WESIZWE PLATINUM LIMITED

TAILINGS STORAGE FACILITY

WASTE CLASSIFICATION, CONTAINMENT BARRIER SYSTEM DESIGN AND ASSOCIATED STRUCTURES

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REPORT 301–00509/02/R1

JANUARY 2016

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WESIZWE PLATINUM LIMITED

TAILINGS STORAGE FACILITY

ADDITIONAL DESIGN FOR SUBMISSION TO DWS FOR APPROVAL

WASTE CLASSIFICATION, CONTAINMENT BARRIER SYSTEM DESIGN AND ASSOCIATED STRUCTURES

REPORT 301 – 00509/02

JANUARY 2016

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WESIZWE PLATINUM LIMITED

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JANUARY 2016

1 INTRODUCTION

Wesizwe Platinum has appointed Knight Piésold (Pty) Ltd (KP) to prepare a design for the proposed Tailings Storage Facility (TSF) for Wesizwe Platinum Limited (Wesizwe). The proposed TSF and associated infrastructure will be located in the North West Province, in the Bojanala District, East of Phatsima village and approximately 7 km South-West of Sun City. The locality map for the proposed site is shown inFigure 1-1 below.

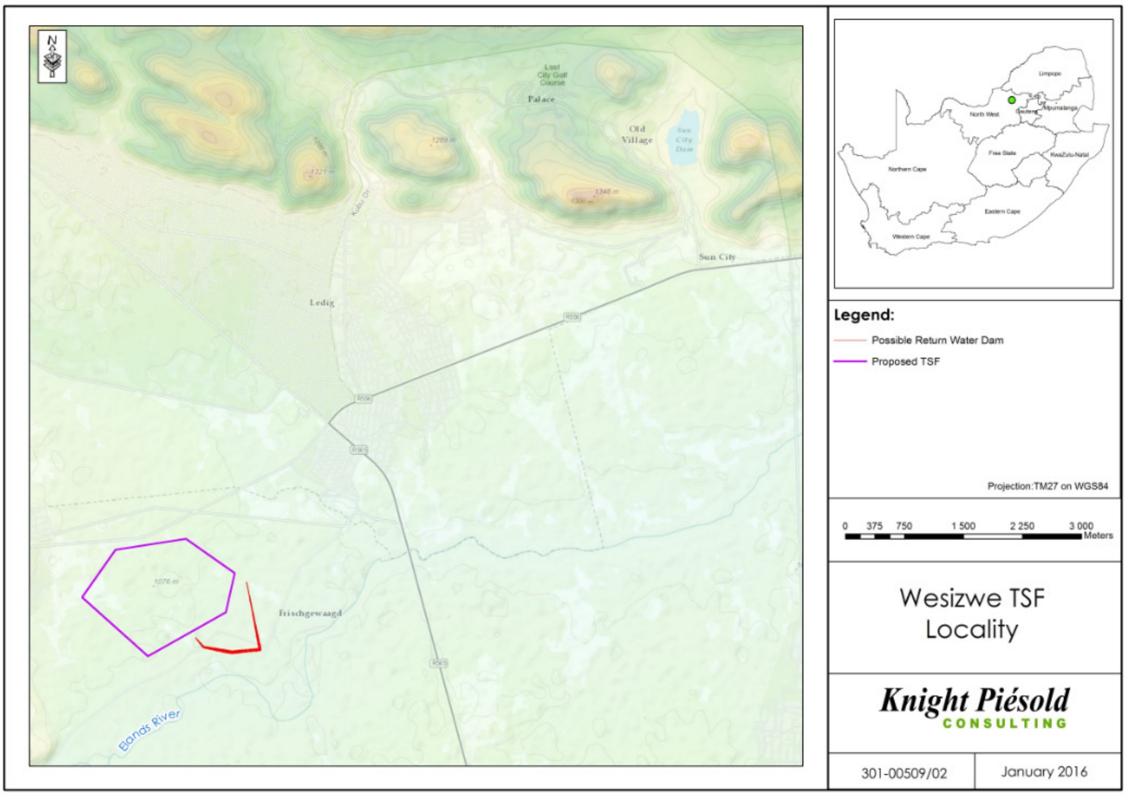

Figure 1-1 Locality map for the project

The design of the proposed TSF and associated infrastructure, (including the barrier system) have been undertaken to adhere to the minimum requirements set out in the National Environmental Management: Waste Act of 2008, Regulation 636. In the regulation, attention is drawn to Clause 3 (1) and (2) requirements and these are presented in **Table 1-1,** These clauses set out the parameters for the design and mitigation measures required for this development. This report describes the design of the TSF and associated infrastructure and the findings of the relevant investigation carried out during the feasibility study.

Table 1-1 Regulation requirements and proposed designs

Clause 3 – Landfill Classification and Containment Barrier Design	Comment/ Reference Docs
(1) The containment barriers of landfills for the disposal of waste in terms of section 4 of these Norms and Standards must comply with the following minimum engineering design requirements – (i.e. liner requirements for various landfill classes)	Liner requirements have been determined based on Norms and Standards requirements. The waste classification report is included as **Appendix A;** and the containment barrier design drawings are included as **Appendix B**.
(2) The following containment barrier requirements must be included in an application for waste management license approval of a landfill site or cell -	See points 2(a) to 2(i) below.
(2)(a) design reports and drawings that must be certified by a registered, professional civil engineer prior to submission to the competent authority;	Feasibility study report (KP Report 301-00509/01/R1) has been certified by a registered professional engineer. Design drawings attached to this document (**Appendix B**) have been checked and signed by a professional engineer. This report discusses investigations that have been conducted, and it covers construction, operations, closure and post-closure details.
(2)(b) service life considerations that must be quantified taking into account temperature effects on containment barriers;	A literature review of the performance of a Geomembrane has been compiled and similar projects are used as a case study, this is presented in **Appendix C.**
(2)(c) total solute seepage (inorganic and organic) that must be calculated in determining acceptable leakage rates and action leakage rates;	Not a landfill site, therefore not applicable
(2)(d) alternative elements of proven equivalent performance which has been considered, such as the replacement of - (i) granular filters or drains with geosynthetic filters or drains; (ii) protective soil layers with geotextiles; or (iii) clay components with geomembranes or geosynthetic clay liners;	No alternative elements have been considered for this project. Protective geotextiles placed on top of drains are for temporary use and will be removed during commissioning of drains.
(2)(e) All drainage layers must contain drainage pipes of adequate size, spacing and strength to ensure atmospheric pressure within the drainage application for the service life of the landfill;	Designed accordingly. Seepage flows have been estimated based on seepage analyses. Stresses on drainage pipes have been estimated using “Burns and Richard solution”
(2)(f) Alternative design layouts for slopes exceeding 1:4 (vertical: horizontal) may be considered provided equivalent performance is demonstrated;	The overall side slope is 1:5 (vertical: horizontal)
(2)(g) Construction Quality Assurance during construction;	The CQA plan is attached to this document as **Appendix D**
(2)(h) Geosynthetic materials must comply with relevant South African National Standard specifications, or any prescribed management practice or standards which ensure equivalent performance; and	Specified lining material will be sourced from accredited suppliers

Clause 3 – Landfill Classification and Containment Barrier Design	Comment/ Reference Docs
(2)(i) Consideration of the compatibility of liner material with the waste stream, in particular noting the compatibility of natural and modified clay soils exposed to waste containing salts.	HDPE membrane has a very high chemical resistance and it is highly unlikely to degrade via chemical reaction. Thermal oxidation has a detrimental effect on the HDPE membrane. The higher the temperature the higher is the oxidation resulting in the degradation of membrane. Experience has shown that leachate from the under-drains generally have temperatures ranging below 25^{0}C. Previous experiments have indicated that at 20^{0}C, the service life of geomembrane may exceed 700 years. The Liner Service Life Memo is attached as **Appendix C** of this report.

1.1 Project Description

The Wesizwe Tailings Storage Facility (TSF) has been designed to accommodate tailings from the development of the Wesizwe Mine in the North West Province of South Africa. The TSF design carried out is based on an upstream development technique.

The design is based on a 25-year life of mine and a maximum capacity of 78 Million tonnes with tailings production of between 223,000 tonnes – 262,000 tonnes per month.

1.2 Design Philosophy

The design of the infrastructure was based on the criteria summarised in Table 1-2 below. The design criteria are based on the life of mine (LoM), tailings production rate, SANS 10286, South African Mine residue legal requirement and standards and industry's best practises.

Table 1-2: Design Criteria

Description	Criteria	Comment / Source
Capacity	78 Mt (Max) 70.5 Mt (Min)	25 year LoM
Tailings production rate	262 kt/month (max) 223 kt/month (min)	Worley Parsons-TWP (WPTWP)
In-Situ Density	1.6 t/m^3	Assumption based on numerous platinum tailings samples tested in South Africa from the Western Limb
Maximum rate of rise	2 m per year	Rate of rise is critical for overall stability. Good practice to achieve consolidation and facilitate upstream construction Determined from laboratory testing and previous knowledge of similar tailings.
Overall outer slopes	1:5	Facilitates rehabilitation and required for overall stability
Individual slopes between berms	1:4	Facilitates rehabilitation
Area of footprint:	1,660,000 m^2 (166 ha)	At feasibility study.
Size distribution	80%<75micron	WPTWP

Slurry Density	1.72 t/m^3	WPTWP
Particle SG	UG2: 3.76 Merensky 3.17 Average 3.3	WPTWP
Slurry delivery rate	420 t/hr	Slurry Calculation
Final Elevation of TSF	1,090 mamsl	Stage curve for this footprint shows that rate of rise at this elevation is 1.71 m/yr
Height of TSF above lowest point	46 m	Volumetric and rate of rise analysis
Slurry distribution line	250 mm rubber lined steel	WPTWP
Decant	Gravity decant with reinforced concrete towers and stacked rings	Common South African practice – minimal water retained on the TSF
Lining	Class C Landfill classification	HDPE liner with finger drains and under drainage system
Minimum Factor of Safety:	1.3 (Static operational) 1.5 (static at closure) 1.1 (With 1:475 year RI seismic event)	Accepted industry norm
Design Storm	1:50 year RI, 24 hr duration 1:100 year RI, 24 hr duration: Not to spill more than once in 50 years	Storm water will be decanted off the TSF over three days.
Return water dam (including storm water storage)	Capacity to contain three days average return water flow and the runoff from the TSF from the 50 year RI storm of 24 hr duration	Capacity 23,000 m^3 (return flow) Capacity 410,000 m^3 (storm water) Capacity 433,000 m^3 (total capacity)

2 WASTE CLASSIFICATION AND CONTAINMENT BARRIER SYSTEM REQUIREMENT

2.1 Geochemistry analysis

The tailings geochemical characterisation was conducted by WSP (refer to **Appendix A**). Two samples of tailings were provided to them for UG2 and Merensky tailings. The summary of the results are as follows:

- The numbers of samples taken were considered to be sufficiently representative for the purposes of this geochemical assessment.
- The tailings were typical Bushveld norite and pyroxenite with a high chromite content.
- Considering the results of this assessment that the waste is a TYPE 3, according to the GN636, the risk to water quality from the TSF classified as low risk. The residual risk can be managed by:
    - Isolating dust migration pathway
    - Simple vegetative capping
- The soil pH is 8.35 with no sulphide phases recorded. The tailings material is considered to be non-acid forming

Using the results from the geochemical analysis and following the process prescribed in the National Norms and Standards for waste classification, the Wesizwe tailings material was classified as a Type 3 waste. A Class C barrier system as prescribed in the Regulation 636 is shown in **Figure 2-1** below.

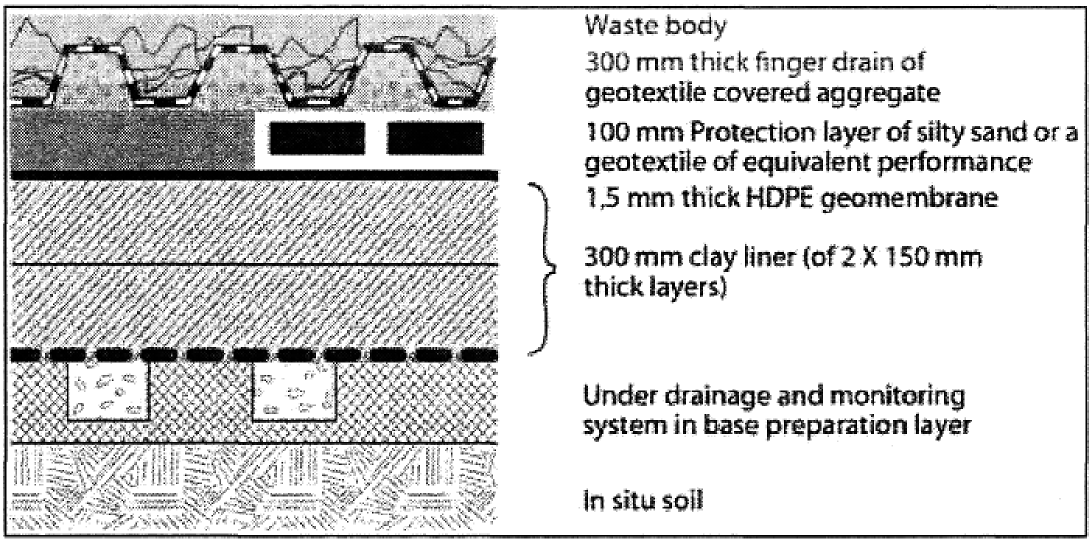

Figure 2-1 Class C Barrier System As Prescribed in the Regulation (Regulation 636)

2.2 Proposed barrier system for the TSF

The proposed design of the Wesizwe TSF barrier system, Class C is as listed below starting from the waste (Platinum tailings) to the natural ground. This is also shown in **Figure 2-2** below.

- Over liner drainage (finger drains) (not shown in Fig 2-2)
- Waste body (Platinum tailings),
- Geotextile A7 or similar approved under drains only (the protection layer will be developed by tailings deposition),
- 1.5 mm thick HDPE geomembrane (double textured)
- 300 mm thick ripped and re-compacted in-situ clay.
- In-situ undisturbed material.

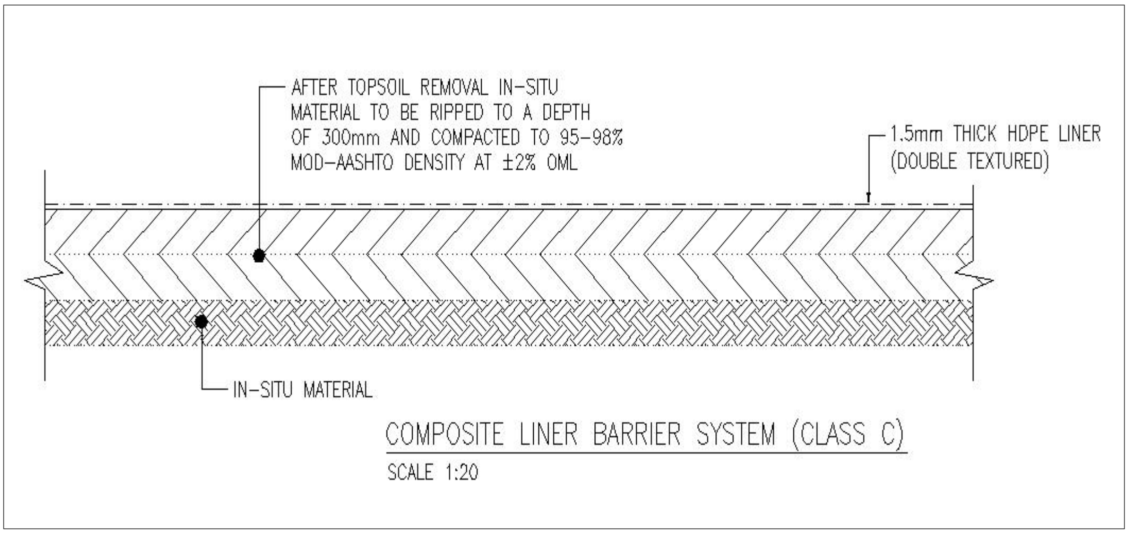

Figure 2-2 Proposed Barrier Design for the Wesizwe TSF

The above proposed barrier system is based on the geotechnical investigation which showed that the in-situ material contains natural clay. It is proposed that a 300 mm layer be ripped and re-compacted to 95% Proctor density. This is proposed as opposed to stripping of a 150 mm layer and stockpiling the material and ripping and re-compacting a 150 mm layer then bring back the previously stripped 150 mm layer and compacting it.

This might pose a risk of contaminating the material during removal from in-situ state and the cost of double handing the material. Above liner drains are specified in the design to reduce the build-up of pore pressure on the liner and increase consolidation of the tailings.

2.3 Proposed barrier system for the RWD

The Wesizwe RWD barrier system will also be a Class C liner. The layers will be as listed below starting from the supernatant water to the natural ground. This is also shown in **Figure 2-3** below.

- Supernatant water (decanted from the TSF)
- 150 mm thick geocells, filled with sand, stone soilcrete or 150 mm fibre concrete,
- Geotextile A4 or similar approved,
- 2.0 mm thick HDPE geomembrane (double textured)
- 300 mm thick ripped and re-compacted in-situ clay.
- In-situ undisturbed material.

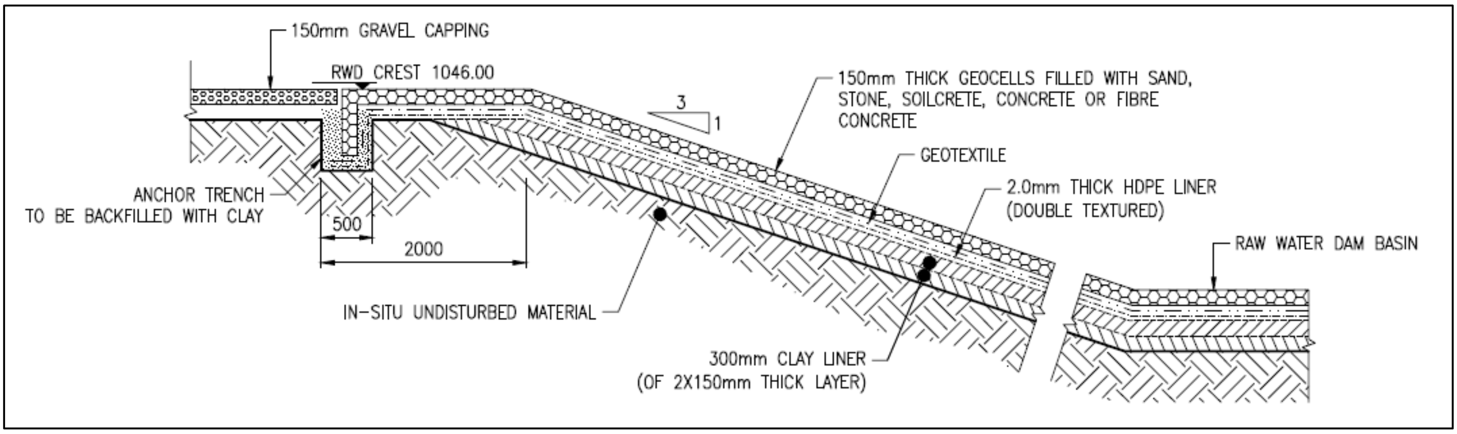

Figure 2-3 Conceptual Barrier Design for the Wesizwe RWD

3 GEOTECHNICAL INVESTIGATION SUMMARY

3.1 Regional Geology

According to the published 1:250,000 Geological Series map, 2526 Rustenburg, the site is underlain by Pyramid gabbro-norite of the Rustenburg Layered Suite, Bushveld Complex. The PGM bearing Merensky Reef and UG2 reef occur within this zone and exhibit the ore body. The Merensky Reef has its shallowest and deepest point below ground surface at respectively 584 m and 1,234 m (Wesizwe mine). The UG2 reef has its shallowest and deepest points at respectively 616 m and 1,272 m below surface (Wesizwe mine). The structural geology of the area is mostly characterized by faults and dolerite dykes (north to south striking), which have been intersected by previous drilling investigations. The Rustenburg fault line bisects the area, while the Caldera fault bisects the farm Frischgewaagd and Ledig to the east.

3.2 Site Specific Geology

According to the map, Aeolian sands occur just to the west of the site. A fault zone (Rustenburg fault) striking in a north-west to south-east direction is shown on the geological map, but was not observed in the field during the site investigation. The geological map further shows the fault intersecting most of the south-western portion of the site. The pegmatite/quartz vein has a west to east strike. According to Weinert's climatic N-value [4], the site falls in an area where the N-value is less than 5, indicating that the area is associated with more humid regions where chemical weathering is the predominant rock weathering mode. The black colluvial clay layer found predominantly in the northern part of the site contains a high content of expansive clay (montmorillonite), which forms numerous cracks in the soil layer upon drying. The clay content generally decreases with depth towards bedrock. A reddish brown/red colluvial layer occurs mostly in the southern portion of the site and also contain clay, but with higher sand contents displaying a pinhole voided soil structure. The Alluvial soils that were observed along the Elands River were not found in the site.

No ground water seepage was encountered in any of the test pits during the site investigation. Previous hydrogeological investigations found groundwater to be 20 to 40 m deep.

Two distinct geotechnical soil zones, Zone A and Zone B, occur on site. Zone A contains black sandy/silty clay colluvial deposits overlying norite bedrock. Zone B contains reddish brown/red sandy clay or clayey sand colluvial deposits also overlying norite bedrock. The contact between these two zones was clearly visible in the field and can also be seen from Google Earth Imagery.

3.3 Geotechnical evaluation and conclusions

The TSF, RWD and associated infrastructure requires foundation options for the embankment walls and the decant system (viz Penstock). The basin will also require preparation to receive the liner and to decrease the permeability. The entire site is covered by either black sandy/silty clay colluvium (Soil Zone A) or reddish brown/red colluvium (Soil Zone B). The geotechnical evaluation for Zone A and Zone B as extracted from the Geotechnical investigation report is discussed below. The geotechnical zoning of the site is shown in Error! Reference source not found..

3.3.1 Zone A

Stiff black colluvium covers the area and has a thickness of between 0.2 m and 2.4 m with an average thickness of 1.2 m. This layer is considered suitable for the foundations of the starter walls. The stiff soil should have a safe bearing capacity of at least 250 kPa although it should be noted that the clayey soil has a very high potential for expansiveness. It should be noted that care should be taken such that the moisture fluctuation is decreased.

The foundations should be ripped to a depth of approximately 0.3 m and compacted to at least 98% of Proctor maximum dry density to ensure that the soil horizon has a low permeability. The compaction of this material should be conducted with a sheepfoot roller compacter due to the high clay content of the soil to ensure easy workability during compaction. Norite bedrock was found in only seven test pits and consisted of very soft to soft rock norite at a depth of between 1.2 m and 2.2 m below surface. Excavation refusal occurred on very stiff colluvium, very dense pebble marker, very dense residual norite and soft to very soft rock norite, generally at depths of between 1.5 m and 2.8 m. It is possible that refusal occurred on small to medium norite boulders. A safe bearing capacity for the very dense pebble marker and the residual norite should be in the order of 200 kPa to 250 kPa. The soft to very soft rock is considered suitable for foundations of heavy structures.

