set up (m)	RDD (m)	Total Freeboard (m)
North RWD	20	0.4	0.12	0.2	0.80
South RWD	20	0.4	0.12	0.3	0.80

Therefore the spillway complies with both the SANCOLD guidelines and GN 704.

8.5 EMBANKMENT MATERIAL AND SLOPES

The RWDs and Sediment Trap must be provided with an adequate containment barrier system in accordance to the Minimum Requirements for Waste Disposal by Landfill 2nd Edition (1998) and the Standard for Disposal of Waste to Landfill (Act no. 59 of 2008).

The design of the embankment walls was based on past experiences. Test pits excavated during the Geotechnical Investigation showed that the material on site consists of Colluvium and residual sandstone (GM/SM) and is suitable for the construction of the dam walls. Shear box test results indicated that the in-situ material has an internal friction angle of 34˚. A summary of the geotechnical findings can be found in **Appendix C**. However, the USBR (1987) recommends an upstream slope and downstream slope of 3:1 and 2:1 respectively, for small dams with a homogeneous earthfill.

Case	Type	Purpose	Subject to rapid drawdown[1]	Soil classification[2]	Upstream slope	Downstream slope
A	Homogeneous or modified-homogeneous	Detention or storage	No	GW,GP,SW,SP	Pervious, unsuitable	
				GC,GM,SC,SM	2.5:1	2:1
				CL,ML	3:1	2.5:1
				CH,MH	3.5:1	2.5:1
B	Modified-homogeneous	Storage	Yes	GW,GP,SW,SP	Pervious, unsuitable	
				GC,GM,SC,SM	3:1	2:1
				CL,ML	3.5:1	2.5:1
				CH,MH	4:1	2.5:1

Figure 8-4: Recommended slopes for small homogeneous earthfill dams on stable foundations (USBR, 1987)

9.0 TOPSOIL MANAGEMENT

9.1 AVAILABLE TOPSOIL

A detailed assessment was done to estimate the available topsoil from the available ash dump extension footprint area. This was achieved by using the LiDAR survey provided for the design of the ash dump. This is shown approximately in brown in **Figure 9-1** below based on the proposed footprint of the ash dump extension.

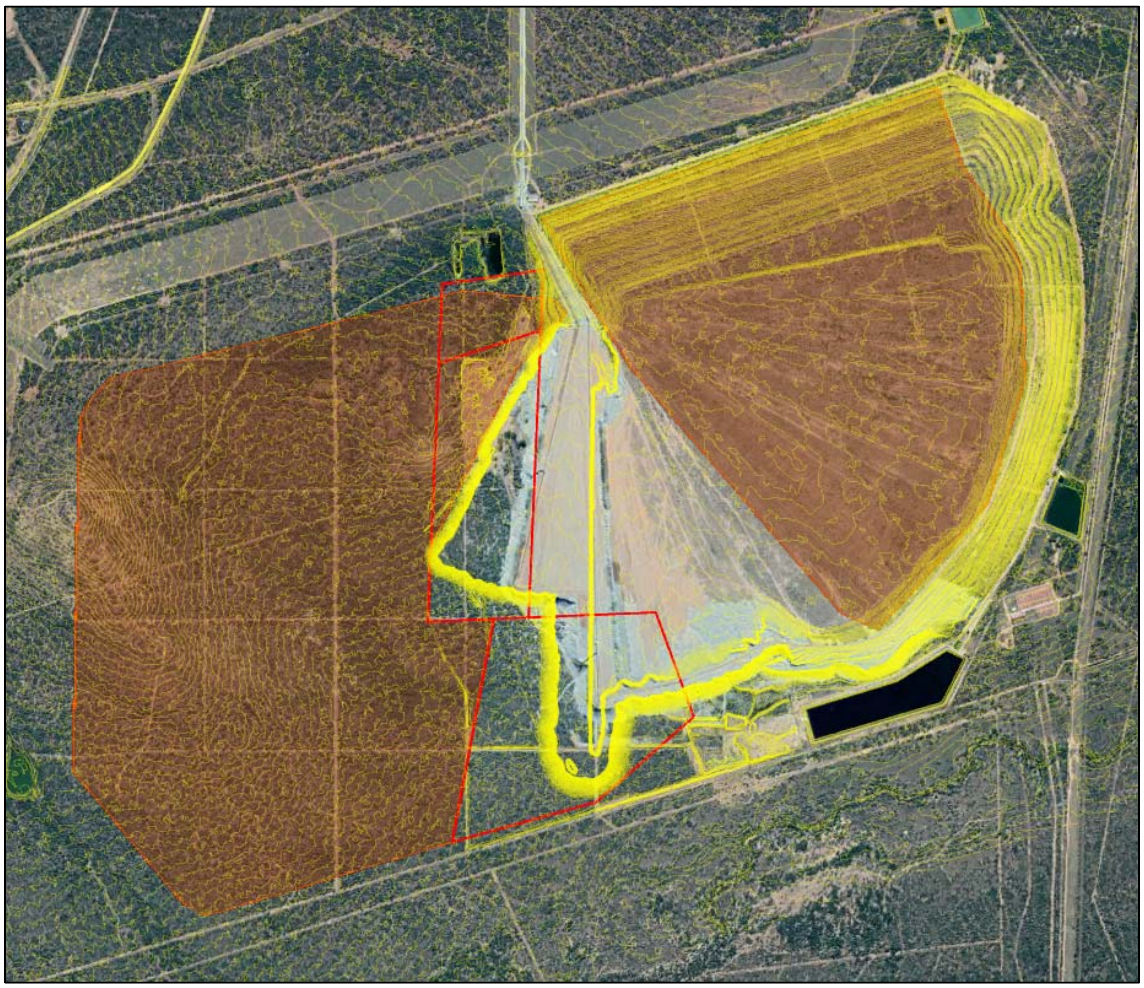

Figure 9-1: Available topsoil assessment after the exemption period

The investigated area is generally covered by transported soils and by a relatively thin topsoil cover. Transported soils comprise colluvium and aeolian material and occurs from surface to a depth of between 0.4 m and 2.9 m. It has a pinhole voided soil structure and a loose to dense consistency with depth. The soil grading is mostly silty sand and is considered to be suitable for top soiling/grassing of the ash dump surfaces.

Twelve auger holes were drilled across the existing ash dump in order to assess the thickness and extent of the topsoil cover. The topsoil is described as orange to dark brown, organic rich silty to gravelly sand with gravels comprising of quartz and ferricrete nodules. The topsoil is present at variable depths, ranging between 0.2 m and 0.8 m from surface, which is underlain by coal ash. The exact profiles are attached under **Appendix C-6.**

9.2 PROPOSED OPERATIONAL PROCEDURE

Topsoil will be removed for every 4-year phase of development and moved to the required stockpile area on top of the new backstack or as directed by the Matimba or responsible engineer on site. The designated areas to be identified by the Matimba or responsible engineer on site. The original design topsoil thickness to be stripped from the footprint is 300 mm and is the basis of this estimate. Should a deficit be found for the current dump, the thickness will be increased, and the costs can be adjusted accordingly. A grid of test holes dug by hand should be done to verify the depth of topsoil is a minimum of 300mm. Erosion protection will be required during the operational phase of the project.

9.3 TOPSOIL REQUIREMENTS

The total topsoil area for the top radial shifting configuration is estimated to be 4,649mil m^2. Assuming the topsoil to be reinstated onto the final ash dump surfaces at a depth of 300mm, the required volume of topsoil for the top layer radial ashing configuration is 1,394mil m^3.

The total topsoil area for the bottom radial shifting configuration is estimated to be 1,889mil m^2. Assuming the topsoil to be reinstated at a depth of 300mm, the required volume of topsoil for the bottom layer radial ashing configuration is 566,731 m^3.

The total dust suppression area for the bottom radial shifting configuration is estimated to be 3,074 mil m^2. Assuming the dust suppression to be placed at a depth of 50 mm, the required volume for dust suppression for the bottom radial ashing configuration is 153,700 m^3. It is expected that the 50 mm gravel layer will be sourced from the area below the topsoil.

The total volume topsoil required for the radial ashing configuration is thus estimated to be 2,115 mil m^3.

Table 9-1: Topsoil requirement summary

Description	Unit	Layer thickness
Layer thickness	mm	300
Available volume from existing stockpiles	m³	0
Available volume – extension area topsoil (assumed at 500mm)	m³	1,299,604
Available volume – existing ash dump topsoil (assumed average 300mm)	m³	434,466
Total available volume	m³	1,664,070
Overall required volume (Radial ashing including Ramp up)	m³	2,310,000
Net volume deficit for (Radial ashing)	m³	645,930

A topsoil availability assessment was done to evaluate if there is an inadequate volume of topsoil for rehabilitation. The assessment confirmed that there will be a deficit volume of 645,930 m^3 of topsoil for rehabilitation using a 300mm topsoiling thickness.

. An average of 800mm of topsoil is required to achieve a topsoil requirement and equilibrium, it is however clear from the geotechnical report that 800mm cannot be stripped from the surface. t It is therefore recommended that in order to maximise the topsoil usage, top soil sampling grids be conducted for each phase to assess the exact amount of top soil available as it varies throughout the site. Should unforeseen rock be encountered whilst excavating, it is recommended that shortfall of material be sourced commercially.

A typical layout of topsoil requirement is shown in **Figure 9-2**. It is envisaged that 50mm of gravel will be used as a dust suppression between conveyor shift positions on the front stacks to eliminate the ash dust from flaring up.

Note: Stripping of topsoil to an average depth of 500mm, followed by further stripping to a depth of 300mm for use as the protection layer above the HDPE liner, means that the HDPE liner will be 800mm below the current natural ground levels. After applying the 300mm protection layer over the HDPE, the final surface will still be 500mm below natural ground level. This creates a drainage issue at the ash dam South perimeter, where it is necessary to discharge polluted stormwater run-off by gravity into the perimeter dirty water drain. It will therefore be necessary to limit the top-soil stripping depth in a zone around the South perimeter to obviate the above step in levels, to achieve unhindered gravity drainage into the dirty water canal.

The required terracing can be achieved by increasing the top-soil and protection layer thicknesses to 600mm in the zone of the natural ridge running across the site and decreasing it slightly to say 250mm near the South perimeter toe wall.

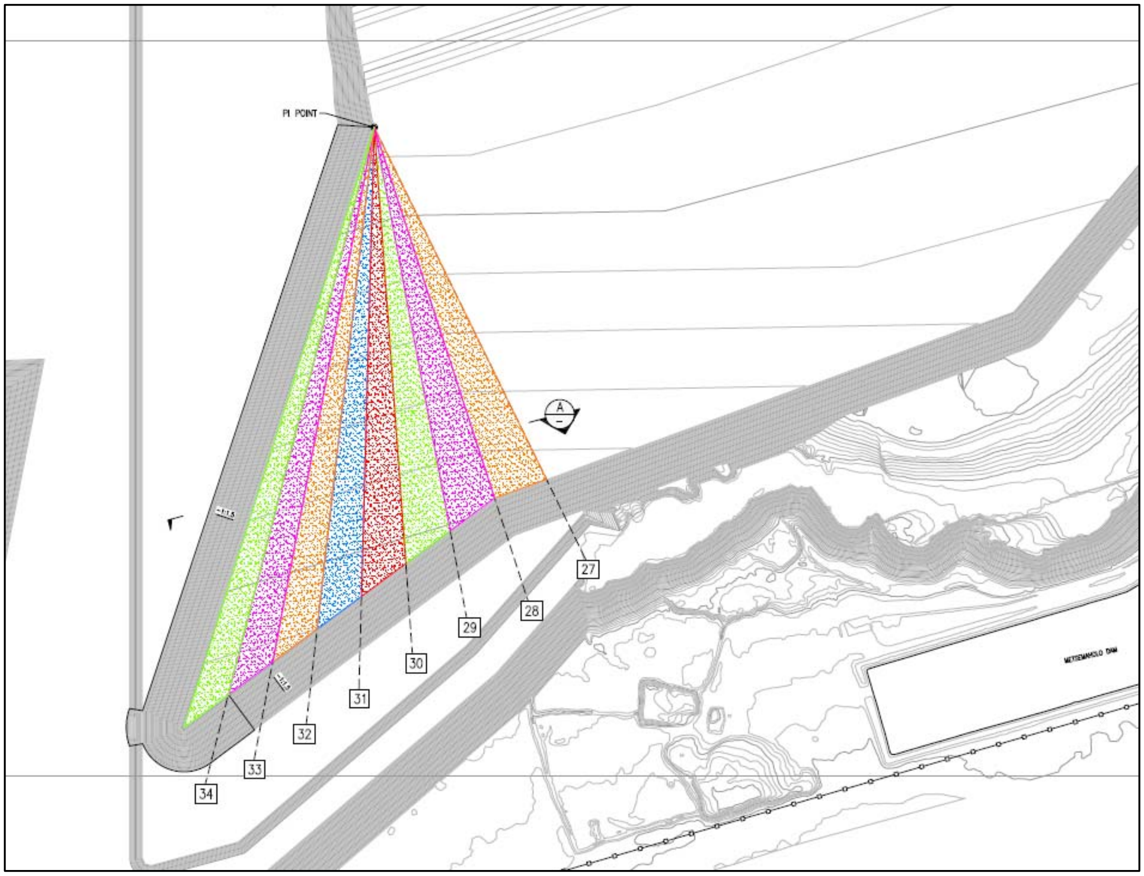

Figure 9-2: Topsoil requirement typical layout

9.4 LINER PROTECTION LAYER

Once the liner has been successfully installed, a 300mm thick earth-fill protection layer will be placed on the liner to protect the liner. It is not anticipated that any material will require sieving. The total area of liner to be placed is estimated to be 3.074mil m^2 (refer to **Figure 9-2**) and will be done in 4 year increments.

The total volume of selected gravel at 300mm deep is estimated to be 922,203m^3. This volume can be excavated from the ash dam footprint prior to the placement of the liner at 300mm deep and subsequently stockpiled while placing the liner. This material will need to be sourced commercially if found to be unsuitable or insufficient.

The overall site foundation stripping depth is thus 800mm (+-500mm for topsoil and 300mm for the protection layer). The site foundation at the stripped depth must then be ripped to 200mm depth, conditioned with a polymer soil binder if deemed necessary, graded to line a level and compacted to 90% Mod AASHTO.

See also the Note in Section 8.3 above in respect of the required terracing around the ash dump perimeter.

The option of using coarse ash as a pioneer layer on top of the liner has been investigated but proved not to be feasible for use. The coarse ash offloading area available at Matimba Power Station is 900 m^2. As the coarse ash is dependent on plant availability, it is not possible to estimate how much course ash can be produced.

10.0 STORMWATER MANAGEMENT PLAN

10.1 INTRODUCTION

The ash dump extension footprint is on sloping ground with a high point at the north-west extent of the ash dump extension site. The footprint thus slopes towards the current ash dump and towards the current advancing ash face of the active phase of lined ash deposition area, see **Figure 10-1** below.

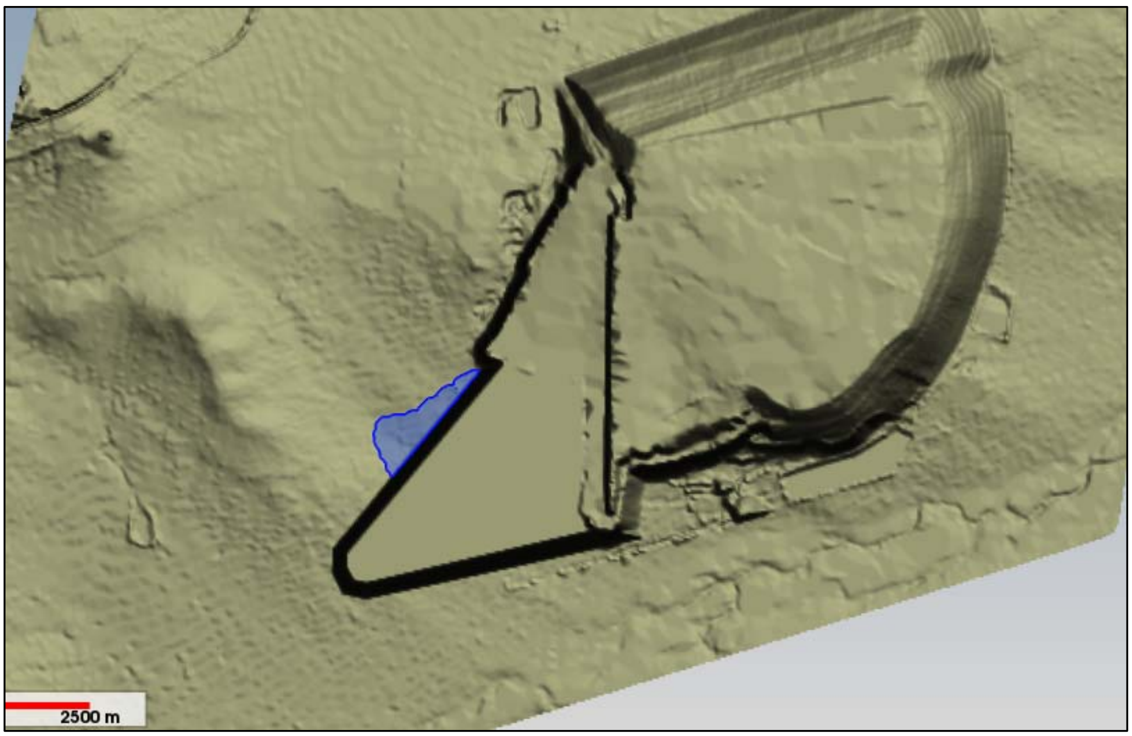

Figure 10-1: Stormwater run-off trapped against advancing ash face

The sloping footprint also features a valley depression running from the north-west high point in a south-easterly direction towards the western edge of the lined ashing areas. For the phase 1 and 2 lined areas (years 1 to 8), dirty storm water run-off from the advancing ash deposit slopes and from the un-covered lined area will drain naturally by gravity southwards to the south boundary dirty water drain. For phase 1 and 2 to drain using gravity drains effectively, partial reshaping of the terrace during the installation of the liner system will be needed to ensure the that the storm run-off does not pond against the advancing slopes. Phase 1 and 2 terrain is relatively flat, and the valley along the south west region not too high relative to the footprint.

For each of the remaining lined ashing areas phase 3 to 8, (years 12 to 33), seen in **Figure 10-2**, the clean water from the un-lined Western catchment portion of the site will collect in the valley depression against the East perimeter of the current lined section. As the valley depression is lower than the ash dump perimeter drains, collected clean water run-off will be discharged by gravity to the ash dump perimeter via a below-ground decant pipe to achieve discharge to the environment.

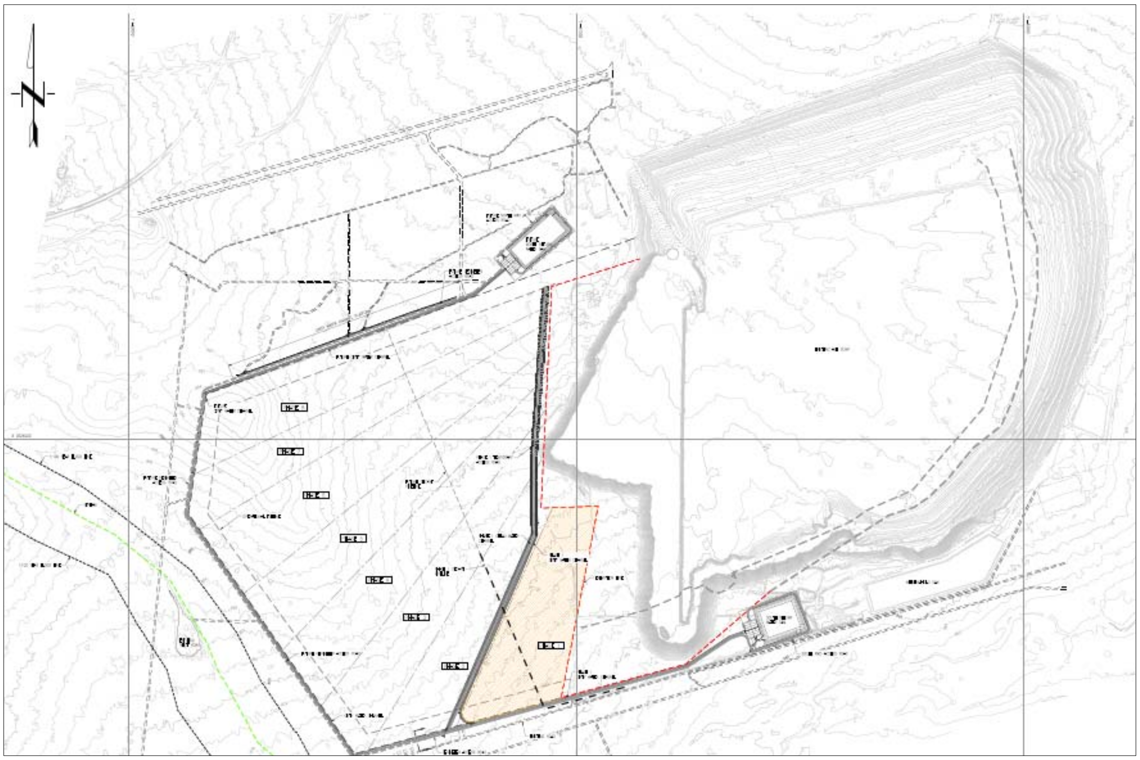

Figure 10-2: Continuous ashing 4 years phase lines

The proposed final ash dump design covers approximately 279 ha of footprint and will require two additional return water dams to ensure no spillage on the existing system as a whole system will have to be integrated upon closure of facility. The proposed new North Dam will require capacity of 60,000 m^3 and 80,000 m^3 for the new South dam, see **Section 7.0** of this report. The two proposed dams will only cater for the dirty water which will captured by the dirty run-off area. Clean run-off water will be diverted to the environment and not stored in the system.