A remoulded black colluvium resultant in a coefficient of permeability (k-value) of 2,6 x 10^{-9} m/s. The low coefficient of permeability indicated that the ripped and re-compacted basin material can be used in a barrier system.

The measured shear strength parameter of the remoulded black colluvium was found to be 23º with cohesion of 18 kPa. It should be noted that for design purposes the cohesion will be taken as 0 kPa.

The residual Norite that was encountered has a maximum dry density (MDD) (Modified AASHTO) values of between 2,050 kg/m^3 to 2,075kg/m^3 at an optimum moisture content

(OMC) of between 10% to 13% and classifies as G7 quality material. The USCS classification of this material is SC (Clayey Sand) and the measured internal friction angle was 43º with zero cohesion and is within range of typical SC material. For design purposes, an internal friction angle of 30º should be used. This material can be used for construction of the starter wall and other embankments.

3.3.2 Zone B

Reddish brown colluvium covers part of the site and its depth ranges between 0.4 m to 2.8 m (average thickness is 1.2 m). The consistency is generally stiff with isolated firm to stiff areas. The colluvium is estimated to have a safe allowable bearing capacity of approximately 150 kPa, but exhibits a pinhole voided soil structure. The pinhole voided soil structure may cause substantial settlement upon moisture content increases and therefore the following foundation recommendations for the embankment walls are recommended:

- Strip the colluvium to a depth of approximately 0.5 m below surface over the entire area of the base of the embankment. This material can be stockpiled or be used to construct the inner core of the zoned embankment.
- Rip and re-compact the foundation of any excavation to a minimum of 98% Proctor Density
- The base for the inner core of the zoned embankment may be placed on the compacted colluvial layer.

Norite bedrock was only found in TPM34 at a depth of 1.5 m below surface and consisted of very soft rock norite. Excavation refusal occurred on very stiff colluvium, very dense pebble marker, very dense residual norite and very soft rock norite, generally at depths of between 0.8 m and 2.8 m. It is possible that refusal was encountered on small to medium norite boulders. A safe bearing capacity for the very dense pebble marker and the residual norite is estimated to be 200 kPa to 250 kPa. The soft to very soft rock is considered suitable for foundations of heavy structures. The reddish brown/red colluvium may be used as the inner core of the zoned embankment for the TSF.

The measured in-situ coefficient of permeability for the reddish brown / red colluvium varied from 1 x 10^{-10} m/s to 1x10^{-9} m/s. The low coefficient of permeability indicated that the in-situ material in the basin of the proposed TSF and RWD can be used in a barrier system.

The measured shear strength parameter of the remoulded this material ranged from 32º to 33º with cohesion of 4 to 7 kPa. It should be noted that for design purposes the cohesion will be taken as 0 kPa.

The residual Norite from Soil Zone B has a coefficient of permeability k-value of 6,2 x 10^{-8} m/s. This exhibited similar properties to the residual Norite in Zone A. The reworked residual Norite has MDD values of between 1,442 kg/m^3 and 2,015 kg/m^3, with OMC of between 11,9% and 16,8%. The reworked residual Norite from soil Zone B has a coefficient of permeability k-value of 1,8 x 10^{-9} m/s. This material can be used for construction of the starter wall and other embankments.

3.4 Dispersivity assessment

The dispersivity tests conducted on both soil samples, representing the recommended foundations for the tailings storage facility, indicates that the materials are not sensitive with respect to dispersivity once the leachate samples are introduced. The report describing the test method and results is presented in **Appendix F**.

3.5 Conclusion and recommendation

The geotechnical investigation using a test pit method of investigation was conducted for the proposed TSF at Mimosa Farm. A total of fifty six (56) test pits (TPM1 to TP56) were excavated across the site to obtain information on the soil and bedrock conditions and representative disturbed and undisturbed soil samples were retrieved for laboratory testing. The following conclusions were made from the investigation:

- The site is underlain by Pyramid gabbro-norite of the Rustenburg Layered Suite, Bushveld Complex, while Aeolian sand deposits occur to the west of the site. The Rustenburg Fault intersects the site in a north to south direction and a pegmatite/quartz vein occurs in the central west portion of the site. Previous studies showed that many north-south striking dolerite dyke intrusions occur in the region.
- The site is broadly divided into two geotechnical soil zones, viz. Zone A and Zone B. Zone. A contains black sandy/silty clay colluvial deposits overlying norite bedrock, while Zone B contains reddish brown/red sandy clay or clayey sand colluvial deposits also overlying norite bedrock. The thickness of the black colluvium varies between 0.2 m and 2.4 m, while the reddish brown/red colluvium varies between 0.4 m and 2.8 m. Both the zones have an average thickness of 1.2 m.
- A safe bearing capacity for the black colluvium (Zone A) is at least 250 kPa. The foundations should be ripped to 0.3 m below surface compacted to at least 98% of Proctor density by a sheepfoot roller. The stiff clayey soil has a very high potential for expansiveness therefore moisture fluctuation should be monitored or controlled.
- The reddish brown/red colluvium (Zone B) has a safe bearing capacity of at least 150 kPa. This layer has a pinhole voided soil structure and the foundations must

therefore be excavated to 0.5 m depth and ripped an additional 0,3m (0,8m below surface). The base of the excavation must then be compacted to 98% Proctor maximum dry density.

- Norite bedrock occurs at depths of between 1.2 m and 2.2 m at Zone A, while only one test pit showed a depth of 1.5 m in Zone B. The soft to very soft norite bedrock should have a safe allowable bearing capacity of estimated at 500 kPa.
- The black and reddish brown/red colluvium soils may be reused for the inner core of the zoned embankment walls. This material has very low permeability k-values, which is in the order of 1 x 10^{-6} m/s to 1 x 10^{-7} m/s.
- The black colluvium from Zone A has a very low angle of internal friction of 23° with cohesion of 18 kPa, while the reddish brown/red colluvium from Zone B has a higher angle of between 32° and 33°, with cohesion of between 4 kPa and 7 kPa. For design purposes cohesion of 0kPa should be used.
- The residual norite might be used for the outer walls if the zoned embankment. It has permeability k-values, which is in the order of 1 x 10^{-7} m/s to 1 x 10^{-8} m/s. The residual norite has an internal angle of friction of 43° with cohesion of 0 kPa. It is recommended that for design purposes an internal friction angle of 30º should be used.
- The reworked residual norite might be mixed with the residual norite and used as the outer core of the zoned embankment tailings storage facility. This material has a permeability k-value in the order of 1 x 10^{-9} m/s.
- Soft excavation to a depth of between 0.8 m and 2.8 m is expected over the site.
- **No water seepage was encountered in any of the test pits throughout the site.**
- The TSF foundation materials are not sensitive with respect to dispersivity to the leachate.

A detail Geotechnical Investigation report is presented in **Appendix 1** of the Feasibility Design Report prepared in 2014.

4 HYDROGEOLOGICAL STUDY SUMMARY

A geohydrological report was prepared by African Environmental Services and made two key conclusions of importance to the TSF:

1. The ground water is mostly encountered in weathered base aquifer, combined with localised deeper weathered zones associated with vertical rock fracturing.
2. Contamination during operation was found to be over a wide range between 2-200 m per year, due to the uncertainties in the distribution of fractured aquifer system in the surrounding hard rock aquifer. The modelled results showed plume migration of 300 m over 15 years.

Further information is required to better define the groundwater levels. In the study a simplified model was used during the study which assumed that the groundwater mimicked the topography.

The numerical flow model needs to be refined by conducting further investigations in the affected areas of plume migration to determine the hydraulic properties accurately.

Groundwater monitoring is needed to identify the seasonal fluctuations on the site and the various impacts on the groundwater quality.

The data collected previously was considered sufficient for the current phase of the study, however a detailed geohydrological investigation is recommended that should include the following:

1. Electrical resistivity and/or electromagnetic surveys, particularly to identify structural features that could affect the groundwater model
2. Percussion drilling within the assumed affected areas
3. Pump testing to determine hydraulic conductivity and transmissivity properties
4. Water quality testing
5. Updating groundwater and numerical models.

Based on regional geology and hydrogeology information the proposed TSF and RWD are underlain by the mafic rocks of the Bushveld Igneous Complex of the Rustenburg Layered Suit. Hydro geologically, the site is underlain by a low yielding weathered zone (saprolite) aquifer with yield up to 1.5 L/s and an average hydraulic conductivity for this formation is 0.4 m/d. The groundwater at the proposed TSF site is has an electrical conductivity (EC) < 100mS/m.

According to monthly and quarterly monitoring reports, the groundwater quality around the proposed TSF area is clean and odourless. The water level is between 20 to 40 mbgl in the vicinity of the proposed area.

The water level in the boreholes is not yet affected by the current mining activities at Wesizwe since they have remained stable within narrow limits, and fluctuate seasonally.

The findings of the hydrogeological desktop study have facilitated the development of a number of recommendations for further work required with regards to the proposed tailings site. These recommendations would serve to assist in guiding the design of the TSF, as well as provide input where additional information is required from a hydrogeological perspective.

A detailed review of the AES hydrogeological analysis can be found in **Appendix G.**

Subsequently, an additional three monitoring boreholes have been drilled as part of the next phase hydrogeological study. The report “Bakubung TSF Impact Assessment”, (report no DTMP042016) was prepared by DTM. This confirmed the previously determined deep groundwater levels (18 – 22 m deep) and found that no pollution will reach the Elands River during or after operations.

5 SEEPAGE ANALYSIS

Seepage analysis was performed to determine the most suitable drainage design system for the Wesizwe Tailings Storage Facility (TSF) at its final elevation (1,090 mamsl) using steady state finite element analysis in the limit equilibrium software package Rocscience-Slide, Version 6.035.

Cross sections from the highest point in the starter wall and for the toe wall were taken and modelled separately. The tailings dam profile was kept constant for the different drainage design options. The material properties used for analysis are listed below in Table 5-1.

Table 5-1: Material properties

	Unit weight (kN/m^3)	Cohesion (kPa)	Phi (degree)	Permeability (m/s)
Tailings	20	0	33	$1x10^{-7}$
Starter wall	18	0	20	$1x10^{-8}$
Residual Norite	20	0	33	$1x10^{-8}$
Black Colluvium	17	0	23	$1x10^{-8}$
Bedrock	21	50	35	$1x10^{-9}$
Liner	5	1	12	$1x10^{-12}$
Filter Sand	20	0	35	$1x10^{-7}$
Waste Rock	21	0	38	$1x10^{-7}$

The drainage system will reduce the phreatic surface which in turn improves the stability of the outside slope as the water moves away from the outer walls. As stability is based on the resisting forces divided by the mobilising forces, a dryer tailings outer wall increases the resisting force. The reduction in the phreatic surface in turn reduces the pore pressure (mobilising force) and hence improves the stability on the dam. The seepage analysis is used to determine the amount of flow that can be expected in the drainage system. This is taken on board in preparing the design of the drainage systems.

The different drainage design options were considered:

- Case 1 – Curtain/vertical drain and toe drain

This case consists of a toe drain, that is located along the inner toe of the starter wall, and curtain/vertical drains which are located approximately 120 m away from the toe drains and runs along the perimeter of the tailings dam. The concept of the curtain/vertical drain is to intercept and draw down the phreatic surface within the tailings dam. The toe drain will further reduce the phreatic surface.

- Case 2 – Toe drain

This case consists of a toe drain located along the inner toe of the starter wall. This case also gives us an indication of what the behaviour of the phreatic surface will be like if the curtain/vertical drains suggested in case 1 fails.

- Case 3 - Herringbone drainage system and toe drain

This case consists of a toe drain that is located along the inner toe of the starter wall and a herringbone drainage system. The herringbone drainage system consists of a spine/ stem and has ribs/branches that branches out from the spine. The branches lower the phreatic surface and transports the water to the stem, which conveys the water out of the TSF. The herringbone drainage system lowers the hydraulic gradient over the liner to near atmospheric pressure.

This case was further analysed for two different branch-spacings, which were 100 m and 200 m centre to centre, case 3.1 and case 3.2 respectively.

Sensitivity analysis was also done on case 3.1 to observe the behaviour of the phreatic surface and drainage quantity when the toe drain on the herringbone branch fails.

The seepage analysis results are in **Table 5-1**, it consists of the critical discharge at the specific drains.

Table 5-2: Results from the Seepage Analysis

Case	Discharge (l/hr)		
	Vertical drain	Toe drain	Herringbone (critical discharge)
1	433.22	1.89	-
2	-	25.60	-
3.1	-	2.01	2,648.48
3.1 (When the toe drain has failed)	-	-	2,719.37
3.1 (When the herringbone rib has failed)	-	48.96	-
3.2	-	0.14	6,315.12

The discharge volumes from the seepage analysis were used to determine the size of the drainage pipes and the total discharge volume from the TSF to design the solution trench. The total flow allowed in the pipes is calculated using Mannings equation (Equation 1).

$$Q = VA = \left(\frac{1}{n}\right) AR^{\frac{2}{3}}\sqrt{S} \qquad - \qquad \text{Equation 1}$$

Where

Q	= total discharge(m^3/s)
V	= velocity (m/s)
A	= the cross sectional area of the flow (m^2)
n	= roughness coefficient, which is the frictional resistance of the material surface
R	= the hydraulic radius, it is a ratio area to wetted perimeter (A/P) (m)
S	= the bed slope

The fixed parameters used in the assessment are:

- The roughness coefficient, n, was obtained from literature to be 0.01 for polyethylene PE-Corrugated with smooth inner walls.
- The bed slope is estimated to be 0.01;

The pipes being assessed are slotted HDPE Drainex pipes which have a flow channel of 120 degrees. For the DN 160 pipe the inside diameter is 137 mm, which allows for a flow of 2.13×10^{-3} m^3/s at full capacity at a specific point. The critical flow from the herringbone drainage system, case 3.1, is 8.22×10^{-7} m^3/s at a specific point. The DN 160 pipe can hold more flow than what is anticipated from the seepage thus this indicates that the DN 160 pipe will be adequate for the herringbone ribs. The herringbone spine needs to account for all the flow that it will be receiving from the ribs, and it has been observed that the stem should consist of at least three DN160 slotted HDPE pipes so that the accumulated flow from the ribs can be accommodated for.

6 SLOPE STABILITY ANALYSIS

Stability analysis was carried out to determine the slope stability of the tailings with the starter wall starting at 1,058 mamsl and the dam's crest set at its future final height of 1,090 mamsl.

Cross sections were obtained from AutoCad drawings and plotted into Rocscience-Slide, Version 6.035. The cases used in Section 5 were also reviewed and analysed for slope stability and were the basis for selecting the best drainage system for this TSF.

This best drainage case was then used to model various starter wall options so that costs and footprint size is optimized. Each model was assessed for local failure and deep failure. The material properties used for the models are presented in Table 5-1.

The Factor of Safety (FoS) obtained from the models is then compared to the accepted minimum FoS of 1.3 under static loading for an operating TSF that is monitored and if any minor damages happen they can be remedied immediately. The acceptable minimum FoS under long term static loading and the TSF is no longer in operation is 1.5.

The FoS for the cases mentioned in Section 5 can be found in

Table 6-1. The results showed that the deep failures and the toe wall have FoS that are higher than 1.5, thus analysis for the toe wall and deep failures on the starter wall were not further analysed for the other options. Whereas the FoS from the local failures in the starter wall were not above the required 1.5, so using drainage case 2, which gave the highest FoS of all the drainage options, alternative starter wall designs were modelled and analysed to reach an acceptable FoS. The results are presented in

Table 6-1, cases A to H.

Table 6-1 also presents the description of the revised cases.

From the results it has been observed that the flatter the slope of the starter wall the higher the FoS, it was also observed that the use of only residual norite in the starter wall is not adequate enough to ensure stability, as in case B the slope have been flattened to 1:4 and the FoS is still below the required 1.5. Thus cases C – H were analysed where portions of the starter wall consists of waste rock. The starter wall and material interface slope has been optimised to ensure the optimal use of materials and footprint.

Case H, is the final design option as it requires the lowest volume of construction material and covers the smallest footprint while maintaining a FoS exceeding the norm during the operational phase, thus making it the most appropriate option.

Table 6-1: Results of the Stability Analysis for the Revised Cases

Case	Description	FOS
1	• 1 in 2 starter wall slope – residual norite • Toe and curtain drain operational	0.847
2	• 1 in 2 starter wall slope – residual norite • Toe drain operational	0.759
3.1	• 1 in 2 starter wall slope – residual norite • Herringbone drainage system (200 m centre to centre)	0.863
3.1 (toe drain has failed)	• 1 in 2 starter wall slope – residual norite • Herringbone drainage system (200 m centre to centre)	0.856
3.1 (herringbone rib has failed)	• 1 in 2 starter wall slope – residual norite • Herringbone drainage system (200 m centre to centre)	0.825
3.2	• 1 in 2 starter wall slope – residual norite • Herringbone drainage system (100 m centre to centre)	0.81
A	• 1 in 3 starter wall slope – residual norite • Herringbone drainage system (200 m centre to centre)	1.118
B	• 1 in 4 starter wall slope – residual norite • Herringbone drainage system (200 m centre to centre)	1.318
C	• 1 in 3 starter wall slope • 1 in 1 slope division from the centre of the crest towards the downstream direction • Waste rock is placed on the downstream section • Residual norite is placed on the upstream section • Herringbone drainage system (200 m centre to centre)	1.661
D	• 1 in 2 starter wall slope • 1 in 1 slope division from the centre of the crest in the downstream direction • Waste rock is placed on the downstream section • Residual norite is placed on the upstream section • Herringbone drainage system (200 m centre to centre)	1.123
E	• 1 in 3 starter wall slope • A step out berm that is 6m high and 10m width, made of waste rock. • Residual norite is placed on the upstream section • Herringbone drainage system (200 m centre to centre)	1.435
F	• 1 in 2 starter wall slope • A straight vertical division from the centre of the crest • Waste rock is placed on the downstream section • Residual norite is placed on the upstream section • Herringbone drainage system (200 m centre to centre)	1.609
G	• 1 in 1.5 starter wall slope • A straight vertical division from the centre of the crest • Waste rock is placed on the downstream section • Residual norite is placed on the upstream section • Herringbone drainage system (200 m centre to centre)	1.352
H	• 1 in 1.75 starter wall slope • A straight vertical division from the centre of the crest • Waste rock is placed on the downstream section • Residual norite is placed on the upstream section • Herringbone drainage system (200 m centre to centre)	1.503

The seepage and stability model for case H is presented in Figure 6-1 below.

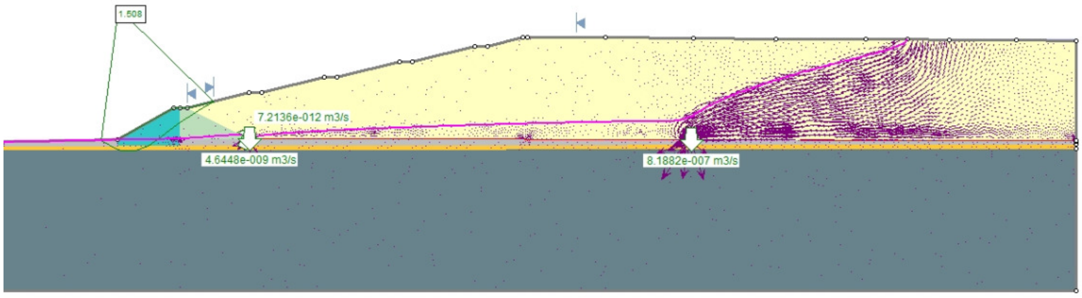

Figure 6-1 Seepage and slope stability analysis

7 DESIGN AND CONSTRUCTION CONSIDERATION

7.1 Description of TSF

The TSF will be developed by constructing a compacted starter wall with borrowed material. The tailings will then be deposited by spigot method of deposition with the spigot pipes placed at the crest of the starter wall connected to a large diameter perforated pipe extending to beyond the toe filter drain, in order to prevent erosion of the starter wall or blinding of the drains. This method of tailings dam construction and development has been used in South Africa in the Platinum mining industry and has a proven track record of its success.

The toe drain installed on the inside toe of the starter wall will draw down the phreatic surface that will develop during the development of the TSF. Furthermore, a herringbone drain system will be installed in the TSF basin with the purpose of reducing the hydraulic gradient on the barrier system and aiding in consolidation of the deposited tailings material. All drains are designed to drain to a concrete lined solution trench which conveys the solution to the RWD.

Paddocks will be constructed at the outside perimeter of the starter wall and their purpose is to collect all the run-off water from the surface of the TSF slopes or any potential tailings spillage and allow the run-off water to evaporate. The details of these components are discussed below.

7.2 BARRIER SYSTEM TSF

The in situ material (clay) in the basin on site will be borrowed to construct starter wall and other small embankments. According to the latest hydrogeological report, the ground water level can be found at depth between 18 m to 22 m below ground level and therefore the ground water drains have been omitted from the design. The geotechnical investigation revealed that the in-situ material has low permeability, therefore is it proposed that the basin be ripped to a depth 300 mm and re-compacted to a 98% Proctor Density at ±2% optimum moisture content (OMC). This layer will be the subgrade to the geomembrane to be installed. A 1.5 mm HDPE (double textured) geomembrane is proposed to serve as the liner.

The tailings material is fine grained and delivered to the facility as wet slurry. For this reason it is considered that the tailings material will form a protection layer over the geomembrane over time. It should be noted the maximum particle size of the tailings is less than 3 mm prescribed in SANS 10409. The prescribed protection layer is therefore omitted from the design.

The proposed composite liner details are shown in **Appendix B** attached to this report.

7.3 BARRIER SYSTEM RWD

The in situ material (clay) in the basin on site will be borrowed to construct the containment wall of the RWD. According to the hydrogeological report, the ground water level can be found at depth between 18 m to 22 m below ground level and therefore the ground water drains have been omitted from the design. The geotechnical investigation revealed that the in-situ material has low permeability, therefore is it proposed that the basin be ripped to a depth of 300 mm and re-compacted to a 98% Proctor Density at ±2% optimum moisture content (OMC). This layer will be the subgrade to the geomembrane to be installed. A 2.0 mm HDPE (double textured) geomembrane is proposed to serve as the liner. A geotextile is proposed over the geomemberane to provide protection from the concrete.

A 150 mm thick geocells filled concrete is proposed to be installed over the barrier system to protect the geomembrane from UV and from puncture during any maintenance or cleaning of the RWD.

The section and details of the barrier system can be seen in the drawings presented in **Appendix B**

7.4 STARTER WALL AND TSF DEVELOPMENT

A starter wall will be constructed from material borrowed from the basin of the TSF at 1V:1.75H downstream and upstream slopes with a crest width of 6 m wide. The starter wall will be constructed in two phases, with the first phase raising the wall to elevation 1,055 mamsl (11 m high wall) and phase 2 raising it to elevation 1,058 mamsl. This will result in the final height of the starter wall of 14 m above the natural ground level.

The TSF will be developed / constructed using the upstream method with an overall outer slope of 1V:5H. This will be achieved by constructing inter benches at every 7 m outer wall height increase with the inter slopes at 1V:4H. The final elevation of the TSF will be 1,090 mamsl. The final height of the TSF will be 46 m above the lowest natural ground level.

Outer toe paddocks will be constructed outside the impounding embankment using material borrowed from the basin. The purpose of the paddocks is to collect runoff from the outer slopes of the facility and any potential tailings spillage. The paddocks are designed to contain the 1:100 year storm runoff and will be provided with emergency overflow spillways.

The sections and details of the starter wall are presented on the drawings in **Appendix B**

7.5 UNDERDRAINAGE DRAINAGE SYSTEM

The underdrainage system has been designed to control the phreatic surface, to assist with consolidation of the tailings material and to minimise the hydraulic gradient over the liner. The system consists of the following:

- The toe drain, which runs along the inside toe of the starter wall; and
- A network of finger drains located on top of the geomembrane to reduce the hydraulic gradient over the barrier system.

These drains have been designed as filter drains consisting of a series of filter sand, 6 mm and 19 mm stone aggregate. The intercepted seepage is collected using slotted corrugated HDPE pipes.

The layout and details of the drains are presented on the drawings in **Appendix B.**

7.6 DECANT SYSTEMS

An intermediate decant intakes which consist of a multi–stage stacked concrete rings will be constructed towards the outer wall for decanting supernatant during the early development of the facility. A permanent penstock decant consisting of a 20 m high concrete tower and a multi-stage stacked concrete rings used to final elevation of

1,090 mamsl will be constructed towards the centre of the TSF. When supernatant water reaches the permanent penstock decant the intermediate decant system will be sealed off. The sealing should be designed by a professional registered engineer.

7.7 SURFACE WATER MANAGMENT

The surface water management plan is designed to separate clean and dirty water run-off in compliance with Regulation GN704 and best industry practise. Surface runoff from the outer slopes of the TSF will be collected in toe paddocks and allowed to evaporate. Any rainfall falling within the TSF footprint will be conveyed to the silt trap and RWD. The operating philosophy is that the dirty water will be recycled from the RWD back to the plant.

A clean water diversion canal will be constructed on the outside perimeter of the complex to divert any clean run-off around the facility to prevent possible contamination.

7.8 RETURN WATER DAM

A RWD will be provided to store the decanted water and allow for pumping back to the plant. This RWD is located to the South of the TSF toward the river side. The RWD is designed to have a capacity of 433,000 m^3. The maximum operating level will be defined in the operating manual to store 23,000 m^3 allowing for 3 days of plant demand. The remaining capacity are to be used for storage of excess stormwater. Allowance is made for the run-off generated during a 1: 50 yr 24 hr storm event, with volume 410,000 m^3.

The operating level of the RWD dam will be described in the operating manual which will be submitted separately from this report. An emergency spillway will be provided to allow for the safe passing of flood events in excess of the 50 year recurrence interval storm. This flow will discharge to the natural environment.

The outer wall slopes of the RWD are design to have a 1V:3H inside slope and 1V:2H downstream slopes with crest width of 4 m.

To mitigate the risk of drowning in the lined RWD, nylon ropes (or equivalent) fastened to anchor blocks at strategic positions around the dam will be provided. Furthermore, the RWD is fenced off to prevent any unauthorised access and to prevent livestock from drinking the water in the dam.

7.9 OTHER INFRASTRUCTURE

A concrete lined solution trench will be constructed around the perimeter of the TSF to collect filter drain water, water from the paddock system if there is an overflow, and to convey the water from the decant system to the silt trap and then to the RWD.

8 CONSTRUCTION QUALITY ASSURANCE PLAN

Knight Piésold has developed a comprehensive Construction Quality Assurance (CQA) plan (see **Appendix D**) to ensure that proper construction techniques and procedures are used and that the project is built in accordance with the project Construction Drawings and Specifications. This plan will help with identifying and defining problems that may occur during installation of the barrier system, construction and to observe that these problems are corrected before construction is complete. The CQA details procedures for monitoring works and evaluation of materials and workmanship during construction; and also assigns responsibilities to various parties that will be involved in this project.

9 MONITORING OPERATION

The detailed monitoring procedure of the facility is in the operations and maintenance manual which is submitted separately from this report. Typically, the operations and maintenance manual will outline the following in details:

- Safety during the construction / development of the facility,
- Construction / development of the facility,
- Management of the facility and responsibilities,
- Monitoring procedures to be followed, and
- Maintenance procedures.

Quarterly inspections of the facility by a suitably qualified person and the compilation of an annual report on the construction and operation of the facility is a regulatory requirement of the Department of Mineral Resources. The focus of the quarterly inspections and annual reporting on the facility will be to ensure that:

- The facility is being constructed and operated in accordance with the design requirements;
- Seepage and slope stability models of the facility are periodically reviewed and updated as necessary;
- Safe working practices by the TSF operator are adhered to;
- Ongoing rehabilitation of the facility is kept up to date and that routine maintenance activities are carried out by the operator;

- Monitoring information relevant to the TSF is collected and analysed. Information is expected to be collected as part of the overall environmental management function for the mine which should include collection and analysis of groundwater samples from up and down gradient of the TSF, sampling and analysis of surface water samples from the return water dam and collection and analysis of dust samples from up and down wind of the facility.
- The Code of Practice for the TSF developed and issued to the relevant departments.
- The Operating Manual for the TSF developed and adhered to.

To further enhance the monitoring programme, monthly inspections by a suitably qualified person together with Mine personnel and the TSF operator are also recommended.

A typical operations and maintenance manual for the HDPE is included as **Appendix E**.

10 REHABILITATION, CLOSURE AND AFTERCARE

Rehabilitation and closure works that will take place concurrently with the construction of the facility will include stripping and stockpiling of topsoil from the site for use in the rehabilitation and closure process. Being an upstream constructed facility, concurrent rehabilitation work will be able to be carried out on the outer slopes of the facility. The overall outer slope will be 1V:5H with intermediate slopes between benches at 1V:4H.

It is currently envisaged that topsoil placement and planting of a mix of indigenous grasses will form the basis of rehabilitation.

It is envisaged that monitoring of surface and groundwater quality in the area will be required to continue for a period of up to 30 years after closure.

11 CERTIFICATION

This report was prepared, reviewed and approved by the undersigned:

Prepared by: ____

T Mokoma PrEng
Civil Engineer

Reviewed and Approved by:__________________________________

Andries Strauss PrEng
Manager: Mine Residue Section

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12 REFERENCES

1) Knight Piésold Consulting Feasibility Study Report No. 301-00509/01/RI, March 2014
2) Knight Piésold Consulting Geotechnical Investigation Report No. KHH2076/3010007912 Rev.0, October 2013
3) Knight Piésold Consulting Review of Wesizwe Platinum Geohydrological Evaluation, Memorandum dated 26 October 2015
4) DTM Projects and Services Bakubung TSF Impact Assessment Report No. DTMP042016, February 2016

APPENDIX A
WASTE CLASSIFICATION

MEMO

TO: Andries Strauss

FROM: Dr Jon McStay/ Enéz Nickall

SUBJECT: Wesizwe tailings geochemistry characterisation

DATE: 24 November 2015

WSP Environmental (Pty) Ltd is pleased to provide its findings on the Wesizwe tailings geochemical characterisation.

Samples

Knight Piésold (Pty) Ltd provided two samples representative of tailings arising and tailings supernatant for UG2 and Merensky tailings, for analysis. The samples were labelled "Supernatant" and "UG2". It is understood that the samples were representative of the residues of pilot testing of the proposed mineral processing for Wesizwe. The supernatant samples was submitted to ALcontrol Laboratories in the United Kingdom on 10 September 2015, the UG2 tailings sample was submitted to Jones Environmental Laboratory on 6 November 2015.

Analysis

The following analyses were requested:

- Mineralogical analysis by X-ray diffraction,
- Acid Base Account to confirm the acid generating potential of the tailings samples
- Acid digest followed by ICP MS for total metals,
- Leach testing (Australian Standard Leaching Procedure AS 4439) using deionised water with the extracts to be analysed for :
  - pH, EC, Ca, Mg, Na, K;
  - Alkalinity, SO4, Cl, F; and
  - Metals and Metalloids by ICP-MS

Assessment

- **Mineralogical characterisation**

The tailings material has analysed by XRA Analytical and Consulting using a Panalytical Empyrean diffractometer with PIXcel detector and fixed slits with Fe filtered Co-Kα radiation. Mineral phases were identified using X'Pert Highscore software.

The tailings are considered typical of Bushveld norite and pyroxenite with a high chromite content.

Table 1: Mineralogical composition of Wesizwe tailings

MINERAL GROUP	ESTIMATED %
Enstatite	26.5%

MINERAL GROUP	ESTIMATED %
Plagioclase	18.2%
Chromite	12.9%
Talc	9.3%
Diopside	9.0%
Hornblende	8.5%
Biotite	8.2%
Chlorite	4.3%
Calcite	2.3%
Quartz	0.9%

→ Note on the Application of Waste Risk Profile GN635

Indicative waste profiling considering the total and leachable concentrations of inorganic contaminants has been undertaken based on the National Norms and Standards for the Assessment of Waste for Landfill Disposal (GN 636 of 2013).

The level of risk associated with the disposal of each type of waste to landfill based on the classification system using total concentrations of contaminants or leachable concentration of contaminants in the waste is as follows.

Table 2: Waste Classification according to GN635

CONTAMINANT CONCENTRATION CRITERIA	RISK LEVEL	DESCRIPTION OF RISK LEVEL ASSOCIATED WITH THE DISPOSAL OF WASTE TO LANDFILL
LC > LCT2, or TC > TCT2	**Type 0:** **Very High Risk**	Considered very high risk waste with a very high potential for contaminant release. Requires very high level of control and on-going management to protect health and the environment.
LCT1 < LC ≤ LCT2, or TCT1 < TC ≤ TCT2	**Type 1:** **High Risk**	Considered high risk waste with high potential for contaminant release. Requires high level of control and on-going management to protect health and the environment.

LCT0 < LC ≤ LCT1 and TC ≤ TCT1	**Type 2: Moderate Risk**	Considered moderate risk waste with some potential for contaminant release. Requires proper control and ongoing management to protect health and the environment.
TC < 20 x LCT0, or LC ≤ LCT0 and TC ≤ TCT0	**Type 3: Low Risk**	Low risk waste with low potential for contaminant release. Requires some level of control and on-going management to protect health and the environment.
TC<20x LCTi, or LC ≤ LCTi and TC ≤ TCTi	**Type 4: Inert Waste**	Very low risk waste that- Does not undergo any significant physical, chemical or biological transformation Does not burn, react physically or chemically or otherwise affect any other matter with which it may come into contact, and Does not impact negatively on the environment because of its low pollutant content and because the toxicity of its leachate is insignificant. Only basic control and management required.
TC = Total Concentration LC = Leachable Concentration		

The point of departure from the '*Minimum Requirements'* classification of hazardous wastes is that the classification is no longer influenced by the concept of acceptable environmental loading i.e. mass of contaminant divided by area available for disposal. In addition the risk paradigm used to drive the acceptable risk levels is no longer based solely on the protection of aquatic ecosystem health based on a simple direct pathway equation from contaminant body to surface water source. The effect of the new system is to remove over-conservative limits and is particularly important in the case of some heavy metals where the selection of toxicological risk criteria used in the '*Minimum Requirements'* was based on a sensitive ecological receptor, i.e. rainbow trout LC_{50} values. Although the scientific validity of this data is not in question its use as a criterion for bulk waste management represents an extreme point of compliance unlikely to be experienced in a real situation particularly in South Africa.

It is proposed that GN635 is used to classify materials in residue deposits although these materials would not normally be expected to be landfilled and the disposal regulations of GN635 would therefore not be directly applicable.

The chemical analyses presented below include total concentrations and leachable concentrations in reagent water determined by Jones Environmental Laboratory.

Table 3: GN635 Waste Classification for Wesizwe Tailings based on Total Concentrations

PARAMETER	BULK COMPOSITE TAILINGS TOTAL CONCENTRATION (MG/KG)	TCT0	TCT1	TCT2	WASTE CLASSIFICATION GN635
Antimony	4	10	75	300	Type 4
Arsenic	<0.5	5.8	500	2 000	Type 4
Barium	77	62.5	6 250	2 5000	Type 3
Boron	<0.5	150	1 500	60 000	Type 4
Cadmium	<0.1	7.5	260	1 040	Type 4

PARAMETER	BULK COMPOSITE TAILINGS TOTAL CONCENTRATION (MG/KG)	TCT0	TCT1	TCT2	WASTE CLASSIFICATION GN635
Chromium (iii)	604.4	460 000	800 000	Na	Type 4
Cobalt	15.8	50	500	2 000	Type 4
Copper	17	16	19 500	78 000	Type 3
Lead	<5	20	1 900	7 600	Type 4
Manganese	316	1000	25 000	100 000	Type 4
Mercury	<0.1	0.93	160	640	Type 4
Molybdenum	0.8	40	1000	4000	Type 4
Nickel	137.5	91	10 600	42 400	Type 3
Selenium	<1	10	50	200	Type 4
Vanadium	37	150	2 680	10 720	Type 4
Zinc	16	240	160 000	640 000	Type 4
Sulphate	55				Type 4

Table 4: GN635 Waste Classification for Wesizwe Tailings based on leachable concentrations (reagent water)

PARAMETER	BULK COMPOSITION TAILINGS LEACHABLE CONCENTRATION (MG/L)	LCT0	LCT1	LCT2	LCT3	WASTE CLASSIFICATION GN635
Antimony	<0.002	0.02	0.5	1	4	Type 4
Arsenic	<0.0025	0.01	0.5	1	4	Type 4
Barium	<0.0005	0.7	35	70	280	Type 4
Boron	<0.012	0.5	25	50	200	Type 4
Cadmium	<0.0005	0.003	0.15	0.3	1.2	Type 4
Chromium (iii)	0.0151	0.1	5	10	40	Type 4
Cobalt	<0.002	0.5	25	50	200	Type 4
Copper	<0.007	0.006	0.3	0.6	2.4	Type 4
Lead	<0.005	0.01	0.5	1	4	Type 4
Manganese	<0.002	0.5	25	50	200	Type 4
Mercury	<0.001	0.006	0.3	0.6	2.4	Type 4
Molybdenum	<0.002	0.07	3.5	7	28	Type 4
Nickel	<0.002	0.07	3.5	7	28	Type 4
Selenium	<0.003	0.01	0.05	1	4	Type 4
Vanadium	0.0055	0.2	10	20	80	Type 4
Zinc	0.004	5	250	500	2 000	Type 4
Sulphate	35	250	12 500	25 000	10 000	Type 4
Fluoride	<0.3	1.5	75	150	600	Type 4
Chloride	1.4	300	15 000	30 000	120 000	Type 4

The combined data set indicates that the total concentrations of barium, copper and nickel classify the tailings as a Type 3 hazardous waste, although both copper and barium are only marginal exceedances. Leachable concentrations are well above the level for chemically inert materials. On the basis of the total concentrations and leaching test data a waste classification of Type 3 would be considered to be most representative of the in-situ bulk residue chemistry.

Table 5: GN635 Waste Classification for Wesizwe supernant water based on dissolved concentrations

PARAMETER	DISSOLVED CONCENTRATION mg/l	LCT0	LCT1	LCT2	LCT3	WASTE CLASSIFICATION GN635

PARAMETER	DISSOLVED CONCENTRATION mg/l	LCT0	LCT1	LCT2	LCT3	WASTE CLASSIFICATION GN635
Alkalinity bicarbonate as $CaCO_3$	90					
Conductivity (mS/cm)	0.422					
Antimony	0.00246	0.02	0.5	1	4	Type 4
Arsenic	0.000659	0.01	0.5	1	4	Type 4
Barium	0.00402	0.7	35	70	280	Type 4
Boron	0.0102	0.5	25	50	200	Type 4
Cadmium	<0.0001	0.003	0.15	0.3	1.2	Type 4
Chromium (iii)	0.00197	0.1	5	10	40	Type 4
Chromium (vi)	<0.03	0.05	2.5	5	20	Type 4
Cobalt	0.000065	0.5	25	50	200	Type 4
Copper	0.00137	0.006	0.3	0.6	2.4	Type 4
Lead	0.000023	0.01	0.5	1	4	Type 4
Manganese	0.00311	0.5	25	50	200	Type 4
Mercury	<0.00001	0.006	0.3	0.6	2.4	Type 4
Molybdenum	0.00553	0.07	3.5	7	28	Type 4
Nickel	0.00505	0.07	3.5	7	28	Type 4
Selenium	0.00097	0.01	0.05	1	4	Type 4
Vanadium	0.00401	0.2	10	20	80	Type 4
Zinc	0.00173	5	250	500	2 000	Type 4
Sulphate	2380	250	12 500	25 000	10 000	Type 3
Fluoride	<0.5	1.5	75	150	600	Type 4
Chloride	30.3	300	15 000	30 000	120 000	Type 4

The supernatant water is marked by having moderately high sulphate content but very low concentrations of heavy metals.

In terms of developing the risk assessment for the rehabilitation of the residue deposit the total concentrations are important in the consideration of direct exposure pathways to receptors. The most likely exposure pathway is dust inhalation by workers and the Type 3 classification related to the total concentration of nickel is considered to be the most sensitive human health related risk. The easiest way to remove the human health risk to works is to cap the residue deposit to isolate the dust migration pathway. The leachable concentrations give an indication of the risk to water resources and thus the Type 4 waste class is consistent with a very low risk to water resources in the absence of an immediate pathway to surface water bodies or drinking water supplies. The development of elevated sulphate concentrations in the groundwater closest to the tailings facility is the mostly likely indication of any form of groundwater plume developing. The risk profile of the residue deposit would indicate that a simple vegetative capping with or without an underlying low permeability clay cap would provide a suitable environmental barrier layer to isolate the residue deposit from the environment.

→ Acid Base Accounting

The Acid Base Accounting (ABA) method is a simple chemical screening test to determine the potential for acid generation and the neutralising potential of the material. The criteria applied for the assessment are briefly summarised below:

- Potential Acid Forming (Type 1) – Total S (%)>0.25%; AP:NP ration 1:1 or less; NNP more negative than minus 20 $CaCO_3$ kg/t
- Intermediate (Type III) – Total S (%) > 0.25% and AP:NP ratio 1:3 or less: NNP between - 20 $CaCO_3$ kg/t and 20 $CaCO_3$ kg/t
- Non-acid forming (Type III) Total S (%) <0.25% and AP:NP ratio 1:3 or greater; NNP greater than plus 20 $CaCO_3$ kg/t

The Modified Neutralisation Potential was determined to be 31 $CaCO_3$ kg/t, total S is 0.02%. The soil p H is 8.35 and no sulphide phases were recorded in the XRD analysis. Therefore, the tailings material is considered to be non-acid forming.

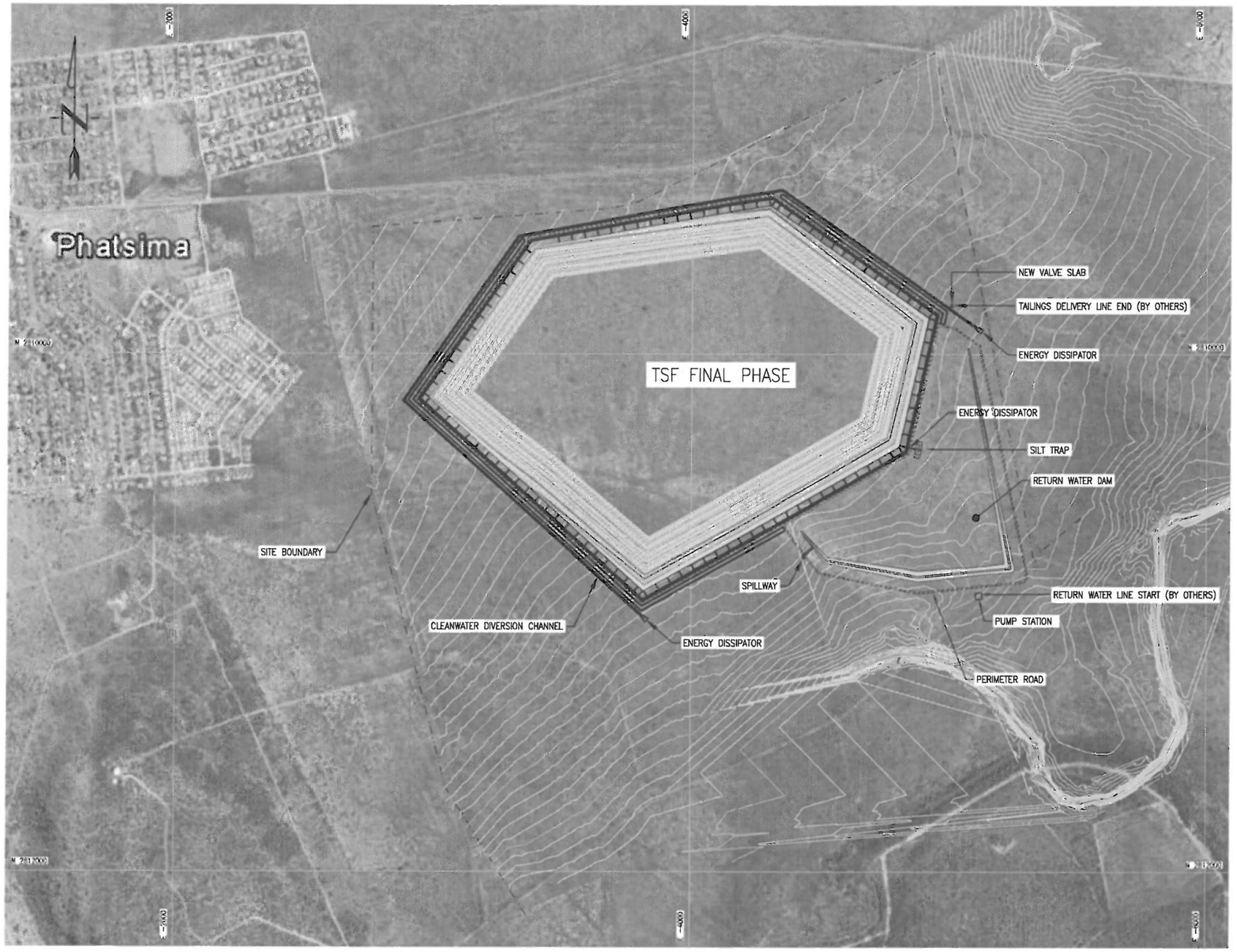
Given the results of the previous testing and the performance of similar findings from tailings, waste rock and slags derived from Bushveld Complex rocks no further testing of acid generation is deemed necessary.