The proposed ashing operation initially involves stacking ash on top of the existing ash dump. Upon completion and full rehabilitation of the existing portion of the dump, the run-off from the entire existing footprint will be considered clean. Therefore, to comply with GN 704, which demands a separation between clean and dirty water systems, it is recommended that the existing South-East dam (Dam 02) is converted to a clean water collection dam. This dam will store run-off captured by the northern and eastern portion of the dirty water trench. This water can be used for irrigation in the rehabilitated areas, for operation in the plant, released to the environment and/or left in the dam to evaporate. **Section 7.3.8** above explains in detail.

10.2 PRINCIPLE OBJECTIVES FOR STORMWATER MANAGEMENT

Stormwater management objectives are:

- To ensure compatibility of the ash dump extension site with the relevant legislation from a surface water perspective.
- To recommend control measures in managing the increased contaminated run-off as a result of the proposed ash stacking extension.
- To provide ways to regulate and monitor how the storm water will be managed during the different stages of stacking.

10.3 LEGAL REQUIREMENTS

A Storm Water Management Plan (SWMP) is a statutory requirement for mining and related activities in South Africa and is defined by General Notice 704 and Regulation 77 of the National Water Act (Act 36 of 1988). No water use licences in terms of this act will be granted without an approved SWMP. The purpose of a SWMP is to prevent the pollution of water resources in and around mining areas, or areas where mining related activity occurs. Regulations define a methodological approach to preventing and/or containing pollution on mining sites, set design standards and specify measures that must be taken to monitor and evaluate the efficacy of pollution control measures that are implemented.

The basic principles of a SWMP include:

- Separation of clean and dirty water - clean water should, as far as possible, be kept separate from dirty water. Water from clean water areas should be diverted away from dirty water areas and should be allowed to pass through to downstream users. Dirty water must be contained and captured on site. Spillage of dirty water is not allowed except during extreme flood events that are, on average, exceeded no more than once in 50 years.
- Containment of dirty water - reasonable measures must be taken to ensure that dirty water is contained. All dirty water must be captured and transported in lined channels (capable of containing 1:50-year design floods) to prevent the seepage of contaminated water into groundwater resources. Dirty water runoff must be stored in a RWD, where reasonable precautions are taken to prevent leaks or seepage. RWD's may not spill more often than (on average) once in 50 years. The design standard is not that a 1 in 50-year flood should be captured, but that the dam may not spill. Design storage volumes are a function of peak storage requirements that often correspond to abnormally wet conditions that continue for an extended period, and not to a specific flood event.
- Reuse and recycling of dirty water - regulations stipulate a clear hierarchy of water use. First reuse any captured dirty water. Recycle as much water as possible. Minimise the import and use of clean water resources. Excess water released from a dirty water area must be treated to a standard agreed to by the regulator (DWA) and any plan to treat and release excess water must be approved and licensed.

- Preventing the pollution of water resources - exposure between water and potential pollutants should be reduced to a minimum. Special precautions may be required to prevent the transport of pollutants in water. Oil traps should be specified below workshops, fuel depots and vehicle wash-bays to prevent the flow of hydrocarbons into RWDs. Silt traps may be constructed where surface runoff is likely to lead to the transport of suspended sediments and the like
- Reducing dirty water areas - special attention should be paid to early rehabilitation of mining and other dirty water areas to reduce the dirty water footprint area to an absolute minimum. This will reduce the total volumes of dirty water and simplify the final measures to be taken at mine closure. Part of any SWMP will include processes that identify and implement opportunities to reduce the dirty water footprint areas.

10.4 STORMWATER MANAGEMENT DETAILS

Storm water management involves the effective handling of the quantity and quality of run-off water being discharged. Effective management requires that possible conditions of storm water be addressed adequately as these impact water bodies downstream. Also, erosion and sedimentation assert a detrimental impact on the existing drainage as the deposited silt and soil particles render the drainage incapable of operating at original design level. For the proposed Matimba Ash Dump development, clean run-off will be diverted away from the facility for all the stages of stacking.

Clean and dirty stormwater run-off during each deposition phase will be achieved by using the phase separation walls to divert the upstream clean run-off to the environment and to collect the deposition phase dirty run-off within the phase operating area, for subsequent transfer to the perimeter dirty water channels and storage dams.

The required elevation of each phase divider wall in the central valley zones was derived by assessing the depth/storage capacities of both the trapped valleys on either side of each divider wall, i.e. where external clean water from a 1:50 year storm on the upstream catchment would pond, and where 1:50 year storm dirty run-off from the active ashing phase would pond on the inside of the wall.

The resulting ponded levels are shown on Figure 10.2a below. The required divider wall elevations across the valley zones of each phase was then set 0.5m higher than the ponded level.

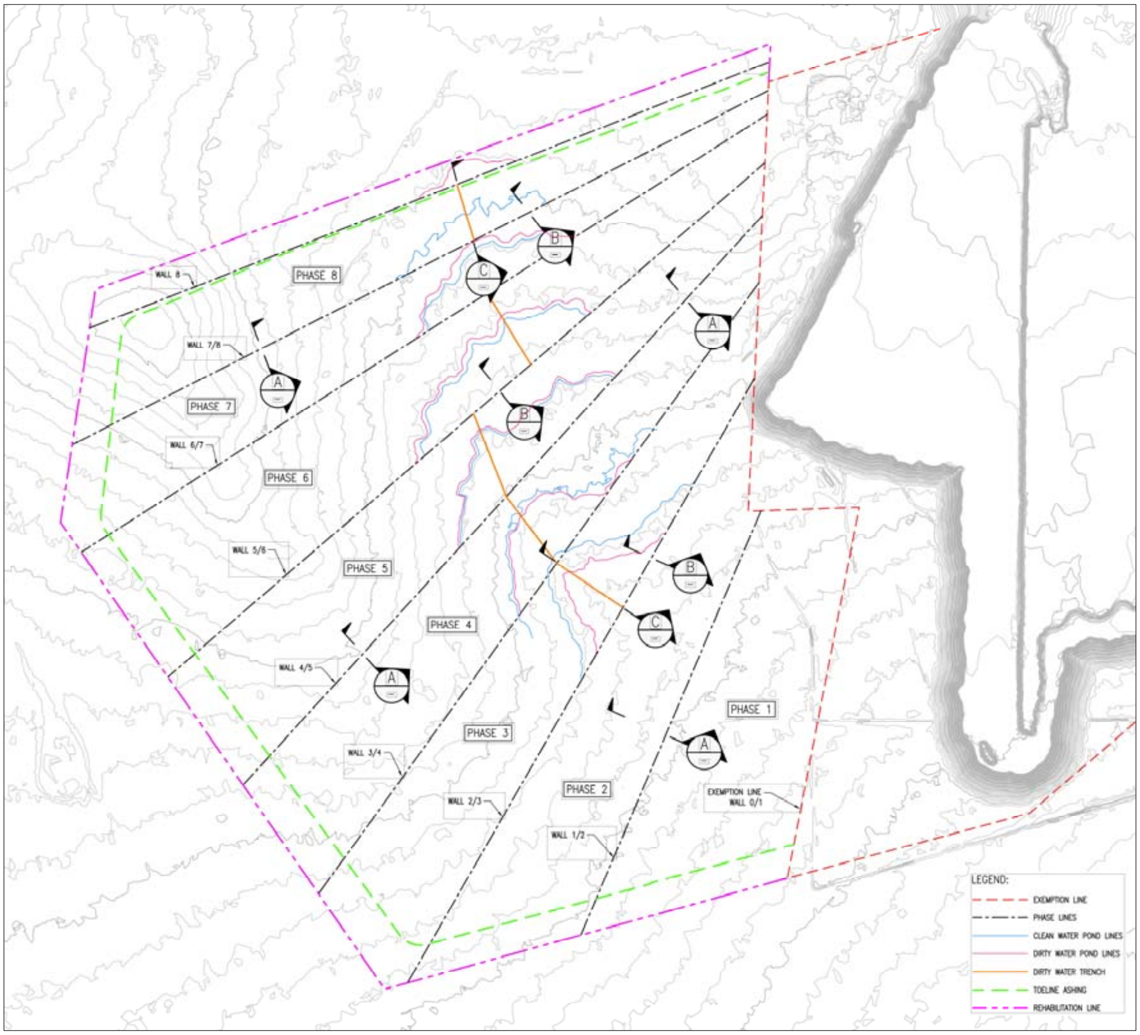

Figure 10-3: Derived Clean and Dirty Stormwater Ponded Levels

10.4.1 CLEAN STORM WATER RUN-OFF MANAGEMENT

10.4.1.1 CLEAN WATER DIVERSION VIA CONCRETE CHANNEL FOR PHASES 1 AND 2

A trapezoidal channel at the west perimeter divider wall of each phase, with base width of 1.5 m, 1:2 side slopes and 1 m metres deep, concrete lined only in the valley depression zone, will be used to divert clean water away from the active ash dump area for phases 1 and 2.

It is only for phase 1 and 2 where the clean water diversion channel can be used to divert water by gravity.

From phase 3 to 8, it is not possible, as there is a valley depression which causes the clean run-off to be trapped against the phase West perimeter.

For lined ashing areas phases 3 to 8, clean storm water run-off will gather in the valley zone against the west edge of the divider wall of the current lined area. The elevation of this pool is below the perimeter elevations of the ash dump phase extension (i.e. there is a ridge between the pool and the dump perimeter) as seen in **Figure 10-1** above. The ponded storm water will need to be transferred to natural ground outside the ash dump footprint by means of a buried gravity concrete penstock system which will drain in a southern direction to natural ground outside the ash dump extension footprint.

10.4.1.2 CLEAN WATER DIVERSION VIA BURIED PIPELINE (PENSTOCK) FOR PHASES 3 TO 8

Clean storm water run-off from the un-lined catchment of the site can be transferred by gravity to the environment in clean water diversion channels at the west perimeter of the phase 1 and 2 lined sites.

However, this is not possible for subsequent phases 3 to 8 due to the valley depression where stormwater will accumulate against the phase boundary. For these phases, a 650 NB buried spigot and socket pipeline of class 125D will be used to decant accumulated clean run-off for phase 3 to 8. The buried pipe will have inlets at lowest points of the water ponding, discharging into the buried decant outlet pipe. (see dwgs 301-825/01-321, 421, 529, 621, 721, and 820 for the phases 3 to 8)

These decant facilities will be used to transfer clean storm water run-off from the external catchment of the site, to natural ground outside the south perimeter of the ash dump extension footprint.