APPENDIX B
SIGNED DESIGN DRAWINGS

WESIZWE PLATINUM

MIMOSA TSF DESIGN

PRO. NO. 301-00509/02



LIST OF DRAWINGS		
DRAWING NO.	DRAWING DESCRIPTION	REVISION
301-00509/02-000	LOCALITY PLAN AND LIST OF DRAWINGS	B
301-00509/02-001	GENERAL ARRANGEMENT	A
301-00509/02-002	PHASE 1 – PROJECTED CONSTRUCTION	A
301-00509/02-003	PHASE 1 – EARTHWORKS AND FILTER DRAIN DETAILS AND SECTIONS	A
301-00509/02-004	PHASE 1 – LONGITUDINAL SECTIONS	A
301-00509/02-005	TRAINING WALL LAYOUT AND SECTIONS	A
301-00509/02-006	PHASE 1 – SLURRY DISTRIBUTION PIPELINE AND DETAILS	A
301-00509/02-007	PERMANENT DECANT DETAILS AND SECTIONS	A
301-00509/02-008	SILT TRAP DETAILS AND SECTIONS	A
301-00509/02-009	PHASE 2 – PROJECTED CONSTRUCTION LAYOUT AND SECTIONS	A
301-00509/02-010	PHASE 3 – PROJECTED CONSTRUCTION LAYOUT AND SECTIONS	A
301-00509/02-011	PHASE 3 – SLURRY DISTRIBUTION PIPELINE AND DETAILS	A
301-00509/02-012	INTERMEDIATE DECANT – CATWALK PLATFORM AND WALKWAY DETAILS	A
301-00509/02-013	PERMANENT DECANT – CATWALK PLATFORM AND WALKWAY DETAILS	A
301-00509/02-014	INSTRUMENTATION LAYOUT AND DETAILS	A
301-00509/02-015	LONGITUDINAL SECTIONS	A

250 0 250 500 750
METRES 1:10000

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CONSULTING

20090268

PRIMARY DISCIPLINE	CIVIL	
DRAWN	ME	28/09/2015
DRAWING CHECK	TM & JZ	28/09/2015
DESIGN	RG	28/09/2015
DESIGN CHECK	AS	28/09/2015
PROJECT ENGINEER	AS	–
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REVISION					
REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
LOCALITY MAP AND
LIST OF DRAWINGS

DRAWING NUMBER	SCALE	REV.
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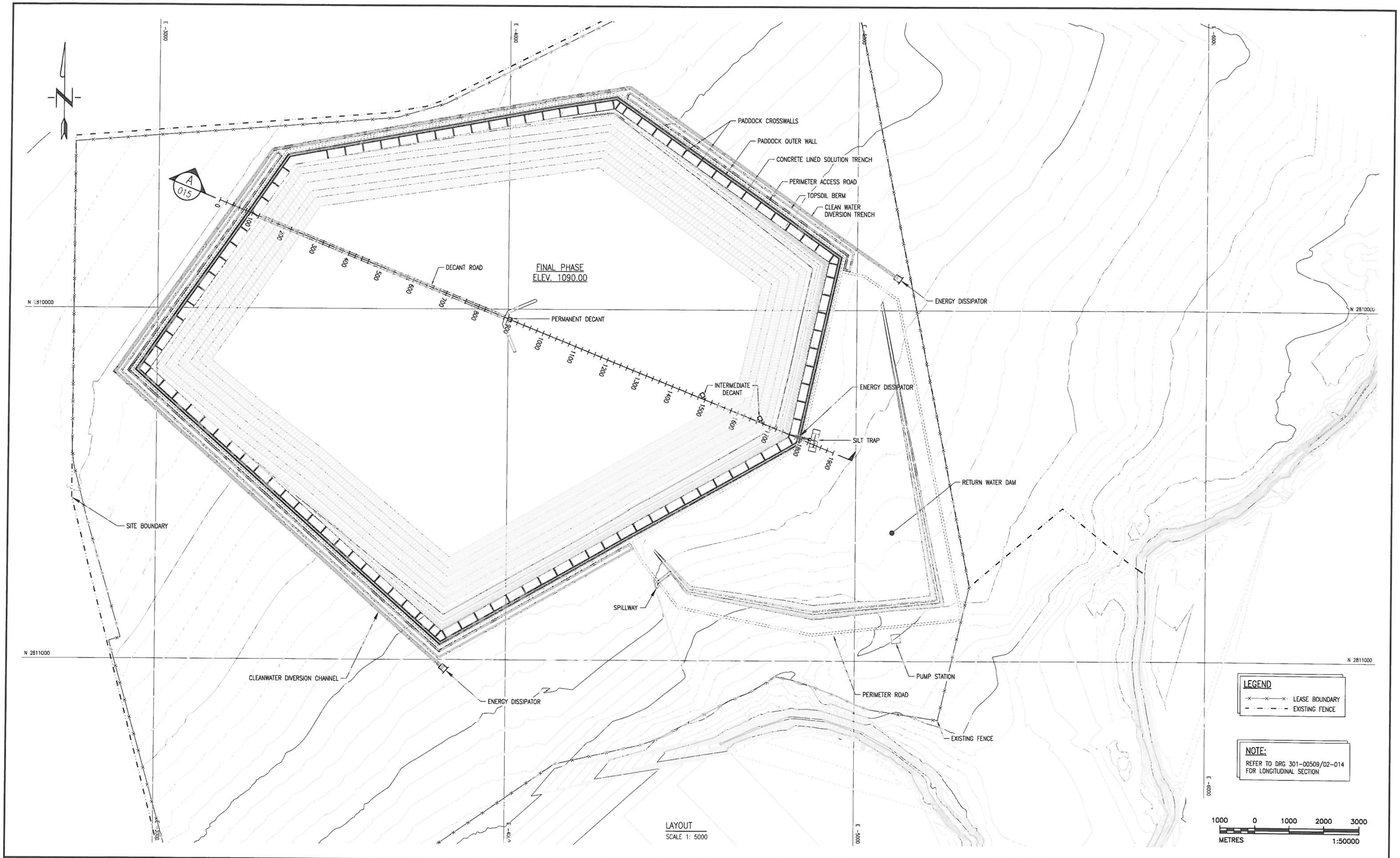

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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
GENERAL
ARRANGEMENT

DRAWING NUMBER	SCALE	REV.
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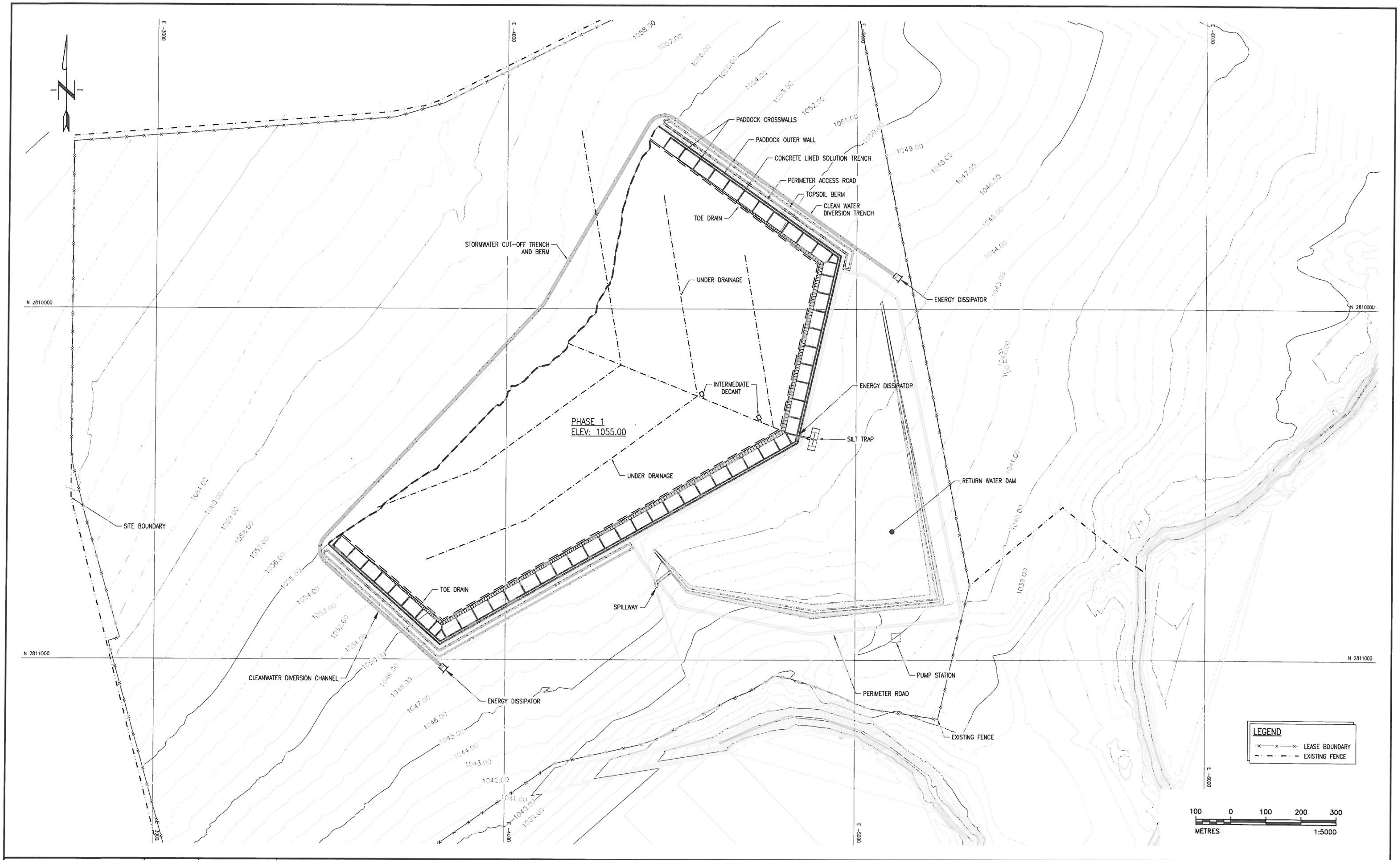

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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
PROJECTED CONSTRUCTION
PHASE 1
LAYOUT

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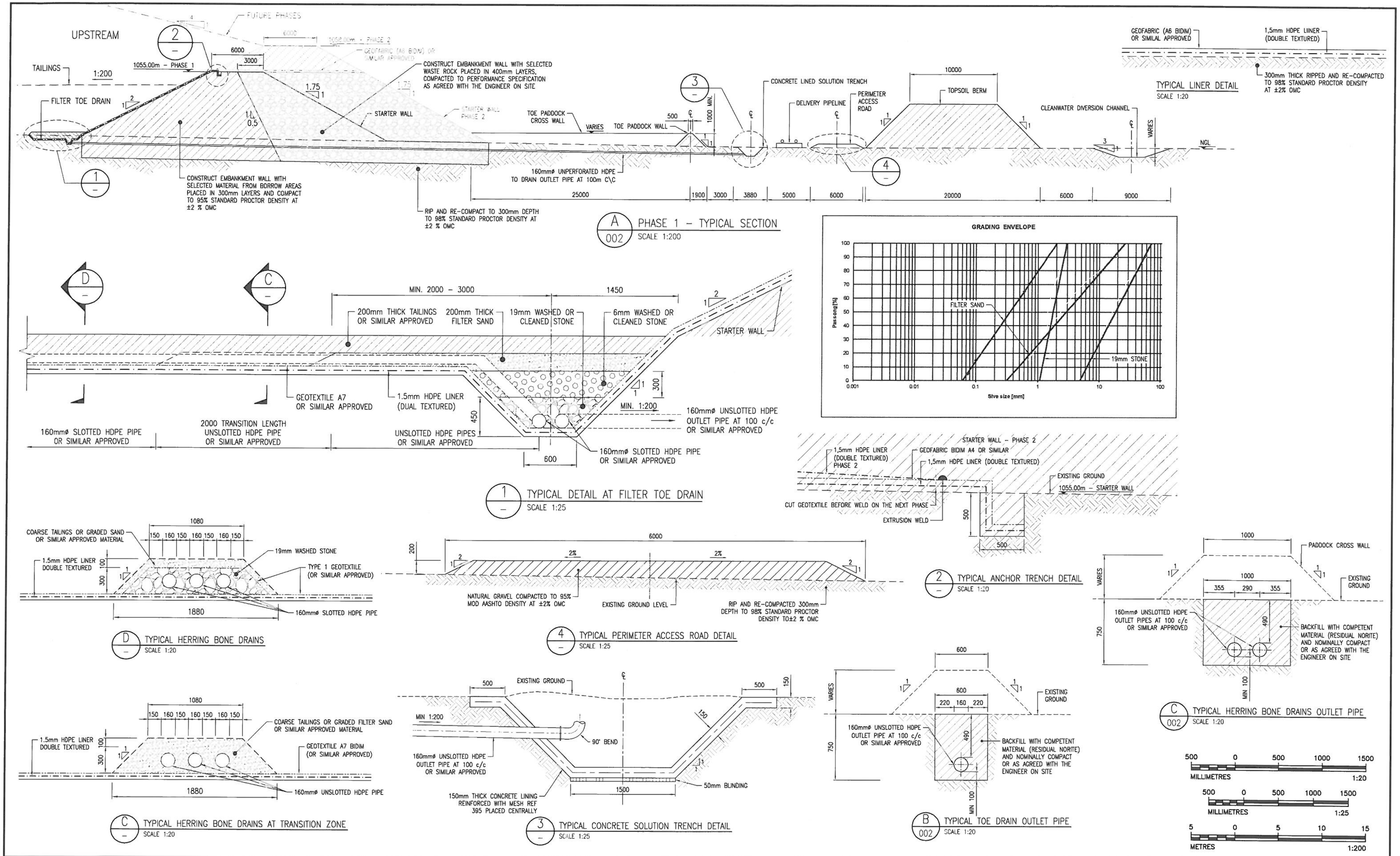

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DRAWING CHECK	TM & JZ	28/08/2015	
DESIGN	RG	28/08/2015	
DESIGN CHECK	AS	28/08/2015	
PROJECT ENGINEER	AS	-	

REVISION

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REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE
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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
PHASE 1
EARTHWORKS AND FILTER DRAIN DETAILS AND SECTIONS

DRAWING NUMBER	SCALE	REV.
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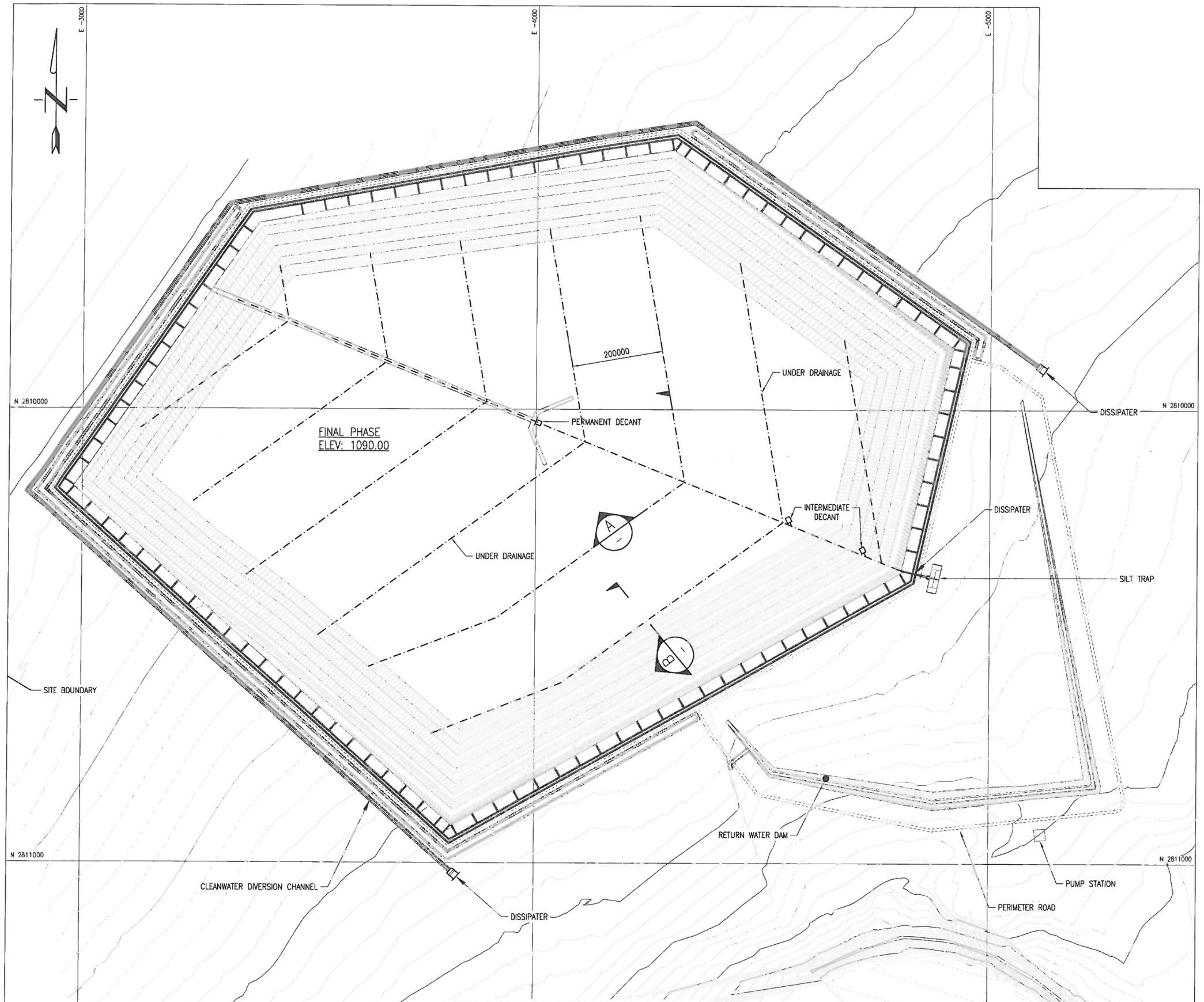
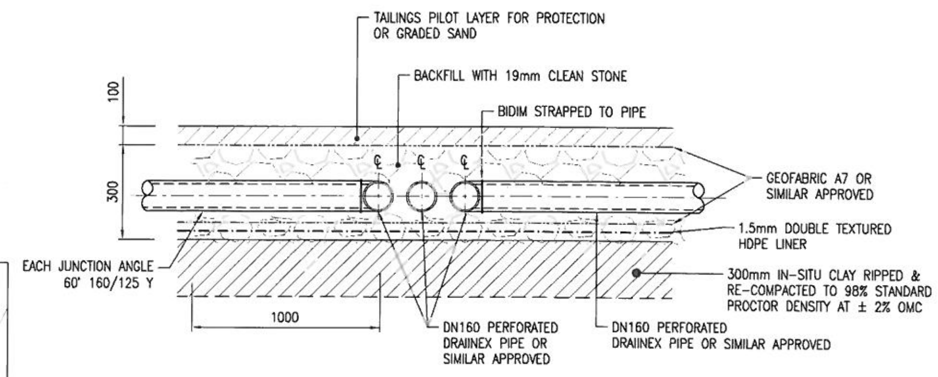


UNDER DRAINAGE DISTRIBUTION PIEWORKS LAYOUT
SCALE 1: 5000



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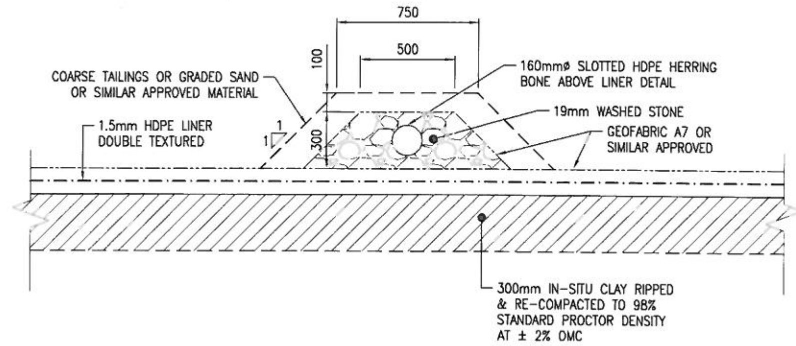

B TYPICAL LINER DETAIL WITH DRAINS
SCALE 1:20

NOTE:
HERRING BONE DRAIN TIE INTO TOE DRAIN

500 0 500 1000 1500
MILLIMETRES 1:20

100 0 100 200 300
METRES 1:5000

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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
UNDER DRAINAGE LAYOUT
SECTIONS AND
LINER DETAILS

DRAWING NUMBER	SCALE	REV.
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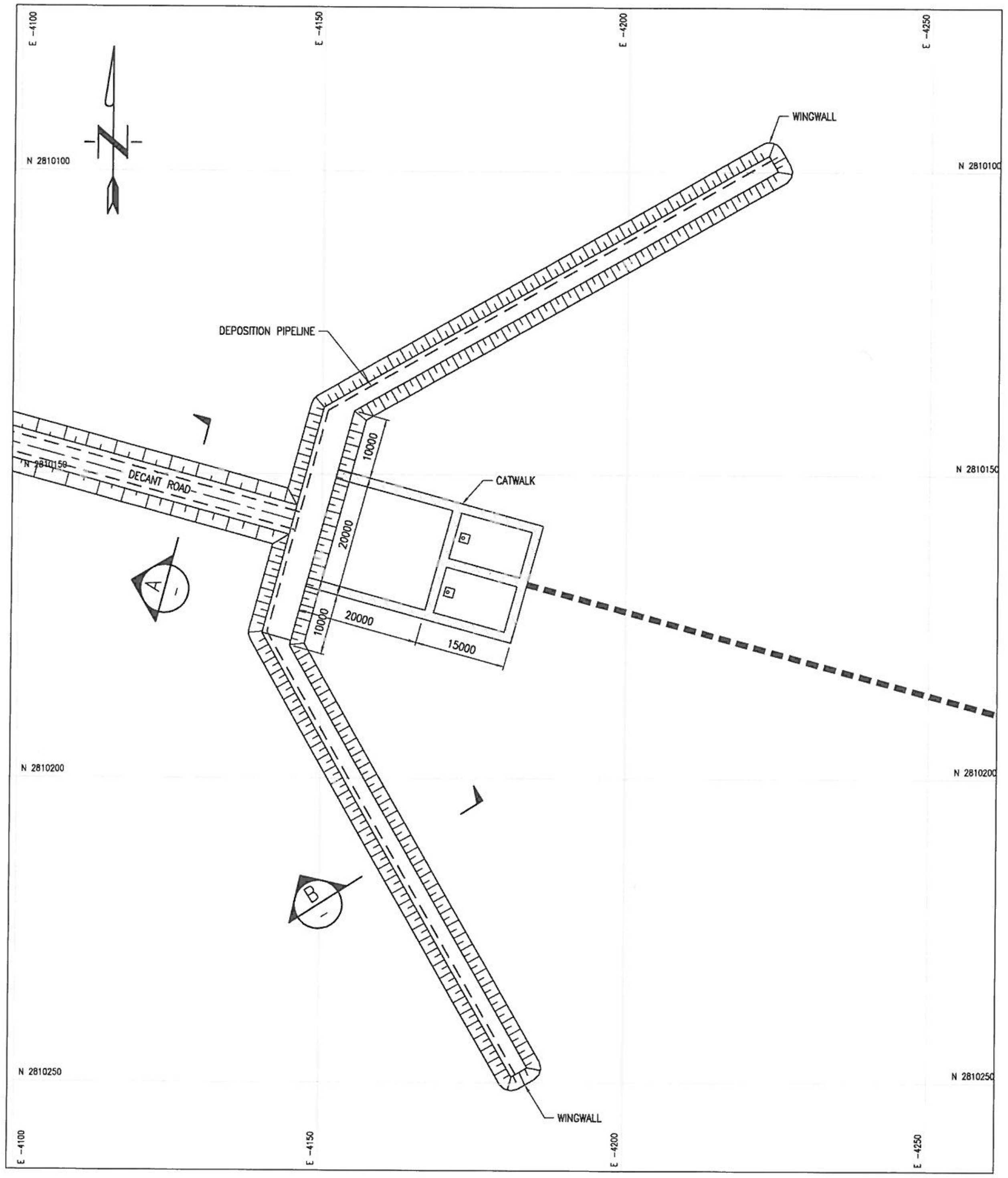
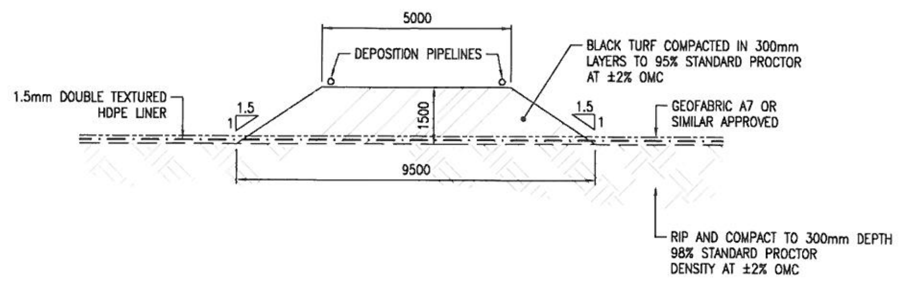