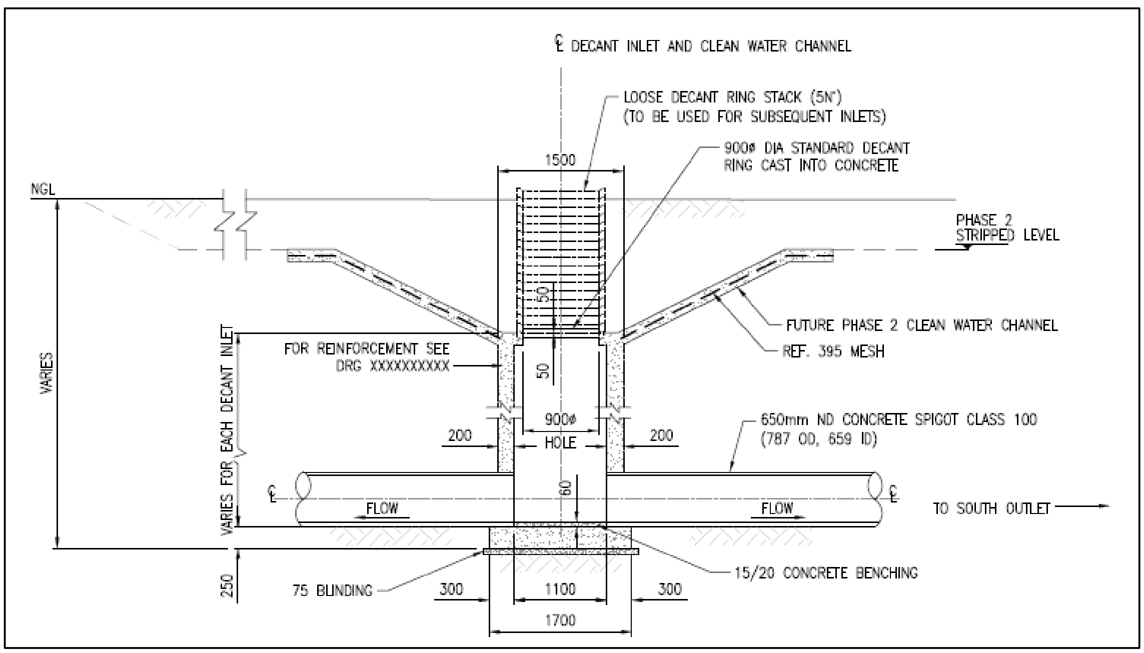

Figure 10-4: Decant Inlet

The change of direction of the penstock pipe at the South boundary will be achieved by construction of a concrete manhole with Rectagrid (or similar) steel grating and steps for access during inspection. The manhole wall will be 0.2 m thick, square with dimension 2.1mx2.1m in plan and X m deep. The base of the wall will consist of a 2.5m base, with benching of at least 0.35m on the manhole internal base portion. the benching will slope such that the water flows into the east facing pipe. See Error! Reference source not found. below for manhole plan layout.

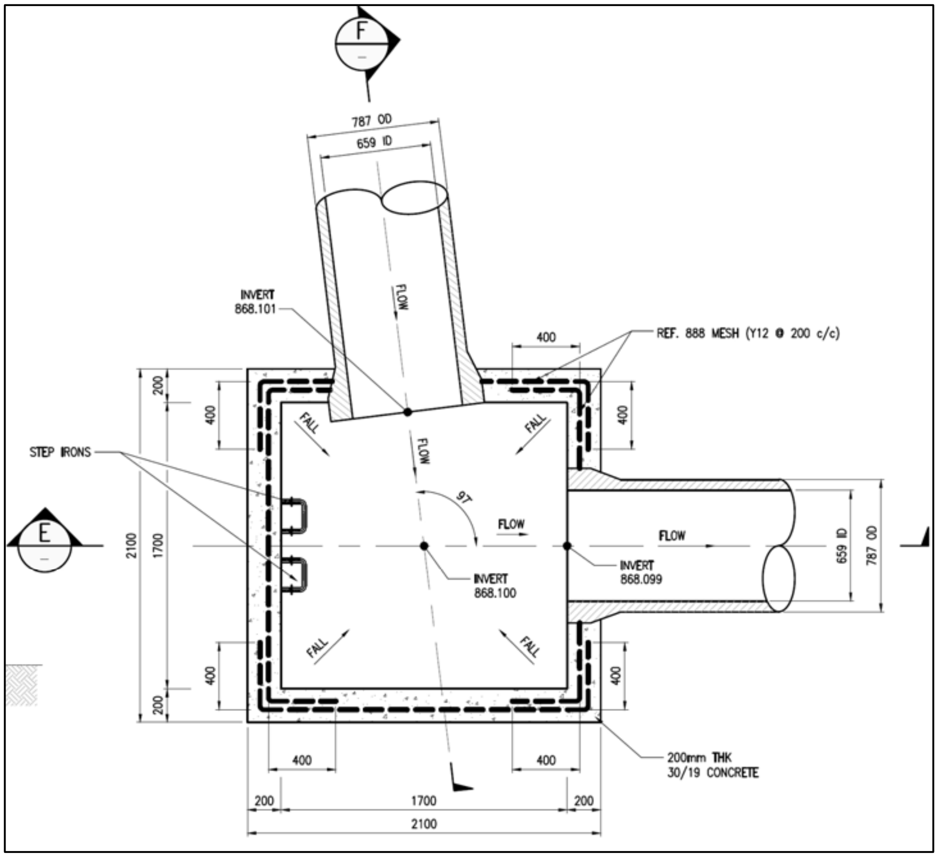

Figure 10-5: Manhole plan layout

In **Figure 10-6** below, the penstock is illustrated using a red dotted line, with the inlet position in the lowest point in the pond to ensure that the entire pond volume is released from the depression after a storm event. The buried decant pipe passes under the Phase 2 extension are so must be installed during the phase 1 works.

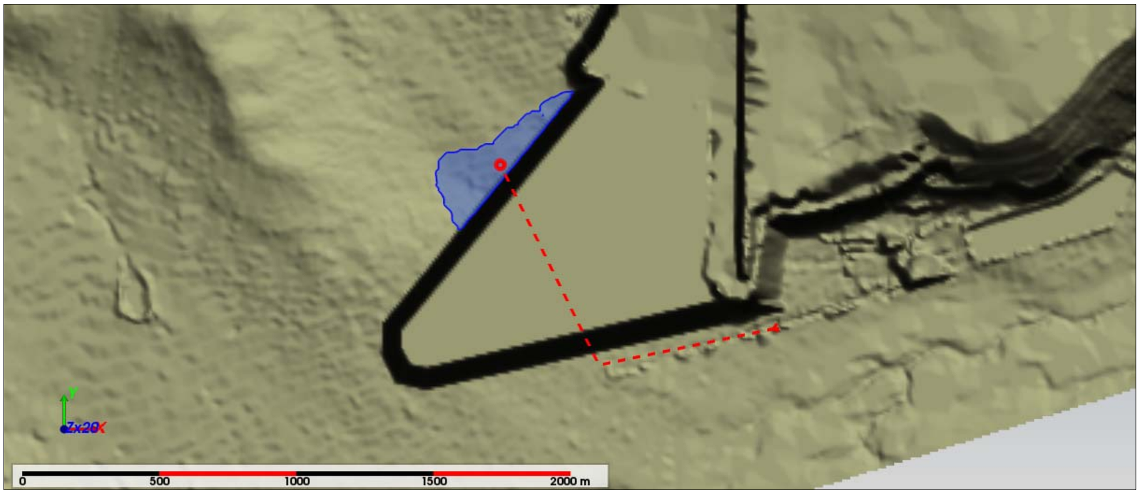

Figure 10-6 Buried gravity penstock for clean water drainage

For phase 3 to 8, the penstock inlets are strategically placed in the lowest points of the valley depression outside each phase line, these inlets will ensure that the entire volume of run-off is collected and transferred to the natural ground on the southern part of the ash dump extension. A 675 NB spigot and socket pipeline of class 75D has been sized to transfer the anticipated phase 1 clean run-off volume of 34 000 m^3. The pipe has been sized to completely transfer the 1:50 rainfall event of volume in 3 number of days at rate 0.13 m^3/s.

The penstock has been designed to take inflow immediately, leaving no time for ponding during the 1:50 rainfall-event. Refer to **Appendix D-1** and **D-2** for penstock sizing (including inlet) and Pipe classification respectively. The penstock will be concrete encased to the dimensions and specifications seen in drawing 301-00825/001-158 The encasing has been designed to take the load which will be imposed by the anticipated 90m high ash overburden.

10.4.1.3 CLEAN RUN-OFF DIVERSION AT CLOSURE OF FACILITY

At closure of the facility the penstock facility will need to be sealed and decommissioned as no personnel will be available to monitor its functionality. The clean run-off at closure will be diverted using a perimeter gravity drain flowing in a north-east direction. The drain will be earth lined, with base dimension of 2 and side slopes 1:2 and 1 m deep. The drain only diverts a portion of the clean catchment. The remaining catchment run-off volume will be left to evaporate as it will be trapped in the natural valley depression outside of the ash dump footprint.

10.4.2 DIRTY STORM WATER RUN-OFF MANAGEMENT

10.4.2.1 CURRENT DIRTY RUN-OFF SYSTEM

The existing ash facility is not lined, therefore there are no measures currently in place to control and monitor seepage. However, there is a storm water management system which involves the collection of the dirty run-off via perimeter dirty water drains discharging into the three existing dams namely Ash Dump Dam 1, Ash Dump Dam 2 and Metsemaholo dam. The surface water runoff is contained in the dirty water system and used for irrigation for dust suppression by spraying the exposed ash face, as well as for irrigation on the rehabilitated areas. The site is in a high evaporation area, so a significant volume of water will be lost through evaporation on the dams.

The existing ash dump storm water management system consists of stepped chutes down the faces of the final ash dump slopes (spaced down the slopes at intervals around the facility) to collect the run-off from the top of the ash dump and convey the flow down the ash dump face into the concrete lined trapezoidal channel along the toe of the facility. Energy dissipation devices are present at the entry point to the concrete lined trapezoidal channel. Which will subsequently flow through a sediment trap before flowing into the three dams mentioned above.

10.4.2.2 DIRTY STORM WATER RUN-OFF FOR THE ASH DUMP EXTENSION

The proposed new ash dump extension facility will be lined in 4-year stages, meaning that the liner with a 300mm earth-fill protection on top will be exposed during progressive stacking on the area. This will reduce the time of rainfall run-off concentration. Dirty storm run-off will also arise from the 40 m high advancing ash face.

For phase 1 and 2 of the lined deposition footprints (years 0 to 4 and 5 to 8), dirty storm water run-off from the advancing ash deposit face and from the remainder of the exposed phase footprint, will flow naturally to the south and enter the south dirty water perimeter drain via concreted weirs cut through the perimeter toe wall. As the lowest weir will be ashed over soon after ashing to the current phase commences, a second slightly higher weir is provided for each phase lined area. A typical weir detail is presented in drawing 301-00825/001-153. This drain will require the southern terrace of the lined area to be shaped after topsoil and earth-fill removal, to drain towards the west corner of the lined area and into the East dirty water drain. This is to ensure that the entire volume of dirty run-off flows through the weirs. Drawing 301-00825/01-158 shows the selected typical section through the weir. For the location of the weirs for phases 1 and 2, see drawings **301-00825-001-153** and 301-00825/01-207

The terrace for phase 1 and 2 will be shaped towards the southernmost of the lined footprint. This will allow the run-off to flow towards the southern direction and in the south-west direction along the southern toe wall to drain to the dirty trench using the weir in the south-west end of the lined footprint. The weirs will be used to release water trapped along the western toe of phase 1 and 2 when the ridge splits the lined catchment.

Dirty storm water run-off from the remaining lined deposition areas phases 3 to 8 (years 9 to 31) will collect in the valley zone against the slopes of the active face and on top of the liner since the stacking is up-slope. The lowest point will always be against the advancing ash slopes. A reverse slope trench has been devised to remove the dirty stormwater away from the advancing ash face and to transfer it to the West perimeter wall of the current phase, from where it will be transferred by pump to the permanent South or North perimeter dirty drains.

The ash dump perimeter dirty water drains are at a higher elevation relative to the trapped dirty pond, therefore requiring pumping to lift the accumulated dirty water to the perimeter dirty drains(transfer to the perimeter drains in a buried decant pipeline has been ruled out due to the high probability of blocking with ash).

For phases 3 to 5, the dirty run-off will be pumped by to the South dam. This will be achieved by pumping to a removeable 160ND PE80, PN4 HDPE pipeline running on the crest of the divider wall and discharging to the perimeter dirty drain.

For phases 6 to 8 the pumped pipeline will be re-located to deliver to the North dirty water drain

10.4.2.3 DIRTY WATER PUMPING - TRENCH AND TRAILER DIESEL PUMP PLATFORM

To remove the pond, for lined area phases 3 to 8, it is proposed to excavate a deep trench 5 m wide and sloping at 1:200 downwards (westwards) from the ponded water at the advancing ash face, to the to the west divider wall of the lined area. This trench will be lined and will create an artificial low point against the west divider wall of the lined area. Water will gravitate and pond within the trench against the west wall. This allows a small, trailer mounted, self-priming diesel pump with its suction in the deep trench, to be used to transfer the dirty storm water to the perimeter dirty drains at higher elevation.

The deep point within the trench provides the required water depth for the pump to operate. The pump can be mobilised to the trench during or after a rainfall event.

At the final stage 8 footprint stage when stacking has ceased on the bottom radial deposition layer, an elevated platform will be constructed for the permanent dirty trench on the northern face of the proposed facility to ensure that the run-off captured from the slopes can flow through gravity to the north dam.

A trailer mounted CP150i auto prime pump unit (or equal approved) with capacity of 500 cubic metres per hour and a 5m suction lift, will be used to lift the dirty run-off by pipeline to the perimeter dirty drain. The pump is trailer mounted and consists of a 150 mm suction and 150 mm discharges pipe and will transfer the 1:50 storm volume into the dirty drain in 3 days when pumping nonstop.

The North dam and dirty drains are utilized for phases 7 and 8. For phases 1 to 6 the dirty run-off will be pumped to the South dam.

See **Appendix D-4** for the trailer mounted pump specifications.

10.4.2.4 DIRTY STORM WATER RUN-OFF FOR THE EXTENSION AREA AT CLOSURE

At closure, the natural topography drains to a low point which lies halfway the final northern wall length. Meaning the dirty and clean water will be trapped against the wall at this low point as deposition progresses and resulting in ponding. To prevent entrapment of the dirty water from the rehabilitated slopes, an elevated platform will be constructed along the length of the northern wall to allow for dirty run-off to be captured and drained towards the new North dam.

The elevated platform will be on the same elevation as the toe wall at closure. This platform will allow for the dirty channel slope in the western-easterly manner for flow to gravitate to the northern return water dam. An alternative would be to backfill the footprint to level, and have it slope towards the perimeter dirty drains, but this option was deemed not economical due to the extremely large backfill volumes require and has thus been ruled out.

10.4.2.5 DIRTY STORM WATER RUN-OFF SURFACE COLLECTION DRAINS

The existing ash dump storm water collection system uses take-down pipes along the rehabilitated slopes of the existing dump. The pipes run along the slopes and discharge into energy dissipaters before discharging to the dirty trench at the toe of the facility.

For the ash dump extension, a 450 mm HDPE pipes is proposed to convey the run-off from the rehabilitated slopes to the bench between the lower and upper stacks. At the bench level, the pipe will run at a fall of 1:100 towards the lower slope face. The pipes will be installed once the closure slope of 1:5 has been achieved prior to topsoiling and rehabilitation of the reshaped closure slopes.

The pipes will discharge into an energy dissipater on the downstream toe of the closure slope, and subsequently into the dirty drain. Reshaping of the operation slope of 1:1.5 to a closure slope of 1:5 will be done every two shifts. The pipes will be installed at every four shifts (approximately 240 m centre to centre).

10.4.3 CONCLUSION

After consideration of options, the following is recommended for implementation:

- Clean storm water run-off:
- Clean run-off will be diverted away from the active ash dump footprint and not stored on the system using the following methods:
- For lined area phases 1 and 2 (years 1 to 8), the west edge gravity drain has sufficient positive slope (i.e. from north to south direction) to transfer the water by gravity to the environment for lined area phases 3 to 8 (years 9 to 33), a buried gravity decant facility with drop inlets will be utilised in the valley pond area for each lined deposition area.
- Dirty storm water run-off:
- Gravity discharge is possible for dirty water transfer to the perimeter dirty water drain and hence to the return water dams for phases 1 and 2.
- Dirty water transfer to the perimeter dirty water drain in phases 3 to 8 requires a trailer mounted de-watering pump to lift the water from its low elevation into the perimeter dirty water drain.
- Upon completion of stacking on top of the existing facility, it is recommended for Dam 02 (South-east dam) to be converted into a clean water dam to capture run-off from rehabilitated slopes. This run-off can be used for irrigation on the rehabilitated slopes during the dry season.

10.5 STORMWATER CHANNEL DESIGN

The main purpose of the system is to effectively convey and control large floods of 1:50 year recurrence interval. All diversion channels have been sized to divert the clean water runoff for the 50-year return period flood peak, as per GN 704 shown in **Table 10-1**.

The results show that multiple channels are at risk of eroding, due to the maximum velocity being greater than 3m/s. The high velocities are due to the steep catchment gradients present on the site. Therefore, concrete lining is required for the channels. This channel lining, when implemented, will greatly reduce the risk of erosion. Another option would be to implement energy dissipation devices. Energy dissipaters are systems designed to protect downstream areas from erosion by reducing the velocity of flow to acceptable limits and will be utilised in the clean water diversion channel. Baffle blocks will be installed at the end of each clean water channel to dissipate energy.

Table 10-1: Phase 1 channels characteristics

Phases	Clean water diversion channel							
	1	2	3	4	5	6	7	8
Catchement Area (km^2)	1.60	1.12	0.95	0.71	0.54	0.40	0.30	0.24
Peak flow (1:50) (m^3/s)	4.40	3.44	2.60	1.89	1.41	1.02	0.74	0.36
Side Slopes (m/m) 1V:XH	2							
Base width (m)	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Depth (m)	0.74	0.63	0.53	0.437	0.364	0.298	0.244	0.157
Freeboard (m)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Design depth (m)	1.0	0.9	0.8	0.7	0.7	0.6	0.5	0.5
Normal velocity (m/s)	3.65	3.045	2.77	2.52	2.3	2.075	1.87	1.46
Lining	Concrete							

The following approach and criteria were used in carrying out the drainage system design analysis:

- Catchment run-off was been computed using the rational method.
- Delineation of dirty and clean catchments.
- Channels assumed to follow natural ground.
- Manning's equation used to size the channels.
- Relevant lining selected to avoid erosion.
- Outlets structure at clean water diversion channels designed in a manner that dissipates flow energy.

11.0 ACCESS ROADS AND FENCING

11.1 EXISTING ROADS

The existing ash dump roads vary in width from 5m to as wide as 8m. the existing road around the fence has a width of around 6m, with the smaller internal roads varying between 5m and 6m. the existing road around the fence will connect to the new ash dump perimeter road in the south and western side of the ash dump to allow for easy access to the dump. The 3 existing roads in the north will each connect to the north area of the perimeter road. **Figure 11-1** below shows the overall site layout and roads.

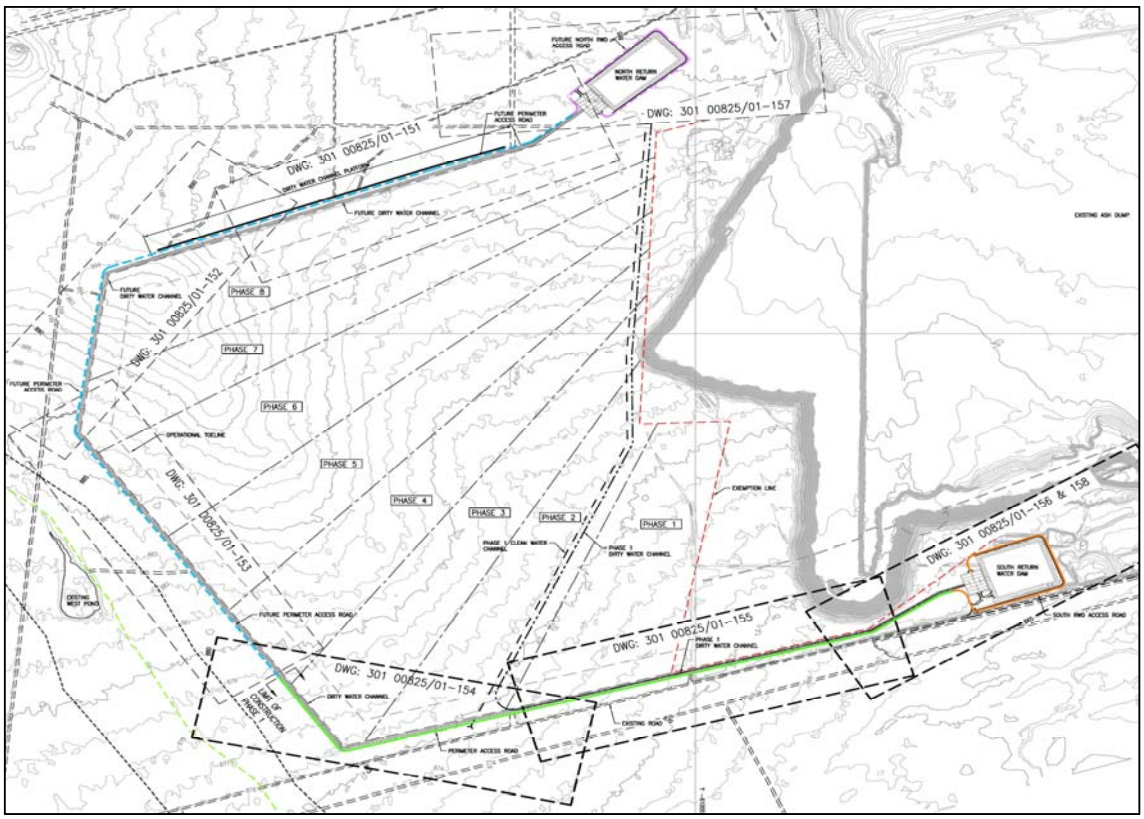

Figure 11-1: Site layout and roads

11.2 FENCING

11.2.1 EXISTING FENCE

The existing fence currently runs around the Matimba ash dump boundary, next to the existing road. The existing fence will not be affected by the Ash Dump development.