1 / 013 PERMANENT DECANT ACCESS ROAD DETAIL
SCALE 1: 500



A / – TYPICAL DECANT ACCESS ROAD SECTION
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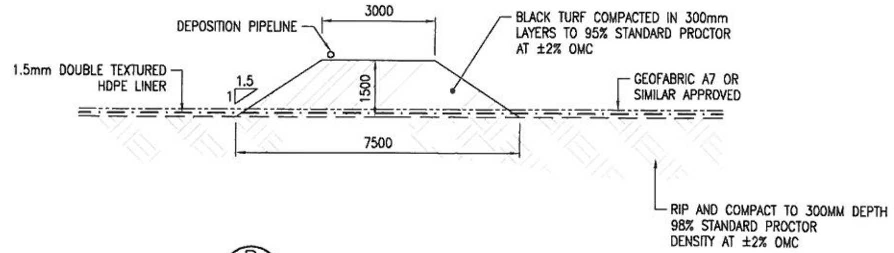

B / – TYPICAL DECANT WINGWALL SECTION
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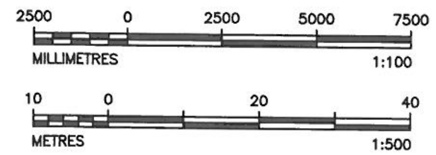

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Streem 20090268

PRIMARY DISCIPLINE	CIVIL	
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DESIGN CHECK	AS	28/08/2015
PROJECT ENGINEER	AS	–
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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
TRAINING WALL
SECTIONS
AND LAYOUT

DRAWING NUMBER	SCALE	REV.
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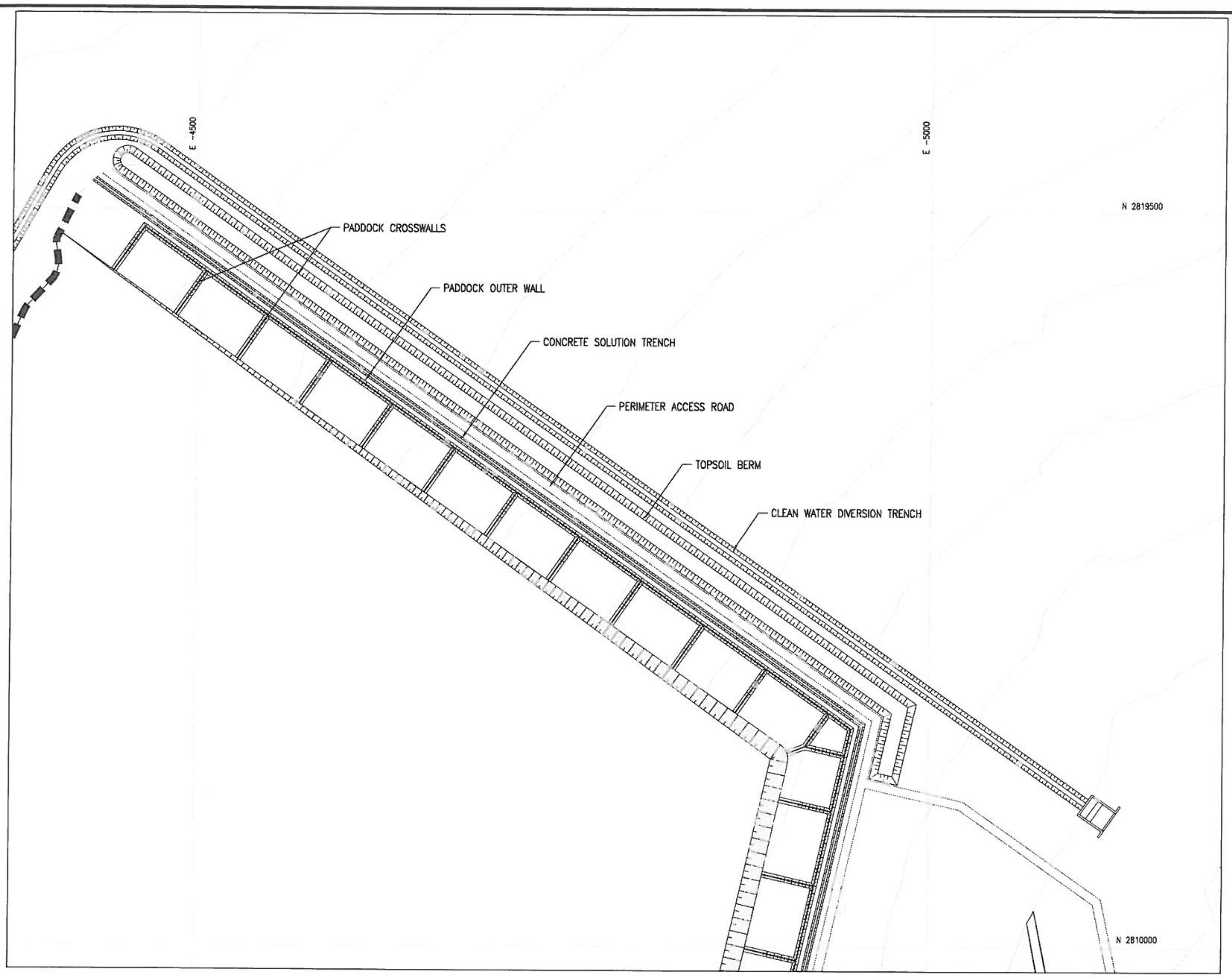

LAYOUT
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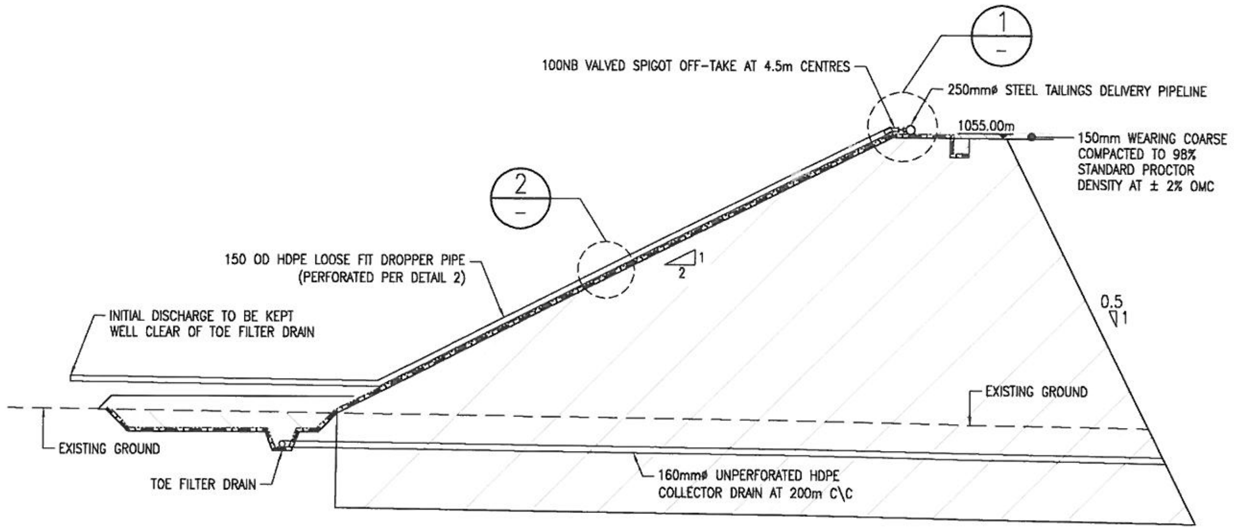
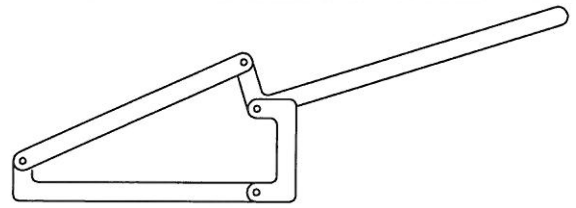
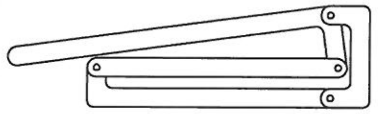


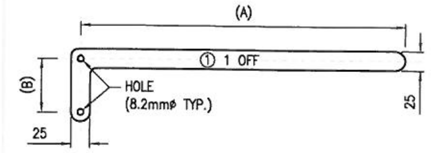
TYPICAL TAILINGS SPIGOT DELIVERY PIPE DETAIL
SCALE 1:100



SPIGOT CLAMP – OPEN POSITION



SPIGOT CLAMP – CLOSED POSITION



PART 1 – COMPONENT FABRICATION DETAILS
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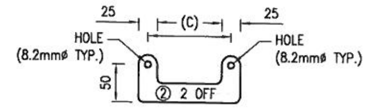

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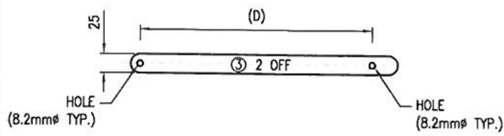

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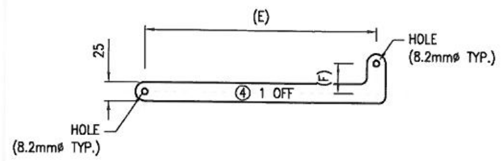

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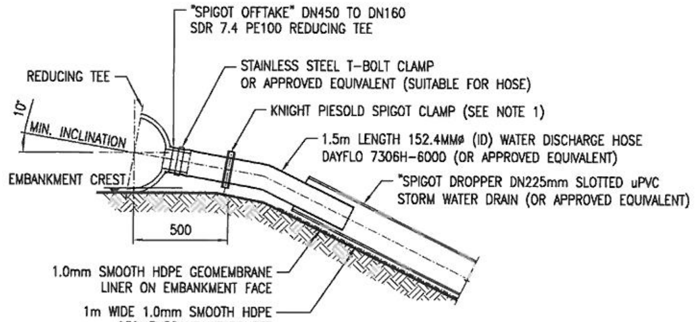

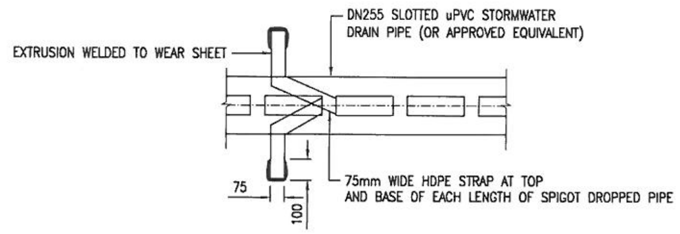

SPIGOT PIPEWORK
SCALE 1:20

SPIGOT PIPEWORK
SCALE 1:20

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Knight Piésold CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	ME	28/08/2015
DRAWING CHECK	TM & JZ	28/08/2015
DESIGN	RG	28/08/2015
DESIGN CHECK	AS	28/08/2015
PROJECT ENGINEER	AS	–

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REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
PHASE 1
SLURRY DISTRIBUTION
PIPEWORK AND DETAILS

DRAWING NUMBER	SCALE	REV.
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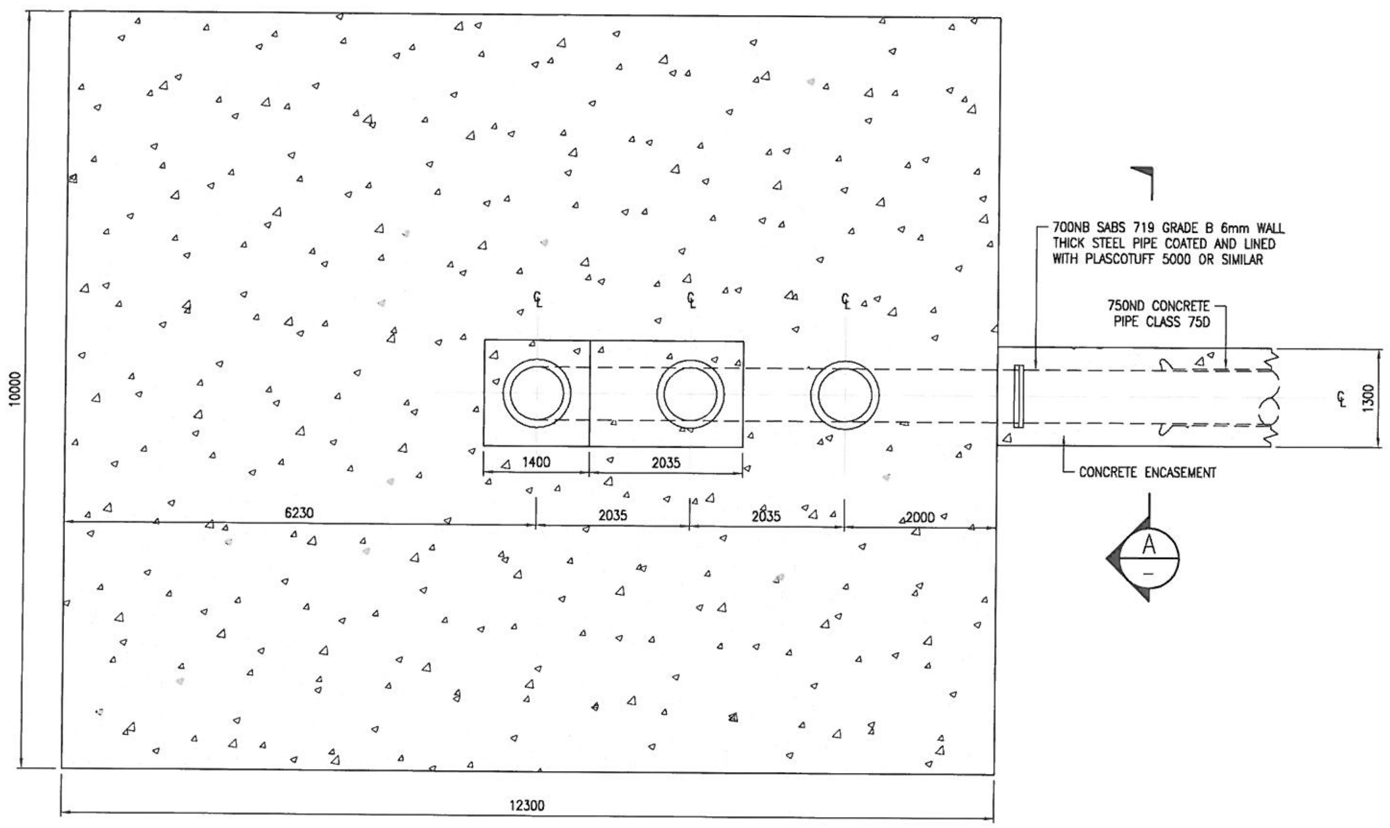
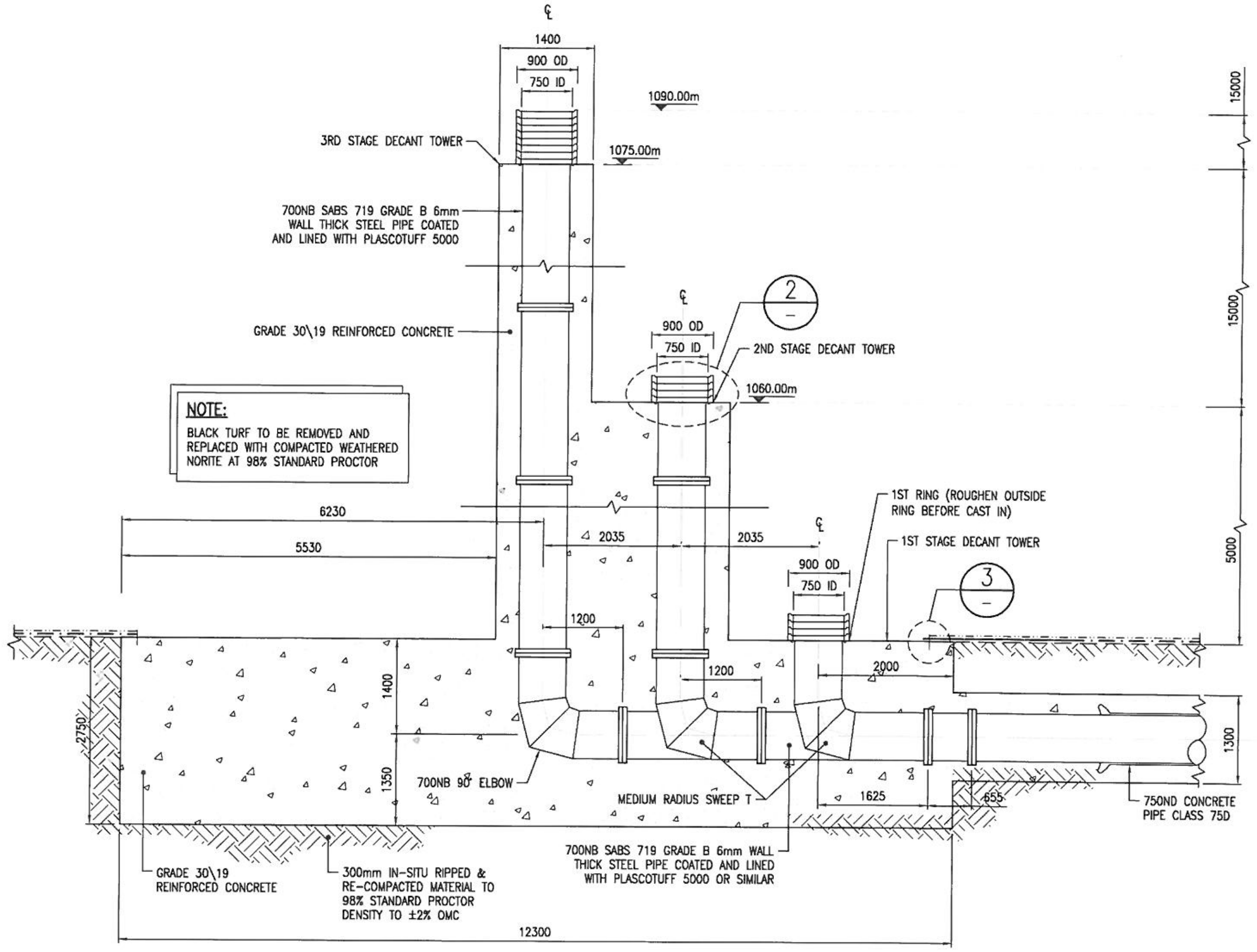


PERMANENT DECANT PLAN VIEW

SCALE 1:50



TYPICAL PERMANENT DECANT DETAILS

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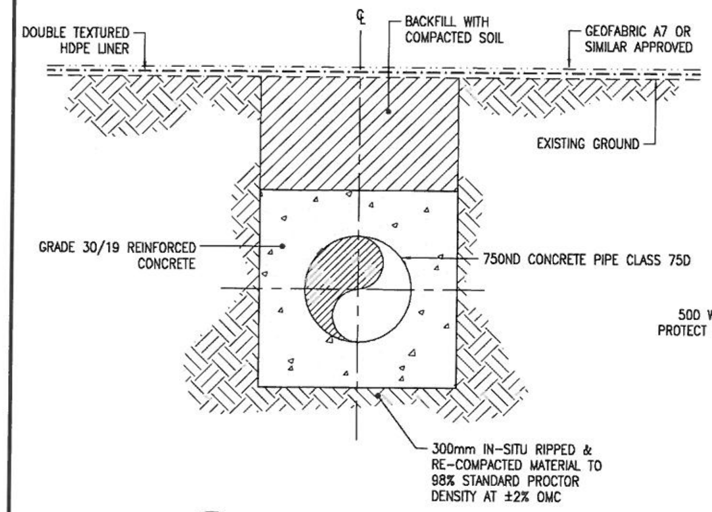

A TYPICAL CONCRETE ENCASING SECTION

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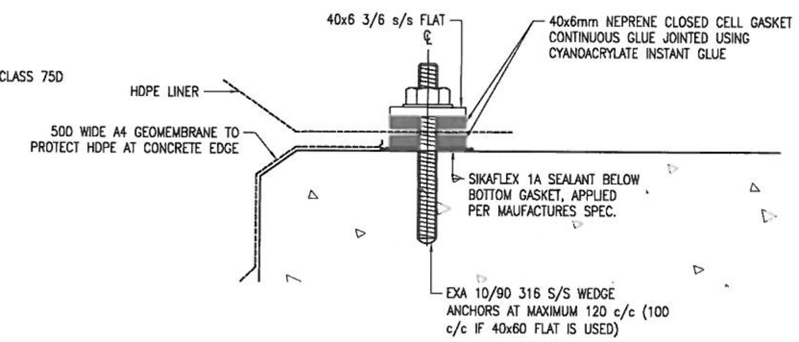
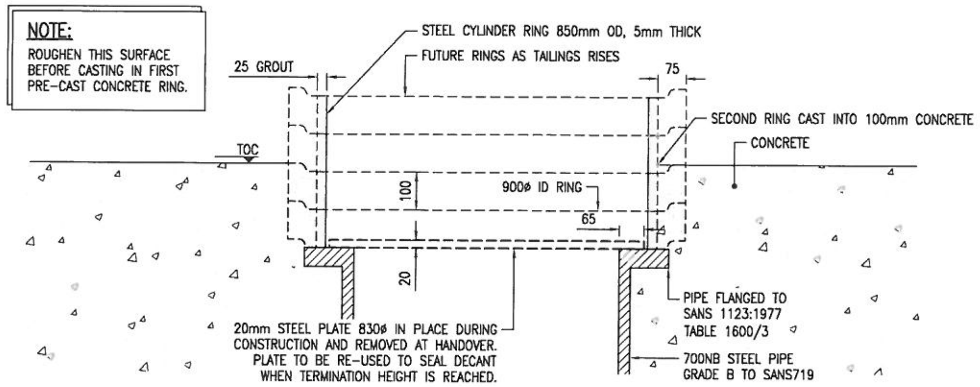


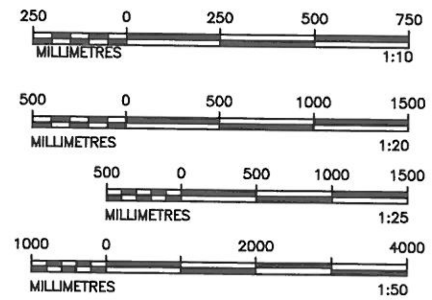
3 TYPICAL CONCRETE LINER INTERFACE

SCALE 1:2



2 TYPICAL DECANT CONCRETE LIFT DETAIL

SCALE 1:10



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PRIMARY DISCIPLINE	CIVIL	
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DESIGN	RG	28/08/2015
DESIGN CHECK	AS	28/08/2015
PROJECT ENGINEER	AS	–

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WESIZWE PLATINUM LTD.