11.2.2 NEW FENCE

A new fence will be constructed around the Matimba return water dams and silt traps based on the existing fence configuration already installed around the existing dams. The fence will go around the dam and will have one access gate for the dam inspection and cleaning.

It is expected that the existing fence around the Matimba ash dump site boundary will be maintained. The fence currently encloses the existing Matimba ash dump and the road around the ash dump will allow for patrolling and monitoring activity around the ash dump site and other Eskom properties located within the fenced area.

11.3 ASH DUMP PERIMETER ACCESS ROAD

The perimeter road will connect both the south and north raw water dams and will run along the dirty water channel to allow for access to channel inspection and maintenance at any given time. The perimeter road will connect with the existing roads at various points in the north, west and southern sides of the ash dump to allow access to areas around the ash dump**. Figure 11-2** shows the layer works details.

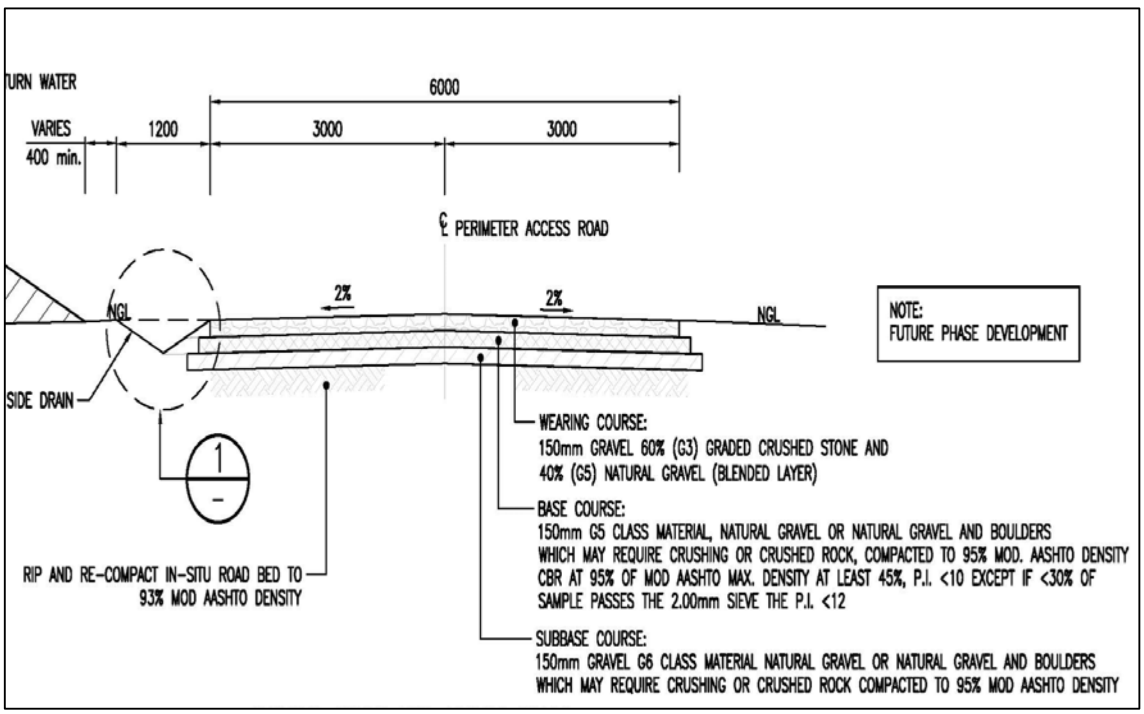

Figure 11-2: Access road layer works

11.3.1 GEOMETRIC DESIGN

11.3.1.1 WIDTH

The new perimeter access road is designed with a width of 6m to allow for traffic in both directions. The width will allow for passenger vehicles and maintenance trucks to access the dump and the dams for maintenance related and cleaning of the silt traps.

11.3.1.2 CROSSFALL/CAMBER

The road will have a camber of 2% fall from the centreline of the road to facilitate proper runoff from the road and thereby eliminating any possibility of standing water on the road. The water runoff from the road surface will drain to the drain/channels on either side.

11.3.1.3 CURVE RADIUS

The perimeter access road will have curves with sufficient safe horizontal turning radius for vehicles travelling at low speeds of not more than 40km/hr. the curve radius is not designed for high speed and a speed limit of 40km/hr maximum should be the allowable at all times. The minimum radius of curve along the road is 30m at the north west and south west corner of the perimeter road.

11.3.1.4 INTERSECTIONS

The intersection points are designed at 3 points in the north, 1 point in the west and 3 points in the south sides of the perimeter ash dump road. these intersections serve to connect the new perimeter road with the existing internal roads and road along the fence. The minimum turning radius of the curve at the intersection is designed at 7m. **Figure 11-3** below shows the perimeter road and existing road intersection points at the north perimeter access road.

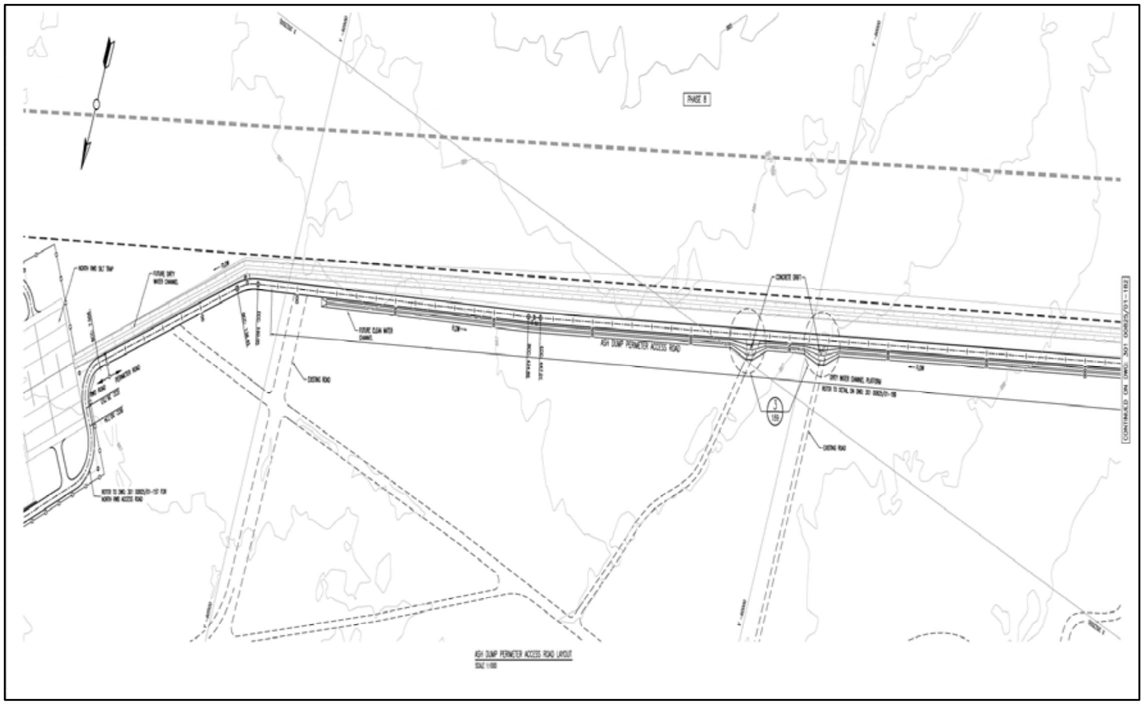

Figure 11-3: North perimeter road intersection points

11.3.2 ROAD DEVELOPMENT

The access roads will be developed in 8 phases as the liner works progresses. The development phases are explained in **Section 3.1** of this report.

The perimeter road sections will be developed as per the chainages and length detailed below:

Table 11-1: Access Road Development Phases

Development Phase	Road Section (Chainage) m	Road Length to be constructed (m)
1	3,600-5,120	1,520
2	3,200-3,600	400
3	3,200-2,900	300
4	2,600-2,900	600
5	2,300-2,600	300
6	2,000-2,300	300
7	1,700-2,000	300
8	0-1,700	1,700

11.3.3 DRAINAGE

A clean water trench with base width 1.5 m wide will run along the north perimeter road and collect water to the penstock that will drain water by means of a gravity pipeline under the ash dump to the environment on the south of the ash dump.

Concrete drift will be provided in the intersection point of the perimeter road and the existing roads to facilitate stormwater drainage through the road from the perimeter channel to the penstock. The drift will be a trapezoidal shape with 3m base width and 1: 5 slopes to allow for vehicles to pass over it easily. See **Figure 11-3** above.

11.3.3.1 FLOOD PEAK CALCULATIONS

In order to produce peak runoff input, rational method was used to determine design flood peaks for the delineated catchment based on its applicability to the catchment area.

The Rational Method (RM) was used for the peak flow run-off calculations and is one of the most widely used methods for the calculation of peak flows for small catchments (< 15 km^2). The formula indicates that Q = CIA, where I is the rainfall intensity, A is the upstream runoff area and C is the runoff coefficient. Q is the peak flow. The point precipitation was determined using the Depth-Area-Duration-Frequency relationships, HRU Report 2/78 (Midgley and Pitman, 1978).

11.3.3.2 FLOOD PEAK RESULTS

Peak flood flows for the 1 in 2 year, 1 in 5 year, 1 in 10 year, 1 in 20 year, 1 in 50 year and 1 in 100-year recurrence interval storm events were estimated for the delineated catchment using the above-mentioned methods. Calculations were based on current conditions at the project site. The estimated peak flows are presented in **Table 11-2** for the 1 in 2 year, 1 in 5 year, 1 in 10 year, 1 in 20 year, 1 in 50 year and 1 in 100-year recurrence intervals. See section 11.4.2.2 below for details.

11.4 NORTH RETURN WATER DAM ROADS

Access to the North return water dam will be from the south access along the perimeter road, existing road on the west and existing access road on the eastern side. A ramp with a slope of 1V:10H will be constructed on the west side of the dam, to facilitate access to the crests of the dam. The RWD will also have a ring road 6m width and centreline horizontal curve radius of 20m on the bends. the ring road will provide access to the silt trap north compartment for maintenance and cleaning. The **Figure 11-4** below shows return water dam road layout.