MIMOSA TSF DESIGN

PERMANENT DECANT TYPICAL SECTIONS AND DETAILS

DRAWING NUMBER	SCALE	REV.
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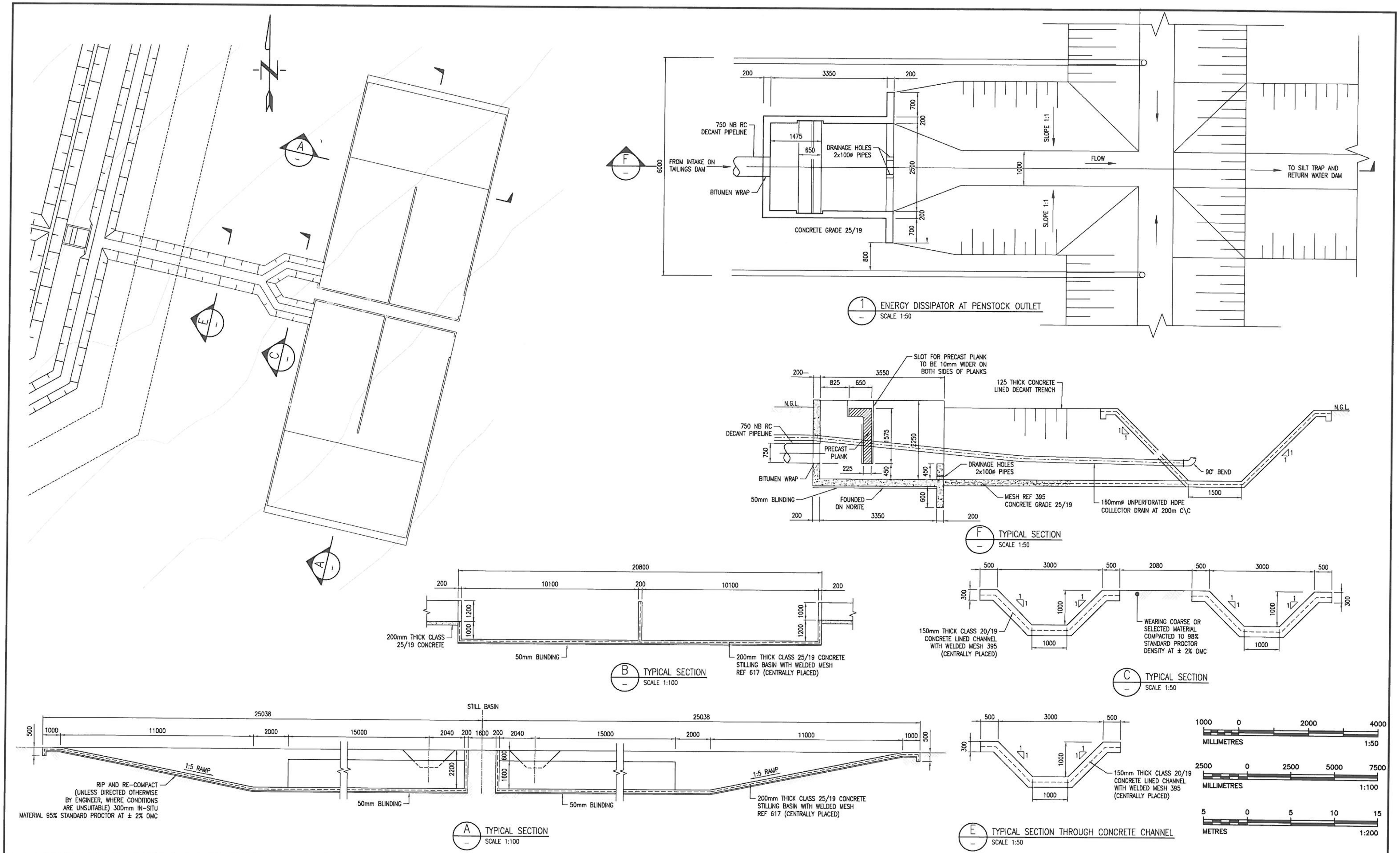

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DESIGN CHECK	AS	28/08/2015
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REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE
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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
SILT TRAP
DETAILS
AND SECTIONS

DRAWING NUMBER	SCALE	REV.
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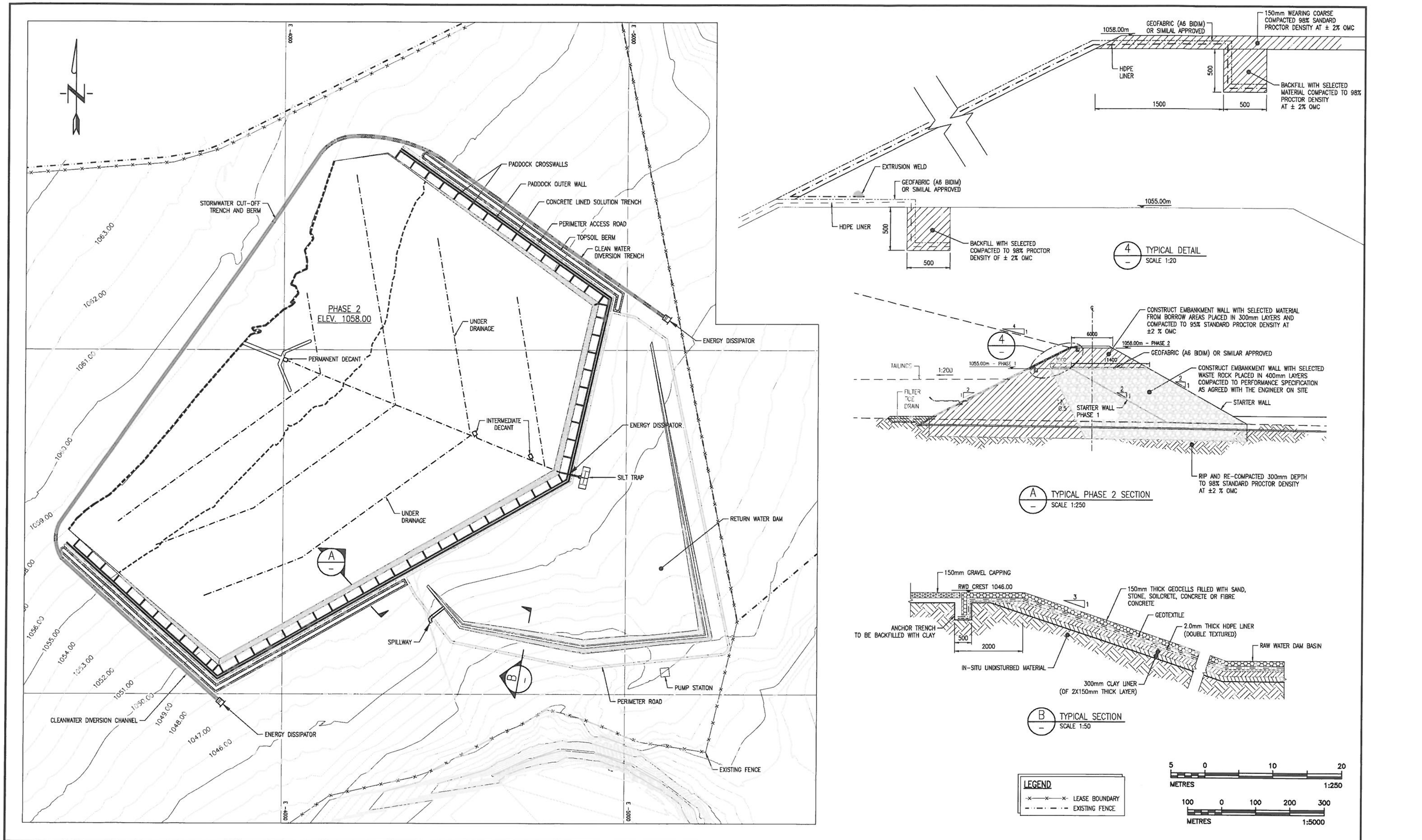

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DRAWING CHECK	TM & JZ	28/08/2015	
DESIGN	RG	28/08/2015	
DESIGN CHECK	AS	28/08/2015	
PROJECT ENGINEER	AS	–	

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20090268

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	22.01.16	ISSUED FOR INFORMATION	ME	MM	AS
B	23.02.16	ISSUED FOR INFORMATION	MKM	AS	AS

REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE
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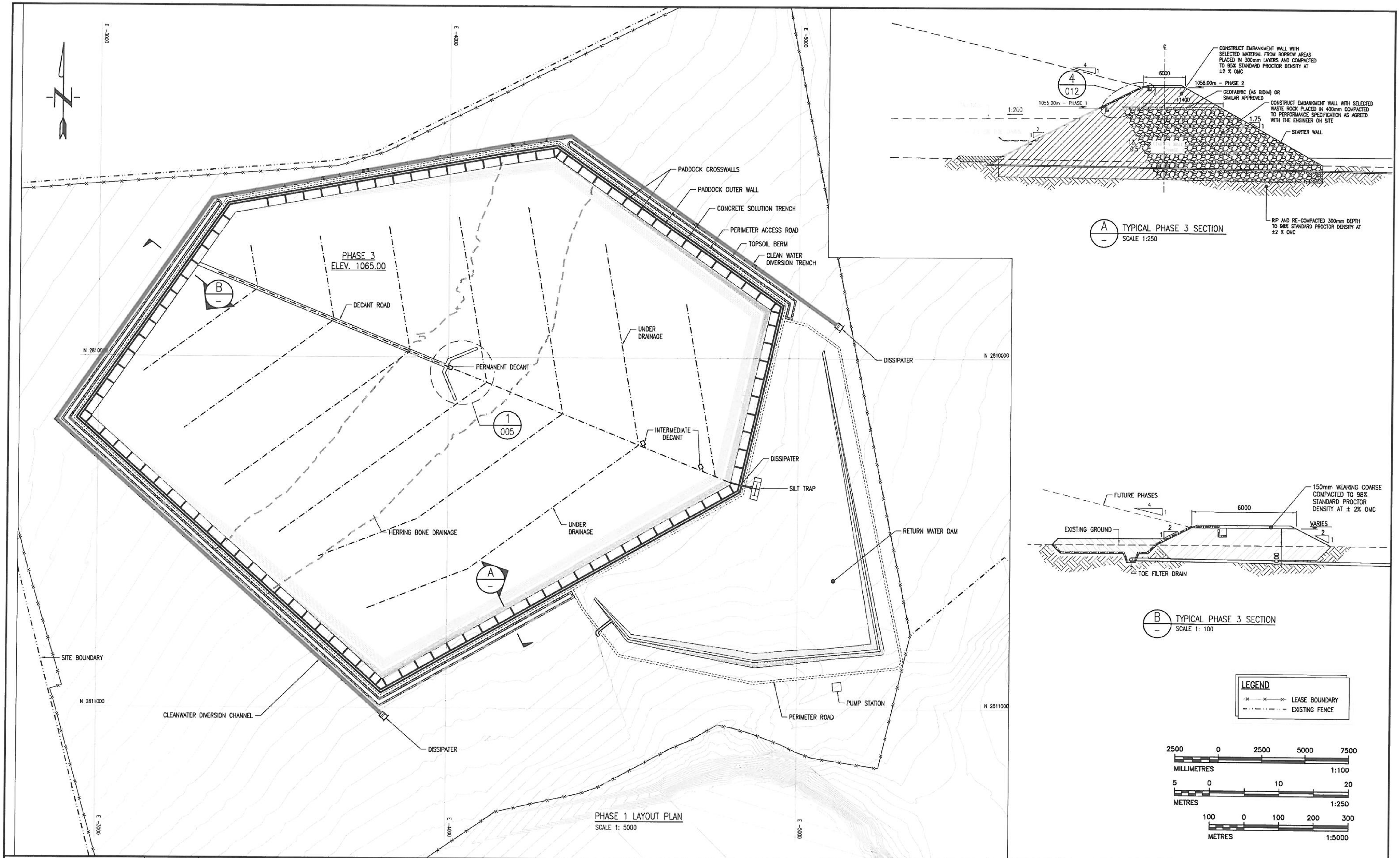
WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
PROJECTED CONSTRUCTION
PHASE 2
LAYOUT AND SECTIONS

DRAWING NUMBER	SCALE	REV.
301-00509/02-009	AS SHOWN	B



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2020268

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DRAWING CHECK	TM & JZ	28/08/2015
DESIGN	RG	28/08/2015
DESIGN CHECK	AS	28/08/2015
PROJECT ENGINEER	AS	–

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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN

PHASE 3
PROJECTED CONSTRUCTION
LAYOUT AND SECTIONS

DRAWING NUMBER	SCALE	REV.
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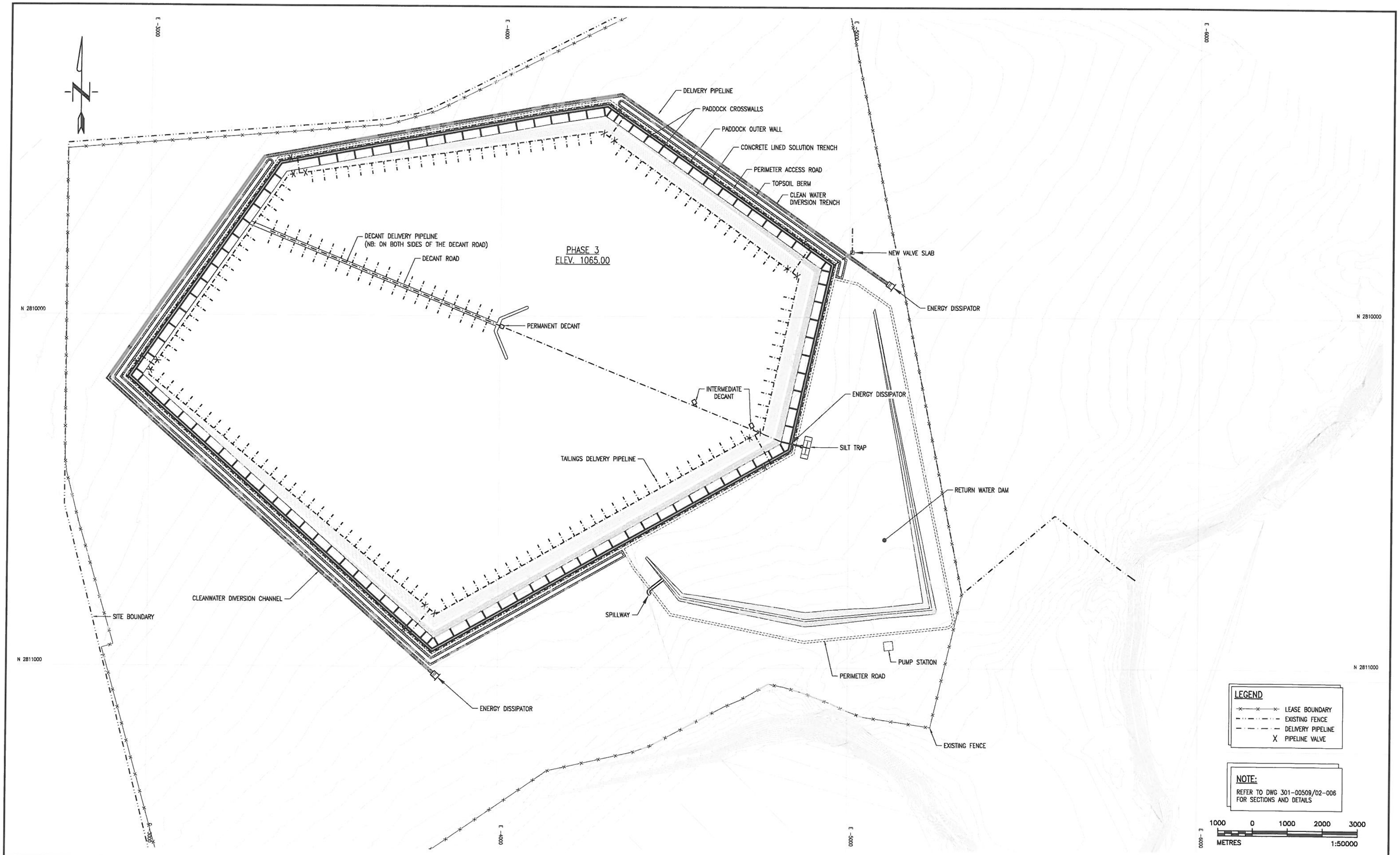

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20090268

PRIMARY DISCIPLINE	CIVIL	
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DRAWING CHECK	TM & JZ	28/08/2015
DESIGN	RG	28/08/2015
DESIGN CHECK	AS	28/08/2015
PROJECT ENGINEER	AS	–
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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
PHASE 3
SLURRY DISTRIBUTION
PIPELINE LAYOUT

DRAWING NUMBER	SCALE	REV.
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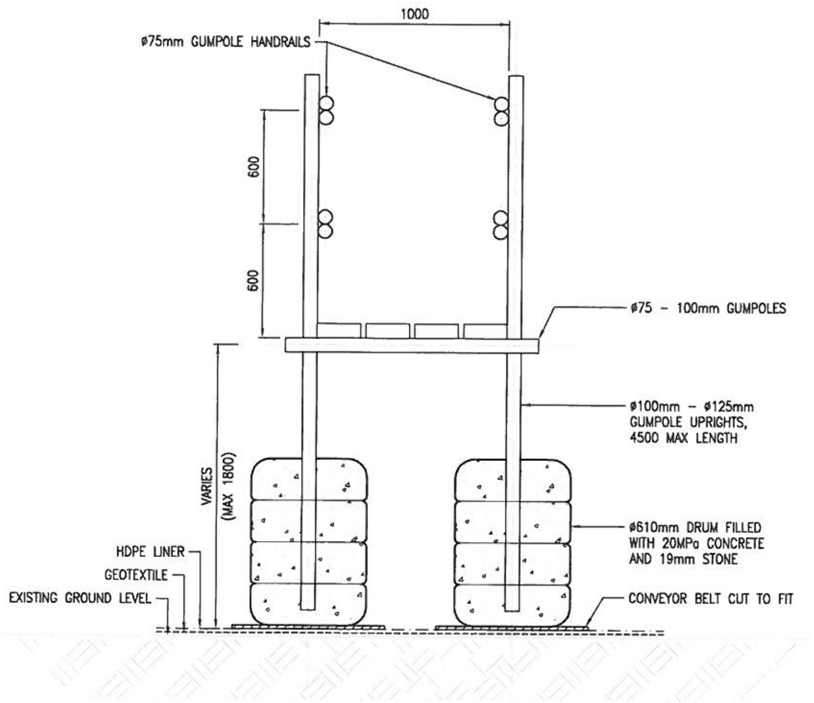

A TYPICAL CROSS SECTION
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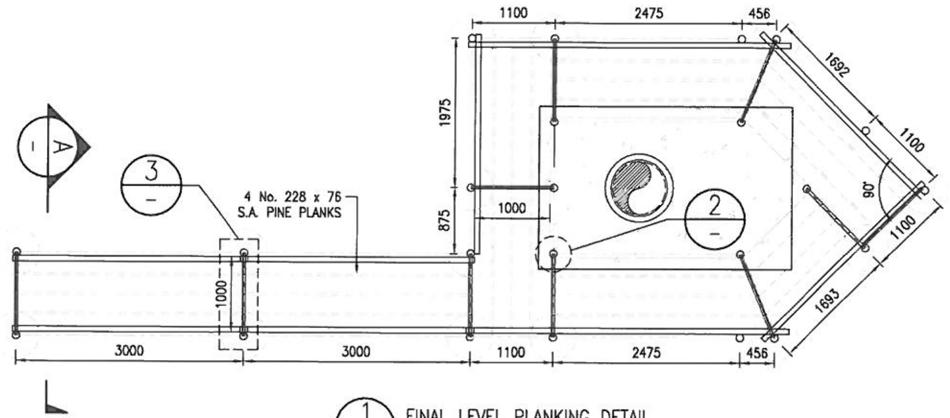
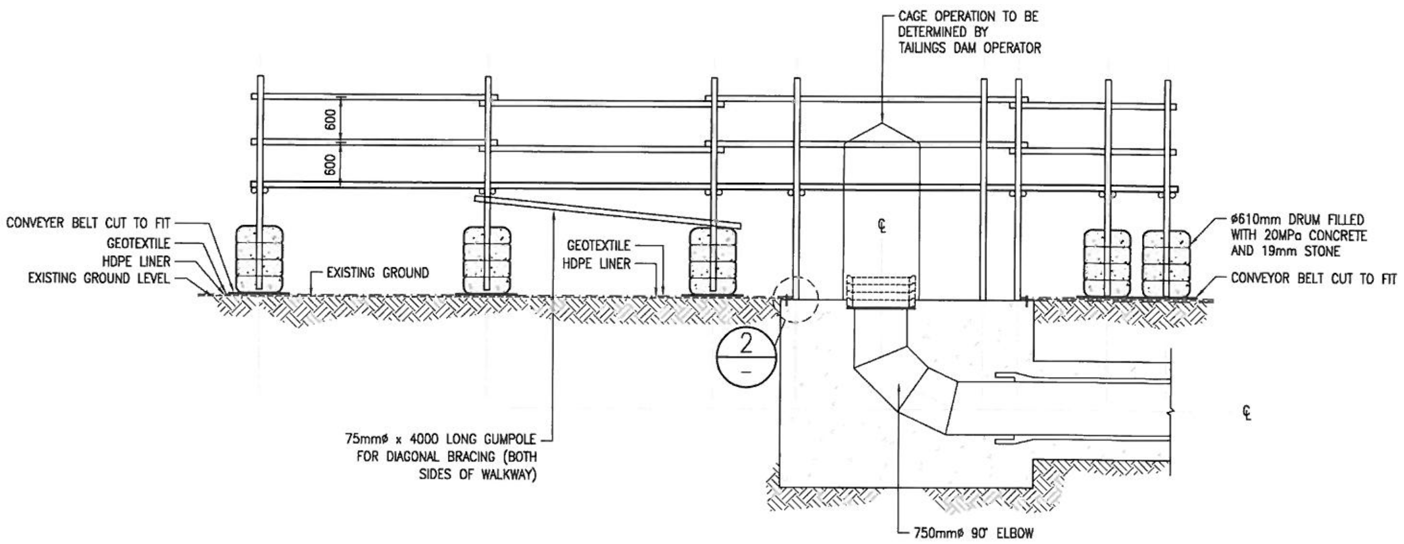
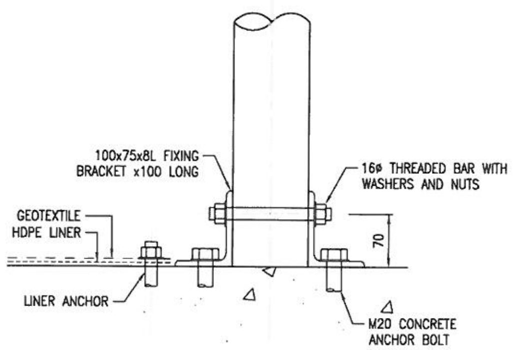


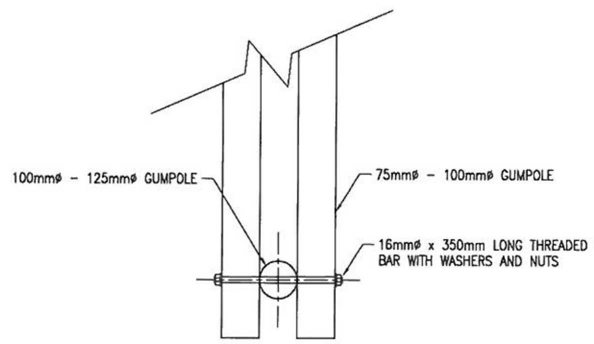
1 FINAL LEVEL PLANKING DETAIL
SCALE 1:50



PERMANENT DECANT SECTION
SCALE 1:50



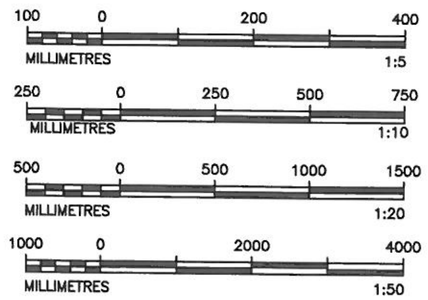
2 CONCRETE FIXING DETAIL
SCALE 1:5



3 TYPICAL DETAIL
SCALE 1:10

NOTE:

ALL WALKWAY PLANKS SHALL BE PRE-DRILLED THROUGH BEFORE NAILING TO SUPPORT GUMPOLES



Knight Piésold CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	ME	28/08/2015
DRAWING CHECK	TM & JZ	28/08/2015
DESIGN	RG	28/08/2015
DESIGN CHECK	AS	28/08/2015
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REVISION

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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
INTERMEDIATE
DECANT CATWALK
AND WALKWAY DETAILS

DRAWING NUMBER	SCALE	REV.
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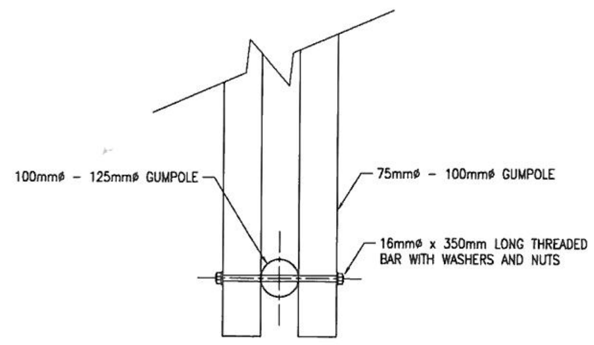
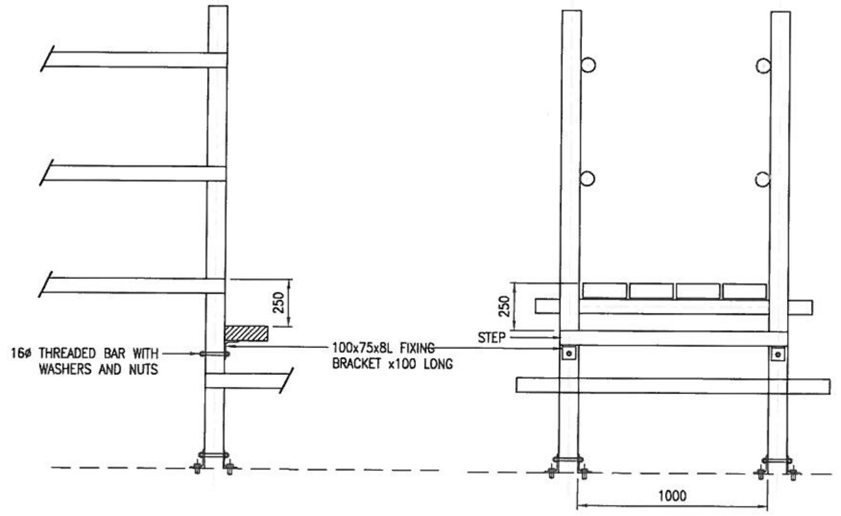
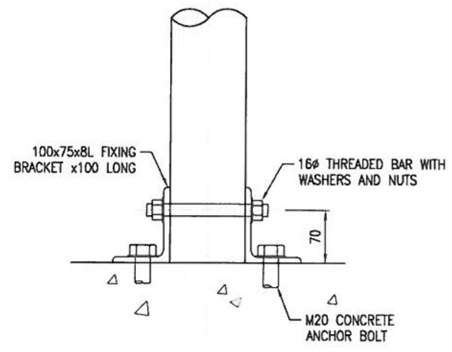


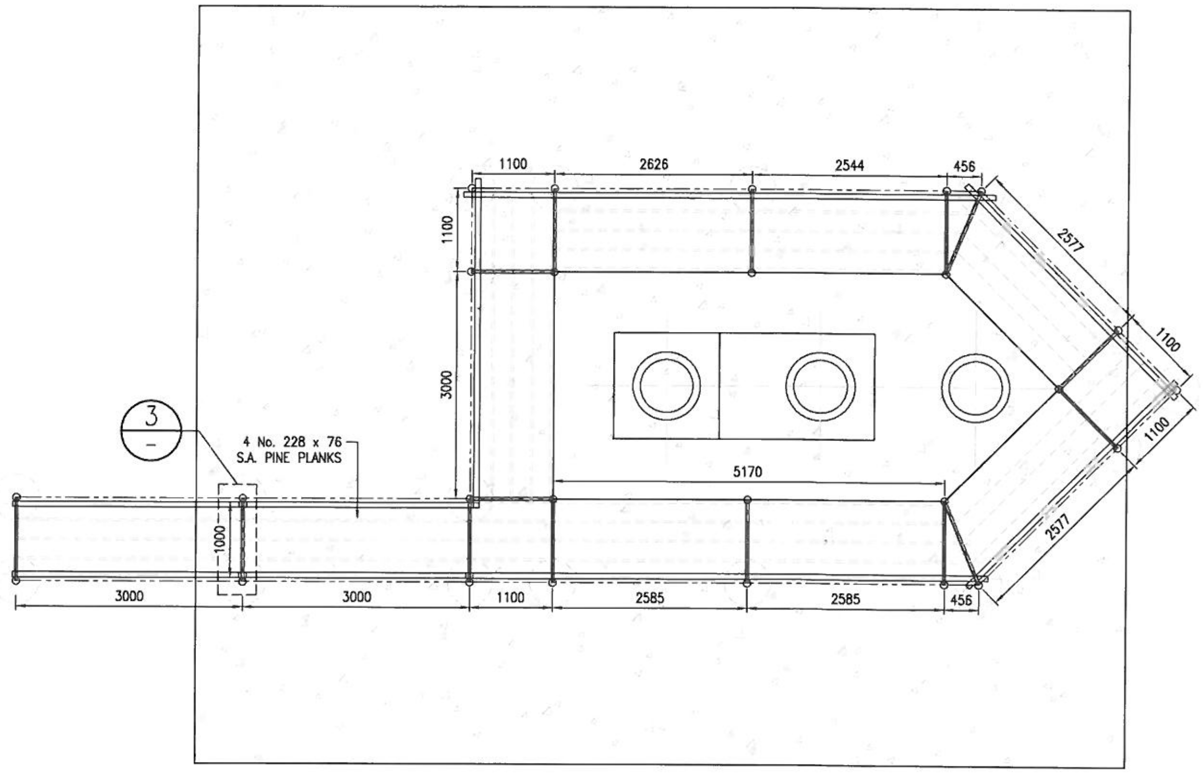
3 TYPICAL DETAIL
SCALE 1:10



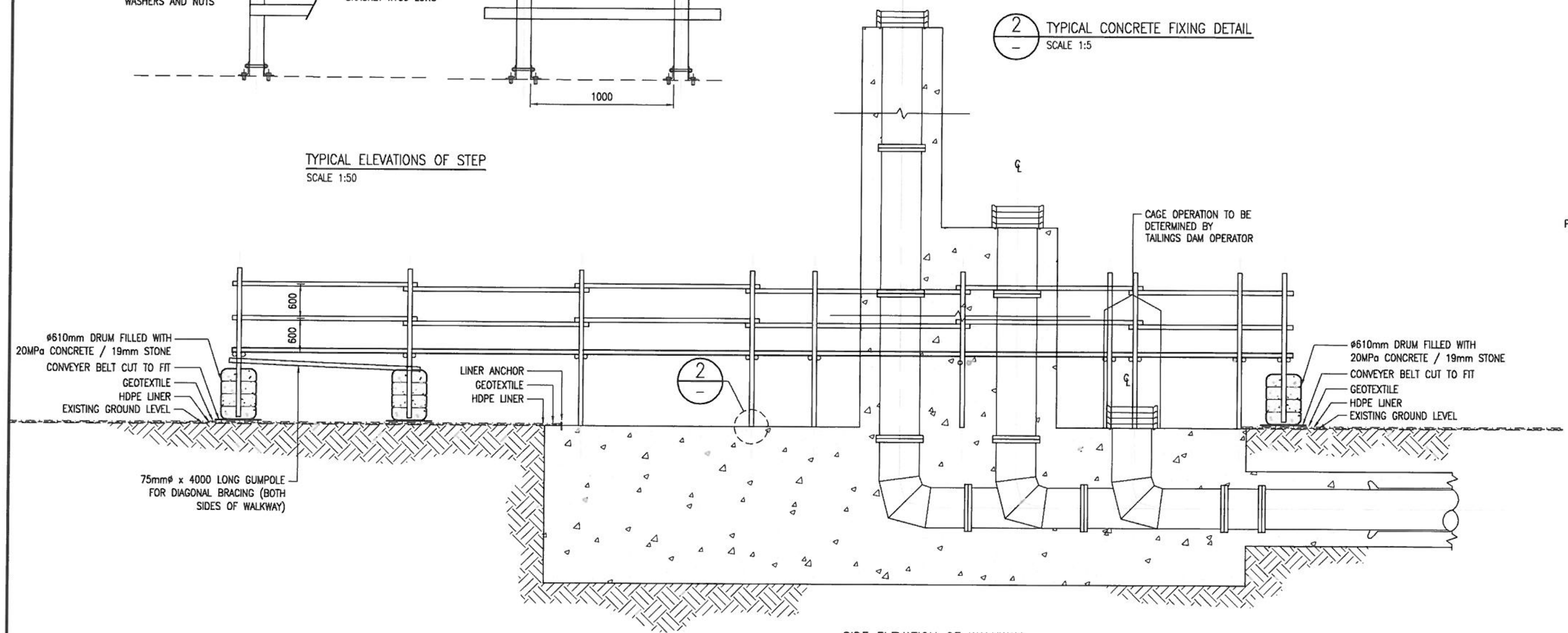
TYPICAL ELEVATIONS OF STEP
SCALE 1:50



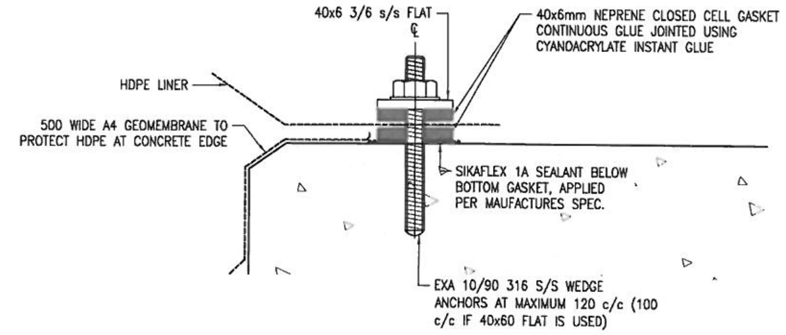
2 TYPICAL CONCRETE FIXING DETAIL
SCALE 1:5



1 FINAL LEVEL PLANKING DETAIL
SCALE 1:50



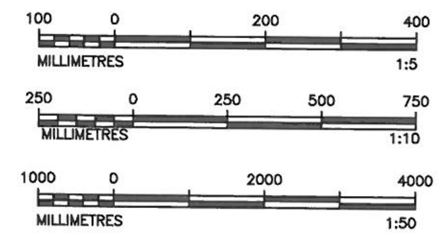
SIDE ELEVATION OF WALKWAY
SCALE 1:50



TYPICAL CONCRETE LINER INTERFACE
SCALE 1:2

NOTE:

ALL WALKWAY PLANKS SHALL BE PRE-DRILLED THROUGH BEFORE NAILING TO SUPPORT GUMPOLES



Knight Piésold CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	ME	28/08/2015
DRAWING CHECK	TM & JZ	28/08/2015
DESIGN	RG	28/08/2015
DESIGN CHECK	AS	28/08/2015
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REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
PERMANENT
DECANT CATWALK
AND WALKWAY DETAILS

DRAWING NUMBER	SCALE	REV.
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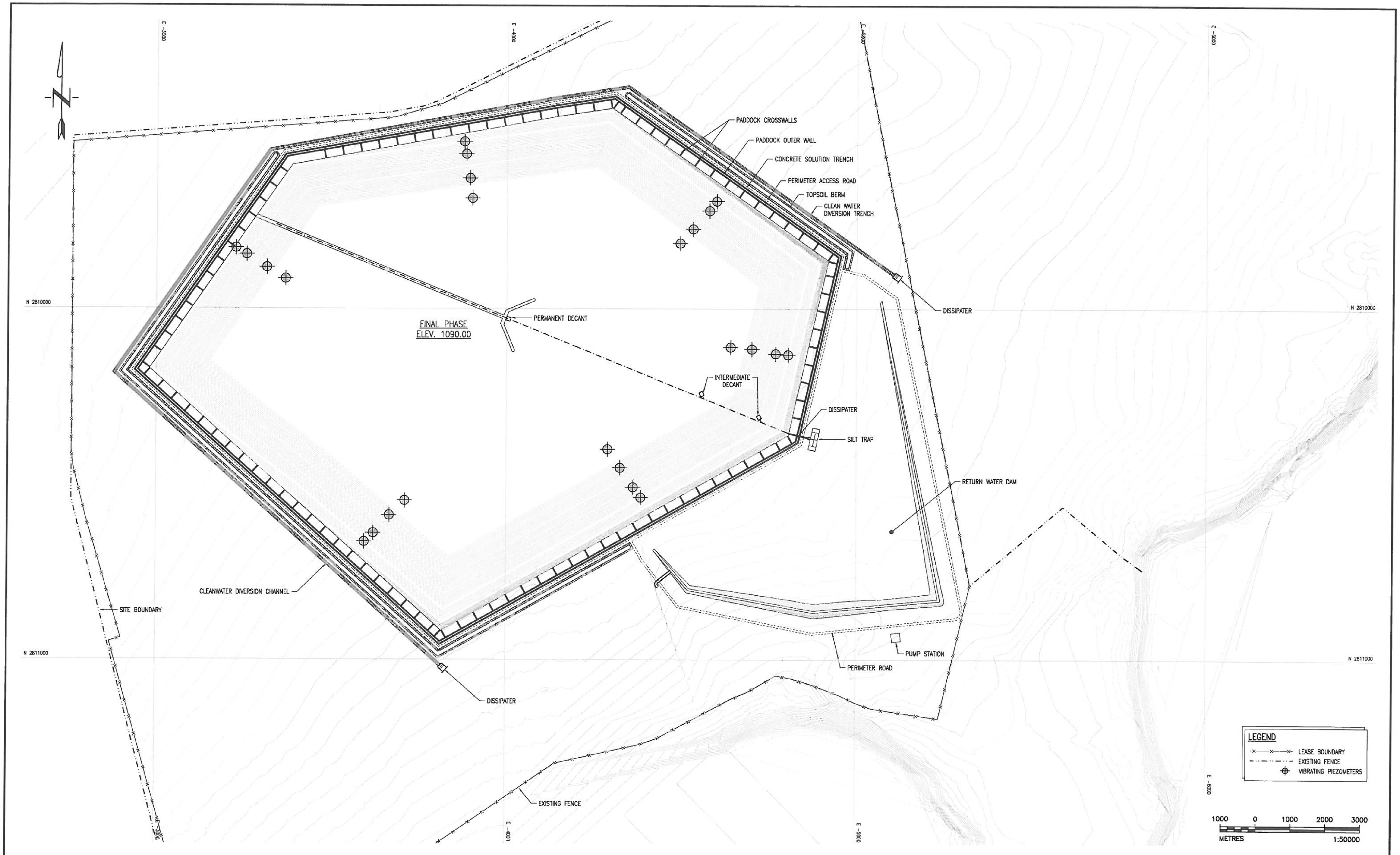

Knight Piésold CONSULTING

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PRIMARY DISCIPLINE	CIVIL	
DRAWN	ME	28/08/2015
DRAWING CHECK	TM & JZ	28/08/2015
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DESIGN CHECK	AS	28/08/2015
PROJECT ENGINEER	AS	-
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REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
INSTRUMENTATION
LAYOUT

DRAWING NUMBER	SCALE	REV.
301-00509/02-014	AS SHOWN	A

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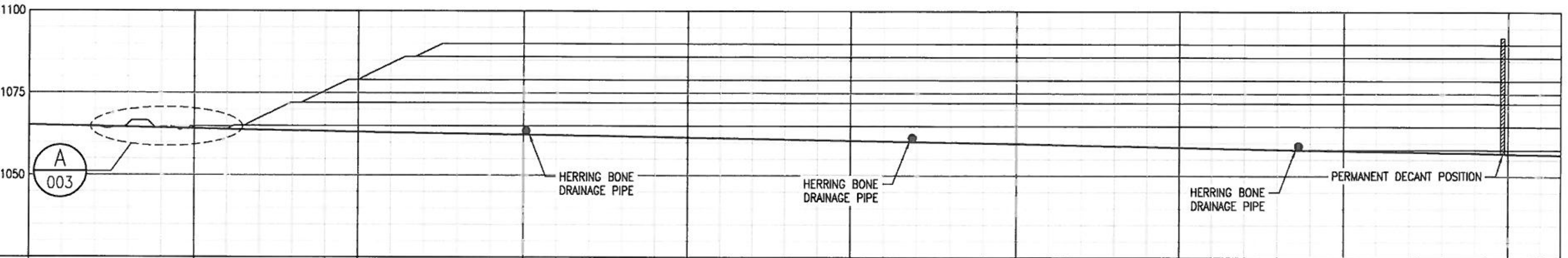
SCALES:
Horizontal 1:2000
Vertical 1:1000

DATUM 1025.00



CHAINAGE (m)	20.00	40.00	60.00	80.00	100.00	120.00	140.00	160.00	180.00	200.00	220.00	240.00	260.00	280.00	300.00	320.00
EXISTING GROUND LEVELS (m)	1064.94	1064.74	1064.53	1064.32	1064.11	1063.91	1063.71	1063.51	1063.32	1063.13	1062.94	1062.78	1062.63	1062.47	1062.31	1062.17

CHAINAGE (m)	340.00	360.00	380.00	400.00	420.00	440.00	460.00	480.00	500.00	520.00	540.00	560.00	580.00	600.00	620.00
EXISTING GROUND LEVELS (m)	1062.02	1061.84	1061.67	1061.49	1061.31	1061.14	1060.96	1060.78	1060.60	1060.43	1060.25	1060.07	1059.87	1059.67	1059.46

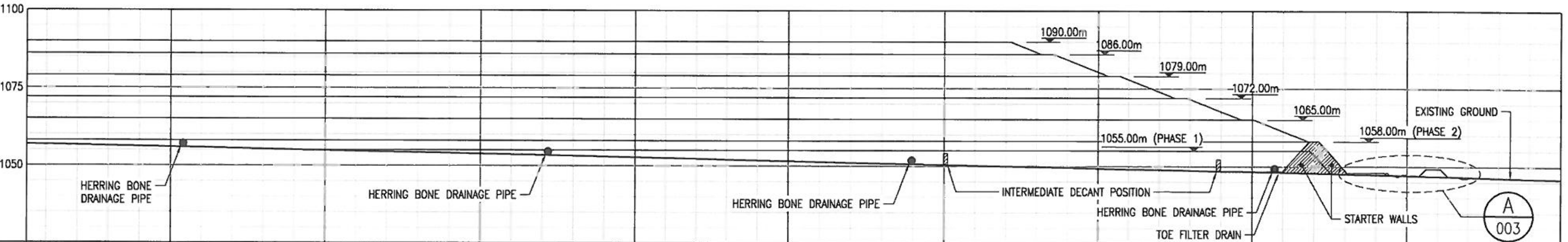
CHAINAGE (m)	640.00	660.00	680.00	700.00	720.00	740.00	760.00	780.00	800.00	820.00	840.00	860.00	880.00	900.00	920.00
EXISTING GROUND LEVELS (m)	1059.25	1059.04	1058.83	1058.63	1058.44	1058.24	1058.04	1057.85	1057.66	1057.52	1057.37	1057.18	1057.00	1056.81	1056.64

LONGITUDINAL SECTION CH: 0.00m - 920.00m

SCALE HOR - 1: 2000 VER - 1:1000

SCALES:
Horizontal 1:2000
Vertical 1:1000

DATUM 1025.00



CHAINAGE (m)	920.00	940.00	960.00	980.00	1000.00	1020.00	1040.00	1060.00	1080.00	1100.00	1120.00	1140.00	1160.00	1180.00	1200.00	1220.00	1240.00
EXISTING GROUND LEVELS (m)	1056.64	1056.45	1056.25	1056.05	1055.87	1055.70	1055.50	1055.26	1055.01	1054.80	1054.59	1054.40	1054.20	1053.99	1053.78	1053.56	1053.34

CHAINAGE (m)	1260.00	1280.00	1300.00	1320.00	1340.00	1360.00	1380.00	1400.00	1420.00	1440.00	1460.00	1480.00	1500.00	1520.00	1540.00	1560.00	1580.00
EXISTING GROUND LEVELS (m)	1053.12	1052.90	1052.66	1052.43	1052.20	1051.96	1051.72	1051.48	1051.24	1051.00	1050.78	1050.58	1050.36	1050.13	1049.91	1049.68	1049.45

CHAINAGE (m)	1600.00	1620.00	1640.00	1660.00	1680.00	1700.00	1720.00	1740.00	1760.00	1780.00	1800.00	1820.00	1840.00	1860.00	1880.00	1900.00
EXISTING GROUND LEVELS (m)	1049.23	1049.02	1048.82	1048.63	1048.44	1048.23	1048.02	1047.76	1047.50	1047.29	1047.06	1046.81	1046.56	1046.33	1046.09	1045.81

LONGITUDINAL SECTION CH: 920.00m - 1900.00m

SCALE HOR - 1: 2000 VER - 1:1000

NOTE:

REFER TO DRG. 301-00509\02-001 FOR ALIGNMENT

25 0 25 50 75
METRES 1:1000

50 0 50 100 150
METRES 1:2000

Knight Piésold
CONSULTING

20090268

PRIMARY DISCIPLINE	CIVIL	
DRAWN	ME	28/08/2015
DRAWING CHECK	TM & JZ	28/08/2015
DESIGN	RG	28/08/2015
DESIGN CHECK	AS	28/08/2015
PROJECT ENGINEER	AS	-
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WESIZWE PLATINUM LTD.
MIMOSA TSF DESIGN
LONGITUDINAL
SECTIONS
CH. 0.00M - 1920.00

DRAWING NUMBER	SCALE	REV.
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APPENDIX C

LINER SERVICE LIFE MEMOS (LITERATURE REVIEW)

GSE Lining Technology GmbH
Normannenweg 28
D-20537 Hamburg Germany
[O] 0049 40 767420
[F] 0049 40 7674234

Knight Piésold

22nd January 2016

Dear Andries Strauss,

Please find below the technical summary of your enquiry to us and our response.