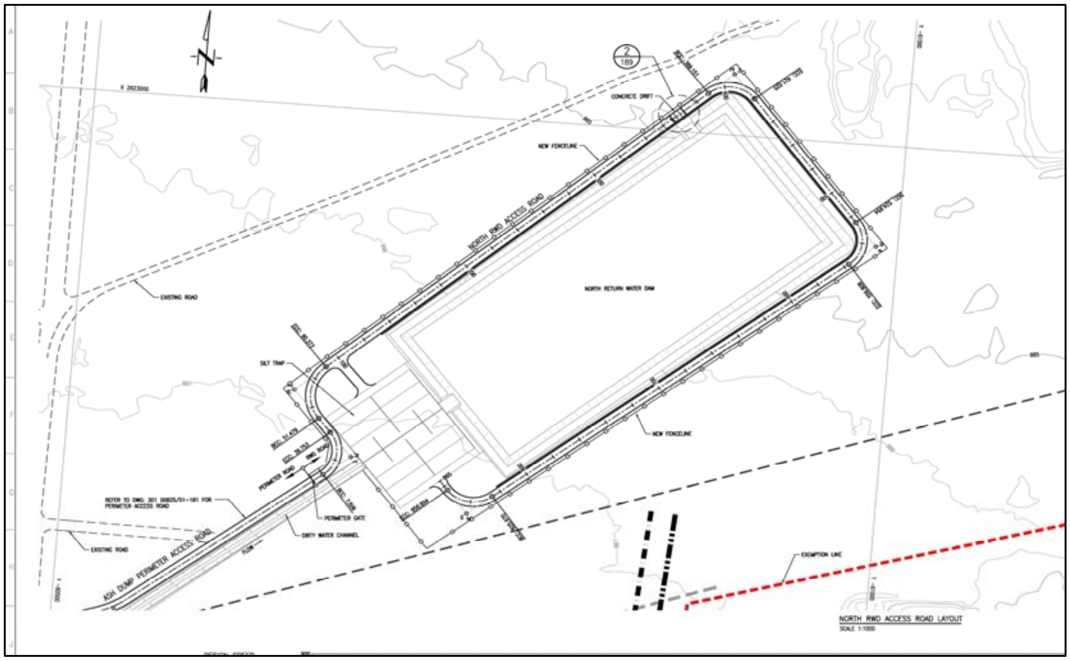

Figure 11-4: North return water dam road layout

11.4.1 GEOMETRIC DESIGN

11.4.1.1 WIDTH

The new access ring road is designed with a width of 6m to allow for traffic in both directions. The width will allow for passenger vehicles and maintenance trucks to access the silt traps and dam for maintenance and cleaning.

11.4.1.2 CROSSFALL/CAMBER

The road will have a camber of 2% fall from the centreline of the road to facilitate proper runoff from the road and thereby eliminating any possibility of standing water on the road.

11.4.1.3 CURVE RADIUS

The ring road will have curves with sufficient safe turning horizontal radius for vehicles travelling at low speeds of not more than 40km/hr. the curve radius is not designed for high speed and a speed limit of 40km/hr maximum should be the allowable at all times. The minimum radius of curve is 20m.

11.4.2 DRAINAGE

11.4.2.1 SIDE CHANNELS

The North RWD ring road will be provided with side drains to collect runoff from the road, outside walls of the dam and surrounding catchment. The drains are sized to convey a 1:50year peak discharge of 0.104m^3/s. The side drain is sized with a 0.3 m depth with 1:2 side slopes and will discharge to a trapezoidal concrete drift through the ring road to the downstream catchment and to stream.

The side drains will have to be grass lined and topsoil can be used to rehabilitate the channels.

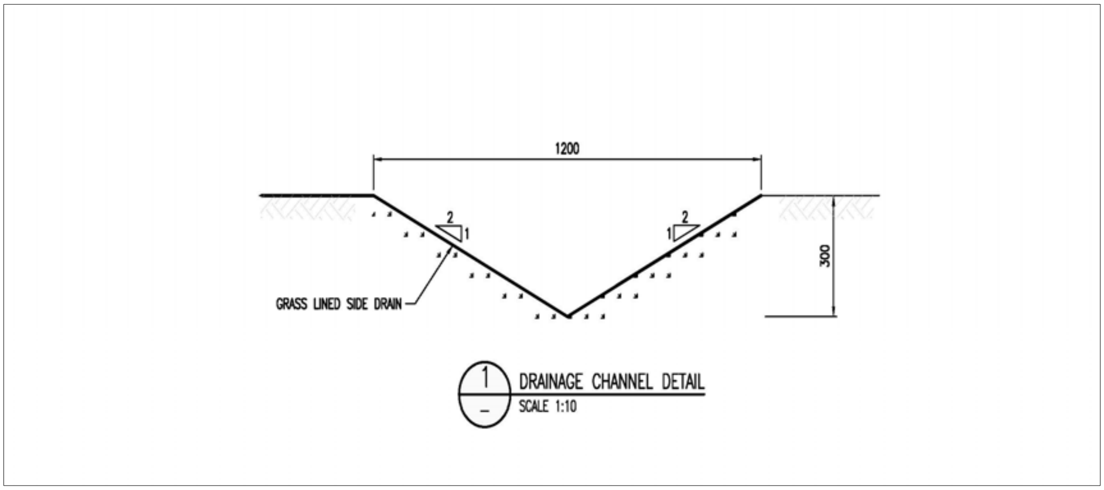

Figure 11-5: Side channel detail

11.4.2.2 FLOOD PEAK RESULTS

The Rational Method peak flows were selected for use in the channel sizing analysis.

Table 11-2: Summary of the peak flows

Flood Calculation Method	Return Period (years)					
	2	5	10	20	50	100
Selected Peak Flows(m^3/s)	0.022	0.034	0.049	0.071	0.104	0.156

The rainfall depths with durations corresponding to the Time of Concentration (Tc) for any sub catchment were used to calculate peak flows for the catchment. The underlying assumption is that the largest possible peak flow is obtained when the storm rainfall event has duration equal to the time required for the whole catchment to contribute runoff at the outlet. A short description of the above-mentioned methods is given below.

11.4.2.3 CONCRETE DRIFT DESIGN

A concrete drift is provided in the in the South side of the dam to facilitate drainage from the dam wall outside slopes and the outlet spillway. The drift will be a trapezoidal shape with 3m base width and 1:5 slopes to allow for vehicles to pass over it easily. The concrete drift is designed with enough capacity to convey the overflow from the spillway without overtopping.

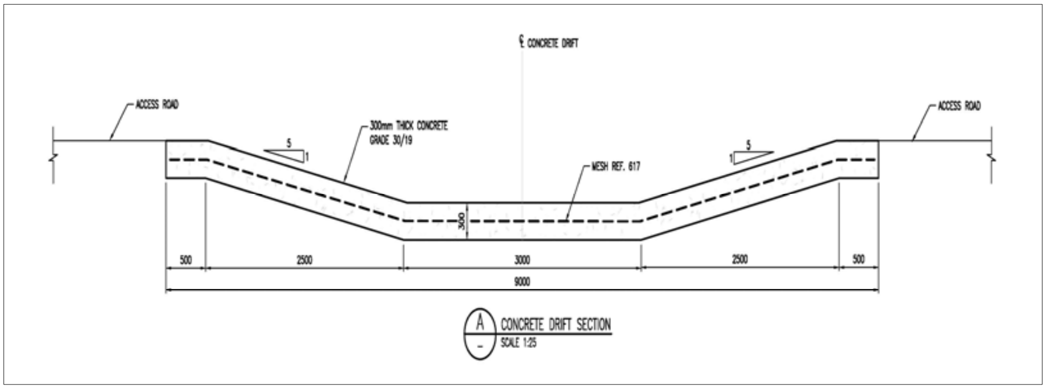

Figure 11-6: Concrete Drift Detail

11.5 SOUTH RETURN WATER DAM ROADS

Access to the South return water dam will be from the eastern side along the perimeter road and existing road on the southern side. A ramp with a slope of 1V:10H will be constructed on the western side of the dam, to facilitate access to the crest of the dam. The dam will have a ring road that connects the existing road and provide access to the silt trap on the northern side of the silt trap. The ring road will have a 6m width and centreline curve radius of 20 m.

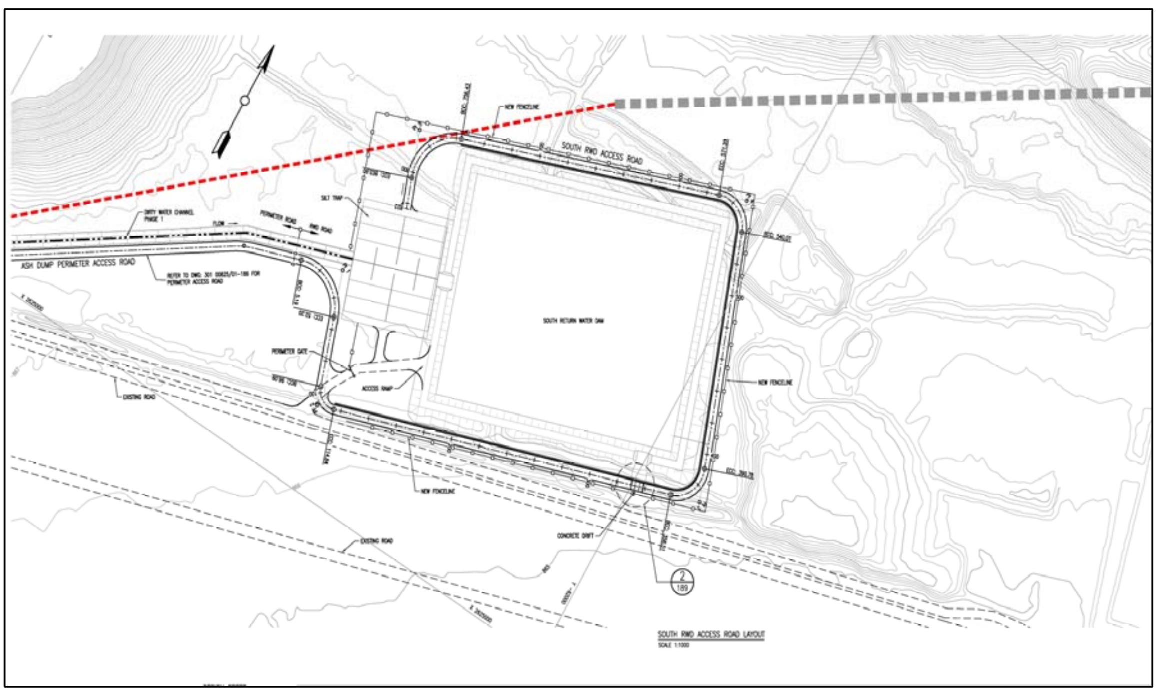

Figure 11-7: South Return Water Dam Road Layout

11.5.1 GEOMETRIC DESIGN

11.5.1.1 WIDTH

The new access ring road is designed with a width of 6m to allow for traffic in both directions. The width will allow for passenger vehicles and maintenance trucks to access the silt traps and dam for maintenance and cleaning.

11.5.1.2 CROSSFALL/CAMBER

The road will have a camber of 2% fall from the centreline of the road to facilitate proper runoff from the road and slopes of the dam walls thereby eliminating any possibility of standing water on the road.

11.5.1.3 CURVE RADIUS

The ring road will have curves with sufficient safe turning horizontal curve radius for vehicles travelling at low speeds of not more than 40km/hr. the curve radius is not designed for high speed and a speed limit of 40km/hr maximum should be the allowable at all times. The minimum radius of curve is 20m.

11.5.2 DRAINAGE

11.5.2.1 SIDE CHANNEL

The South RWD ring road is designed with side drains to collect runoff from the road, outside walls of the dam and surrounding catchment. The drain is sized to convey a 1:50year peak discharge of 0.109 m^3/s. The triangular side drain is sized with a 0.3 m depth, with 1:2 side slopes and will discharge through a trapezoidal concrete drift downstream the existing road to the downstream catchment and to stream.

The side drain will have to be grass lined and topsoil can be used to rehabilitate the channels.

11.5.2.2 FLOOD PEAK RESULTS

The Rational Method peak flows were selected for use in the channel sizing analysis.

Table 11-3: Summary of the peak flows

Flood Calculation Method	Return Period (years)					
	2	5	10	20	50	100
Selected Peak Flows (m^3/s)	0.023	0.036	0.049	0.069	0.109	0.160

The roads width will be 6m and comprise a base, sub-base and subgrade. The layer specifications are shown on Drawing No. 301-00825/01-105 and summarised in **Table 11-4.**

11.5.2.3 CONCRETE DRIFT DESIGN

A concrete drift is provided in the in the South side of the dam to facilitate drainage from the dam wall outside slopes and the outlet spillway. The drift will be a trapezoidal shape with 3 m base width and 1:5 slopes to allow for vehicles to pass over it easily.

The concrete drift is designed with enough capacity to convey the overflow from the spillway without overtopping.

11.5.3 ACCESS ROAD LAYER SPECIFICATIONS

Table 11-4: Road Layer Specifications

Layer name	Thickness (mm)	Type
Wearing Course	150	Gravel wearing course, G3 (40%) and G5 (60%) mix class material, compacted to 95% modified AASHTO density (compacted layer thickness 150mm)
Base	150	G5 Gravel base course, G5 class material, compacted to 95% modified AASHTO density (compacted layer thickness 200mm)
Sub-base	150	G6 Gravel subbase, G6 class material, compacted to 95% modified AASHTO density (compacted layer thickness 200)
Subgrade	150	In-Situ roadbed to 93% modified AASHTO density:

The material for road construction will be sourced from available and approved suppliers close to the area.

11.6 MAINTENANCE ACCESS

Maintenance access for vehicles is provided at each dam by inclusion of an access ramp to the embankment crest with a slope of 1V:10H. Access to the silt traps is then provided by means of an access road and ramps on each compartment. The ramps will allow for cleaning and maintenance activities in the silt trap. **Figure 11-8** below shows the Access Ramp Layout to the dam crest and road to silt trap.

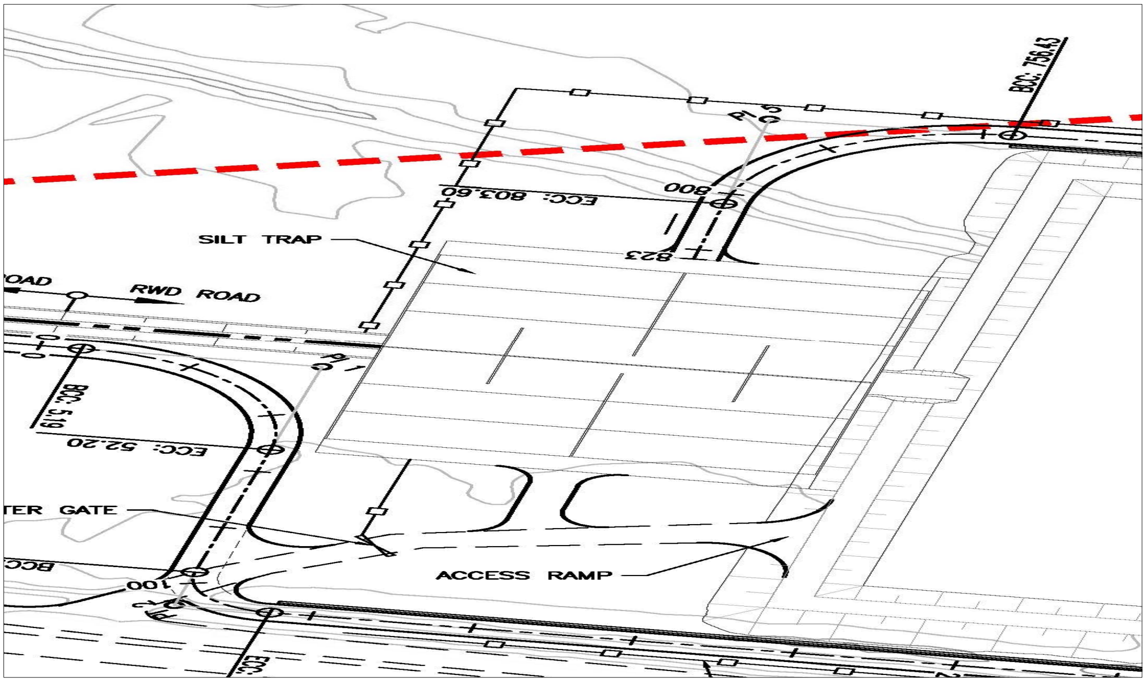

Figure 11-8: Access Ramp Layout

12.0 SCHEDULE OF WORKS

12.1 ASH DUMP EXTENSION DEVELOPMENT PHASES

The Client has requested that the ash dump extension be developed in four-year modules, to spread the capital costs over the life of the dam. The following table sets out the development phases and associated time scales:

Table 12-1: Development phases

DEVELOPMENT PHASE	DESCRIPTION	START DATE	END DATE	CONSTRUCTION START DATE TO BE READY FOR THE START OF THE NEXT PHASE	CONSTRUCTION COMPLETION DATE
0	Exemption Area + Piggyback on Existing Ash Dump	Apr 2019	Apr 2024	Civil Construction: Dec 2022	Mar 2024
1	Year 1 to 4 ashing	Apr 2024	Aug 2028	Civil Construction: Apr 2027	July 2028
2	Year 5 to 8 ashing	Aug 2028	Feb 2032	Civil Construction: Oct 2030	Jan 2032
3	Year 9 to 12 ashing	Feb 2032	Dec 2035	Civil Construction: Aug 2034	Nov 2035
4	Year 13 to 16 ashing	Dec 2035	Nov 2039	Civil Construction: Jul 2038	Oct 2039
5	Year 17 to 20 ashing	Nov 2039	Nov 2043	Civil Construction: Jul 2042	Oct 2043
6	Year 21 to 24 ashing	Nov 2043	Jan 2048	Civil Construction: Sep 2046	Dec 2047
7	Final Year 25 to 28 ashing	Jan 2048	Feb 2051	Civil Construction: Oct 2049	Jan 2051
8	Final Year 29 to 31 ashing (3year period)	Feb 2051	Feb 2054	N/A	N/A

Construction/Procurement Components for each phase are as follows:

Phase 0 Period (0-4 years construction works): (for completion by March 2024, ready for Phase 1 ashing):

- Procure and install new conveyor equipment and spreaders

- Bush clearing of phase 1 liner area
- Top-soil stripping and stockpiling of phase 1 area
- Stripping and stockpiling of selected soil from the phase 1 area for the 300mm protection layer over the phase 1 liner
- Construct the first section of the clean water decant diversion pipe under the liner
- Install the Liner system for the phase 1 area
- Construct the phase 1 area roads, perimeter drains and intermediate clean and dirty water drains along the edge of the phase 1 area
- Construct the new South Return Water Dam, silt traps and irrigation water dams
- Construct the new irrigation water pumphouses
- Procure and install irrigation water pumps and pipelines
- Procure three new trailer mounted, diesel driven de-watering pumps for the clean and dirty water system (one as stand-by)

Phase 1 Period (for completion by July 2028, ready for Phase 2 ashing):

- Bush clearing of phase 2 area
- Top-soil stripping and stockpiling of phase 2 area
- Stripping and stockpiling of selected soil from the phase 2 area for the 300mm protection layer over the phase 2 liner
- Install the Liner system for the phase 2 area
- Construct the phase 2 area roads, perimeter drains and intermediate clean and dirt drains along the edge of the phase 2 area

Phase 2 Period (for completion by January 2032, ready for Phase 3 ashing):

- Bush clearing of phase 3 area
- Top-soil stripping and stockpiling of phase 3 area
- Stripping and stockpiling of selected soil from the phase 3 area for the 300mm protection layer over the phase 3 liner
- Install the Liner system for the phase 3 area
- Construct the phase 3 area roads, perimeter drains and intermediate clean and dirt drains along the edge of the phase 3 area

Phases 3 to 8 Periods (for completion by January 2051, end of ash dump life). For Each Phase:

- Bush clearing of next phase area
- Top-soil stripping and stockpiling of next phase area
- Stripping and stockpiling of selected soil from the next phase area for the 300mm protection layer over the phase 4 liner
- Install the Liner system for the next phase area
- Construct the next phase area roads, perimeter drains and intermediate clean and dirt drains along the edge of the next phase area
- Construct the new North Return Water Dam (Phase 5)

12.2 COST ESTIMATE

A development of the detailed schedule of quantities and cost estimate was carried out and developed by KP as part of our scope and it is submitted as part of the detailed design phase.

The project construction costs are broken down into the first 4-year phases and are estimated to be **R 1,345,054,065.68** and a total of **R 1,546,812,175.53** when including 15 percent contingency.

The rates used were sourced from latest rates by QS Africa Quantity Surveyors & Construction Consultants.

Table 12-2 and **Table 12-3** below summarizes the project costs for the first 4-years and balance of development:

Table 12-2: Civil works cost estimate breakdown

Phase	Cost with 15% Contingency
1st 4-years	R 405,004,547.92
2nd 4-years	R 144,702,258.32
3rd 4-years	R 157,070,672.88
4th 4-years	R 162,561,394.87
5th 4-years	R 165,760,642.58
6th 4-years	R 180,244,155.10
7th 4-years	R 150,778,427.44
8th 4-years	R 197,641,618.67
Total Project Costs	**R 1,563,763,744.78**

Table 12-3: Conveyor cost estimate breakdown

Phase	Cost	With 15% Contingency
1st 4-years	R 158,335,000.00	R 182,085,250.00
Total Project Costs	**R 158,335,000.00**	**R 182,085,250.00**

Refer to **Appendix F** for a full cost breakdown.

13.0 CONCLUSION & RECOMMENDATION

The optimum design by means of radial ashing will result in the most effective ashing deposition at the Matimba Power Station ash dump. It is anticipated that the ash dump will reach full capacity in 2055 using radial ashing.

It is recommended that the Operating and Maintenance manual be studied in conjunction with this detailed design report to ensure the design is executed accordingly.

14.0 REFERENCES

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15.0 CERTIFICATION

This report was prepared and reviewed by the undersigned.

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Approval that this document adheres to Knight Piésold Quality Systems: ✓