<u>Re: Wesizwe Tailings dam liner compliance requirements (alternative proven performance and service liner considerations)</u>

1.<u>Containment Barrier systems background</u>

The purpose of the containment barrier systems is to substantially reduce the rate of seepage into the underlying soils of the waste facility. The most convenient way of providing a base barrier system is to use the in-situ soils but this requires soils with a relatively low permeability (generally less than $1x10^{-8}$m/s).

A composite liner is a liner that consists of two or more components. In virtually all cases where a composite liner is used in a waste containment facility, the composite liner consists of a geomembrane and a low-permeability soil layer. Typically, the geomembrane component of the composite liner is placed on top of the low-permeability soil layer, which decreases percolation of leachate into the liner and promotes lateral flow of leachate in the leachate collection layer overlying the composite liner since the geomembrane is less permeable than the low-permeability soil. Leachate collection and removal is maximized and percolation of leachate into the liner is minimized.

In certain instances a lack of suitable soils has resulted in geosynthetic options being considered and the use of synthetic liners (such as geomembranes) in waste storage facilities has increased in recent years.

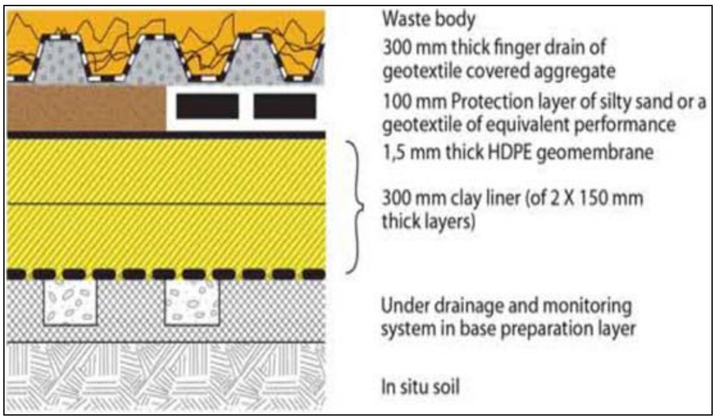
Geomembranes are often included as part of the engineered barrier system for modern landfills. As defined in ASTM D4439-00, a geomembrane is ‘‘an essentially impermeable membrane used with foundation, soil, rock earth or any other geotechnical engineering-related material as an integral part of a man-made project, structure or system’’. The selection of a geomembrane liner depends upon the application in which it will be used.

2. Regulatory Compliance Requirements

The Tailings has to be classified in line with the relevant waste regulations. The Minimum Requirements have been superseded by the Waste Classification and Management Regulations (WCMR) which was promulgated on 23 August 2013 (GN R. 634 of 2013), which were promulgated in terms of the National Environmental Management: Waste Act, 2008 (Act 59 of 2008) (NEMWA) with the following associated Norms and Standards:

- Norms and Standards for Assessment of Waste for Landfill Disposal prior to the disposal of waste to landfill (GN R.635, 23 August 2013); and
- The National Norms and Standards for Disposal of Waste to landfill (GN R.636, 23 August 2013).

The tailings has been classified as a type 3 waste, and therefore needs to comply to a Class C landfill liner design facility as shown in the figure below.
The returnwater dam shall comply with the same landfill liner classification as the tailings dam.



The tailings dam and returnwater liner design needs to further comply to clauses of the Regulation R636 section 3 items (2) (b) to (i) as listed below with regards to the chosen containment barrier system of the facility.

1. Service life considerations that must be quantified taking into account temperature effects on containment barriers;
2. Total solute seepage (inorganic and organic) that must be calculated in determining acceptable leakage rates and action leakage rates;
3. Alternative elements of proven equivalent performance which has been considered, such as the replacement of -
   (i) granular filters or drains with geosynthetic filters or drains;
   (ii) protective soil layers with geotextiles; or
   (iii) clay components with geomembranes or geosynthetic clay liners;
4. All drainage layers must contain drainage pipes of adequate size, spacing and strength to ensure atmospheric pressure within the drainage application for the service life of the landfill;

5. Alternative design layouts for slopes exceeding 1:4 (vertical: horizontal) may be considered provided equivalent performance is demonstrated;
6. Construction Quality Assurance during construction;
7. Geosynthetic materials must comply with relevant South African National Standard specifications, or any prescribed management practice or standards which ensure equivalent performance; and
8. Consideration of the compatibility of liner material with the waste stream, in particular noting the compatibility of natural and modified clay soils exposed to waste containing salts.

3. Selected liner design

Knight Piésold have designed a Tailings dam (TSF) and return water dam (RWD) with a standard GRI GM 13 (latest edition November 2014) HDPE material with a number of variations to the standard as per technical specifications. The TSF liner design with include a 1.5mm thickness smooth HDPE geomembrane and the Returnwater dam a 2.0mm HDPE smooth geomembrane.

The selection of the 1.5mm HDPE thickness has been chosen in light of the equivalent performance of a composite liner of a 1,5mm HDPE GM plus 300mm thick CCL of at least 10^{-6} cm/s material performance could be met.
The underliner soils on the site are predominantly made of either black sandy/silty clay colluvium (Soil Zone A) or reddish brown/red colluvium (Soil Zone B). Therefore partial replacement of the clay with other materials such as a geomembrane with thickness dependant on permeability has been the desirable and best way forward to demonstrate equivalent performance. The assumption is that a 1.5mm thick geomembrane at 10^{-14} cm/s permeability with limited damage (as per the CQA requirement) will meet the specified equivalent performance of the barrier.

For the returnwater dam, a liner of thicker than standard geomembrane (1.5mm in thickness) with stringent CQA has been chosen to argue the equivalent performance as per prescribed regulations.
This recommendation is motivated by the additional 0.5mm HDPE GM thickness having permeability of 10^{-14} cm/s being equivalent to the performance of or at least partially replacing the CCL, with the underlying compacted base preparation providing some composite effect at discontinuities.

The geomembrane on both facilities will be constructed in accordance with the current standard of practice for geomembrane liner installation, as outlined in the technical specifications and CQA plan. Seams will be welded to provide a continuous geomembrane liner. Testing during construction will include both non-destructive and destructive testing as outlined in the technical specifications and CQA plan.

4. Service life aspects of chosen containment barrier systems

Estimates of geomembrane lifetime prediction under exposed atmospheric conditions are required in many civil engineering applications. For example, surface impoundments and canal liners above their liquid levels, floating covers on reservoirs, waterproofing of dams, exposed geomembrane landfill covers, etc., are all major applications for such exposed geomembranes. Comments such as a “long time” or “very long time” are usually inadequate in that an estimate of the expected number of years is required.

DWS would like design engineers (Knight Piésold) to demonstrate the longevity of the proposed 1.5 & 2.0mm HDPE liner in the intended applications for the above project. There is usually concern of the HDPE liner installed along the side slopes of the RWD shall be exposed to UV and deterioration over time without some form of protective cover.
The TSF will be covered with Tailings approximately 5 months within commissioning and therefore exposure concerns are limited. It is important to demonstrate the service life of the HDPE liner in comparison to the Life of Mine (25 years) of the anticipated UV degradation of the geomembrane during this time. If the service life of the HDPE liner is 2x or greater than the Life of Mine, then no protective cover is required.

HDPE geomembranes have been used throughout the world in a range of environmental protection applications which include landfill applications but also several applications in the mining industry. Such projects executed with HDPE-geomembranes are requiring different service life – for tailing impoundments it is usually 30 to 100 years. Such service life can be provided by “high quality” HDPE-geomembranes with sufficient thickness and stringent CQA. Facilities that have been designed and constructed inline with this requirement, technical research has shown that the service life, in particular the impermeability is not impacted, which is a clear advantage of HDPE. The below explains the advantage of HDPE-products based on their molecular structure in general:

i. **Chemical resistance:**

HDPE itself does provide an outstanding chemical resistance which is related to the simple polyethylene chain structure (hydrocarbon and carbon bond structure only) - thus no elements which could potentially leach out and no bonds which are easily affected or altered by chemicals. For this reason HDPE geomembranes have been proven to be the most suitable product for the containment of almost all chemicals (only fuming acids are known to cause a strong ageing process). HDPE is used for small package units of chemicals, for pipe systems and for geomembranes in a range of applications besides its use in a lot of other applications.

ii. **Ageing resistance:**

Nonetheless, not only the chemical resistance needs to be taken into account for applications which demand long-term durability, but also the ageing resistance. Again, due to the simple molecular structure there is only one ageing mechanism which affects HDPE, which is oxidation. The oxidation process is a chain reaction which is triggered or offset by free radicals that result in chain breaking, thus creating shorter polymers and in the end embrittlement of the product. Therefore HDPE needs and has to be stabilized against the oxidation process with anti-oxidants. As long as the stabilizers are active, the polymer chains do not break, thus there is no change of properties- including the impermeability.

iii. **Stress crack resistance:**

HDPE for geomembrane applications consists of the long polyethylene chains, but it does also have short side chains (also C / H bonds - so called □-olefins) along the long polyethylene chains, which allows for the difference in HDPE-types with regard to mechanical characteristics and its stress crack performance. An HDPE- product that provides from the beginning a very high resistance against embrittlement (high stress

crack resistance) is a preferred option if long service live is required and in cases where an aggressive environment is expected.
The above is a simplified and summarized description on HDPE materials, and needs to be further highlighted for geomembrane applications and specifically for the above application.

iv. Service Life of HDPE-geomembranes:

International research work on HDPE-liners has led to the conclusion, that service life time of HDPE-liners with sufficient thickness and produced according to the state of the art from appropriate raw materials can exceed 100 years by far. This statement is based on the evaluation of the properties after accelerated lab ageing, but also per experience on site-exposed liners.

The most reliable method of determining the service life of geomembranes would be from exposure under the actual field conditions, this is not presently feasible due to the length of time that would be required to obtain useful results.
In addition, there is a little of field performance records from which the service life may be deduced. Consequently, several "accelerated ageing" tests have been developed which attempt to simulate long-term exposure of high-density polyethylene (HDPE) geomembranes and to address their durability and degradation issues for landfills and other containment applications. These have been done under different scenarios to those observed at Wesizwe Platinum Project and it most cases exposed to site leachate which is not the case at Wesizwe Platinum Project where the facilities will merely contain stormwater and decant water from the TSF (inert tailings), however the principle in the service life interpretation and extrapolation of the geomembrane from current research work will be applied.

Lifetime predictions of hundreds of years have been estimated by research institutions such as the Geosythenthics Research Institute (GSI) and several authors. Existing proposals for expected geomembrane lifespan in exposed applications as is being considered at Wesizwe Platinum Project are well established. While both Kerry Rowe and Bob and George Koerner have done work in this area.
We have noted that some of Kerry Rowe's research on the subject in particular reference to the following publications;

1. Durability of HDPE geomembranes R. Kerry Rowe*, Henri P.Sangam Department of Civil Engineering, GeoEngineering Centre at Queen's-RMC, Queen's University, Kingston, Ont., Canada K7L 3N6
2. Antioxidant Depletion from a High Density Polyethylene Geomembrane under Simulated Landfill Conditions R. Kerry Rowe, F.ASCE1; M. Z. Islam, M.ASCE2; R. W. I. Brachman3; D. N. Arnepalli4; and A.Ragab Ewais5
3. Effects of exposure conditions on the depletion of antioxidants from high-density polyethylene (HDPE) geomembranes, Henri P. Sangam and R. Kerry Rowe
4. Hsuan, Y. G. and Koerner, R. M., "Antioxidant Depletion Lifetime in High Density Polyethylene Geomembranes," Jour. Geotech. and Geoenviron. Engr., ASCE, Vol. 124, No. 6, 1998, pp.532-541.
5. Yako, M. A., Koerner, G. R., Koerner, R. M. and Hsuan, Y. A. N. G. (2010), "Case History of a 20-Year Old Exposed HDPE Surface Impoundment Liner," Proc. 9th IGS Conference, Brazil, May 23-27, pp. 805-808

6. Koerner, R. M. (2012), Designing With Geosynthetics, 6th Edition, Xlibris Publishing Co., Indianapolis, Indiana, 950 pgs.

GSI white papers titles below are used as the core basis of the technical reference of this report, with detailed reference made to the above authors for verification where necessary.

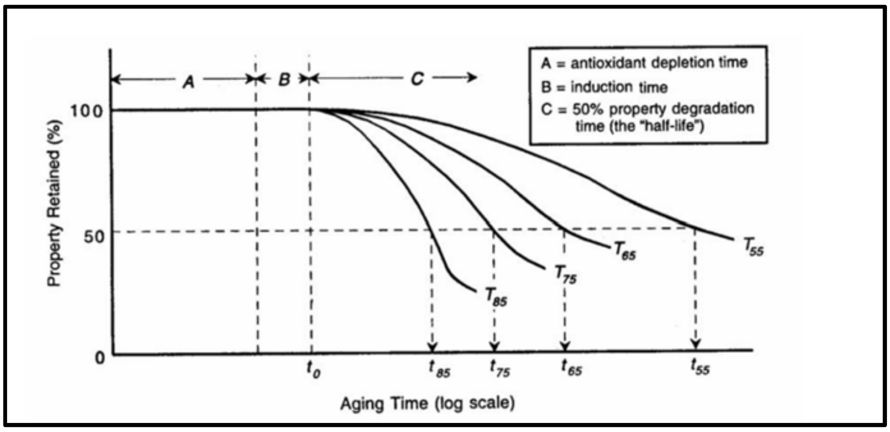
- GRI White Paper #6, 8 February 2011 - Geomembrane Lifetime Prediction: Unexposed and Exposed Conditions
- GRI Report #42, 3 January 2012 - Lifetime Prediction of Laboratory UV Exposed Geomembranes: Part I - Using a Correlation Factor

b. Aging and degradation

The paper titled Durability of HDPE geomembranes explains UV degradation (photo degradation) in detail and defines it as degradation induced by irradiation with UV or visible light. The consequences of long-term exposure include discolouration, surface cracks, brittleness and deterioration in mechanical properties. If further explains that the susceptibility of HDPE geomembranes to UV degradation is reduced by the use of carbon black or chemical-based light stabilizers that prevent the UV light from penetrating the polymer structure.

c. Service life prediction modelling

The oxidative degradation of HDPE geomembranes proceeds in three relatively distinct stages;



Stage A - Antioxidant Depletion Time
Stage B - Induction Time to the Onset of Degradation
Stage C - Time to Reach 50% Degradation (i.e., the Half-life)

Stage A - Antioxidant Depletion Time
The dual purposes of antioxidants are to;

(i) prevent polymer degradation during processing, and

(ii) prevent oxidation reactions from taking place during Stage A of service life, respectively. Obviously, there can only be a given amount of antioxidants in any formulation.

Hence, the rate of depletion of antioxidants is related to the type and amount of antioxidants, the service temperature, and the nature of the site-specific environment. See Hsuan and Koerner (1998) for additional details.
The Arrhenius equation is widely used to provide an estimate of the antioxidant depletion rate at a given temperature, different to those used in a testing program and is generally expressed as follows (Hsuan and Koerner 1998):

$$\boxed{\ln(s) = \ln(A) - \left(\frac{E_a}{R}\right)\left(\frac{1}{T}\right)}$$

Where;
s=antioxidant depletion rate ($month^{-1}$);
Ea=activation energy ($J \cdot mol^{-1}$);
R=universal gas constant (8.314 $J \cdot mol^{-1} \cdot K^{-1}$);
T=absolute temperature (K); and
A=constant often called a collision factor.

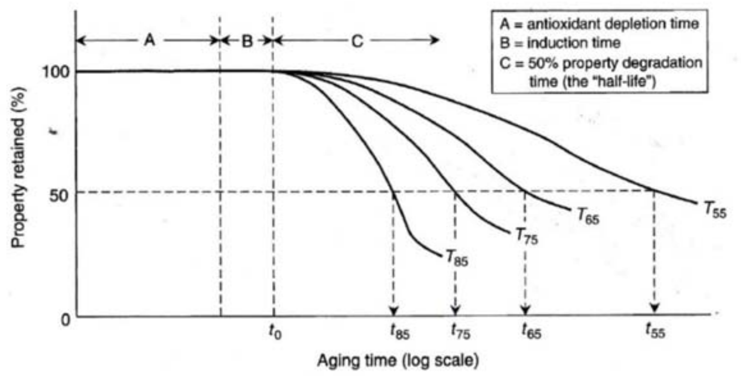
In using the Arrhenius equation for the purpose of extrapolation, it is assumed that:

1. the antioxidant depletion rate s is highly dependent on temperature;
2. the value of the collision factor A does not change with temperature; and
3. the activation energy Ea remains constant over the temperature range of interest (Koerner et al. 1992).

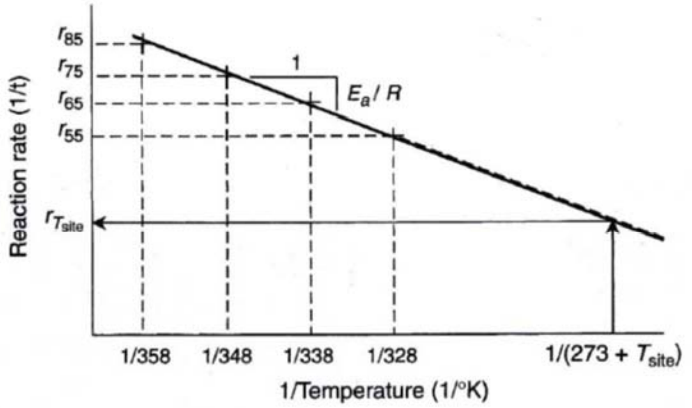
GRI Report 42 discusses the various methods used to predict service life of geomembranes. One such method discussed in the report is ***Time-Temperature-Superposition Followed by Arrhenius Modeling***.

It is generally accepted that the premier laboratory method for predicting the lifetime of polymeric materials is to replicate service conditions as closely as possible at several elevated temperatures, and then extrapolate the change in properties down to a site-specific temperature. The process is called time-temperature-superposition followed by Arrhenius modeling.
At GSI they have performed the requisite research for exposed HDPE geomembranes and other geomembranes was addressed by using accelerating laboratory weathering devices.
The method outlined in the report assumes that all polymer degradation mechanisms are proportionate to temperature with higher values being more aggressive and lower values less so, in a uniform (but not necessarily linear) manner and such method will be applied in evaluating the exposed conditions at Wesizwe Platinum Project.
The above-mentioned concept is embodied in the following two curves.

Arrhenius modeling for lifetime prediction using elevated temperatures.



(a) Incubated property behavior



(b) Arrhenius plot for half-life property

As shown in figure (a) above this procedure requires incubation at multiple temperatures which are all elevated well above the anticipated field service temperature.

The higher these incubation temperatures are set, the shorter will be the time to reach a 50% property retained from which one can then plot the Arrhenius graph as shown in Figure b above.
It is noted that one cannot use excessively high incubation temperatures since there may be degradation mechanisms occurring which do not take place at field service temperatures.

At GSI, they had limited this maximum incubation temperature to a relatively conservative value of 80°C. Since at least three different temperatures are required, the incubation temperature sequence being used at GSI is 50, 60 and 70°C. In the GR1 42 report, they have presented the 70°C incubation data to reach half-life in either strength or elongation as shown in the graphs above.

d. Wesizwe Platinum Project stormwater and returnwater dam service prediction

Stage A, that of antioxidant depletion for HDPE geomembranes as required in the GRIGM13 Specification, has been well established by GSI and others such as Sangram and Rowe (2004) in the paper Durability of HDPE geomembranes.
The following table below "Predicted Geomembrane Lifetimes Based on 50% Reduction of Strength and/or Elongation" (Table 6 GRI Report 42) will be used in the service life prediction of the geomembrane liner at Wesizwe Platinum Project in conjunction with literature that has been carried out and outlined in the paper Durability of HDPE geomembranes.

Table 6 - Predicted Geomembrane Lifetimes Based on 50% Reduction of Strength and/or Elongation

Geomembrane Type	Nominal thickness (mm)	Applicable Specification	50% reduction* (light hours)	Predicted lifetime (years)
fPP	1.00	GRI-GM18	40,000	33
HDPE	1.50	GRI-GM13	≃ 60,000	≃ 50
LLDPE	1.00	GRI-GM17	40,000	33
EPDM	1.14	GRI-GM21	37,000	30
PVC-N.A.	0.75	ASTM D7171	8,000	7**
PVC-Euro	2.50	proprietary	38,000	32

*Using ultraviolet fluorescent weathering devices at 70°C set at 350 nm wavelength for a daily cycling of 20 hours light and 4 hours dark with condensation; see Figures 8a through 8f.
**Only recommended for "buried applications".

Table 6 presents GSI's best-estimate of exposed lifetimes in hot climates of the geomembranes selected and as outlined in the report and one of them being a GM 13 HDPE geomembrane of 1.5mm thickness with the following mechanical properties.

(b) 1.5 mm thick high density polyethylene (HDPE)

Properties	Method	Specification Value (min.)	Tested Value
Thickness - (mm)	D5199	1.35	1.60
Density (g/cm^3)	D1505/D792	0.940	0.949
Tensile	D6693		
- yield stress (kN/m)	TypeIV	22	33
- break stress (kN/m)		40	53
- yield elongation (%)		12	17
- break elongation (%)		700	800
Tear resistance (N)	D1004	187	249
Puncture resistance (N)	D4833	480	667
Stress crack resistance	D5397	200	209
Carbon black content (%)	D1603	2-3	2.3
Carbon black dispersion	D5596	1 or 2	1

The above table was concluded based on a short method that makes use of known field failure times and when compared to the laboratory incubated failure times of the same but unexposed archived samples, allows for the establishment of a correlation factor. By having enough failures and incubation data from unexposed archived samples at those failure sites, a degree of reliance in this factor has been established. The advantage of this method as found by GSI is that only one incubation temperature is needed to obtain the correlation.

e. Project site conditions

As per extensive literature review that has been done on the subject and summarized above we are comfortable to make the followings statements with regards to the UV degradation of the specified HDPE geomembrane at Wesizwe Platinum Project on the RWD.

1. We firstly do not anticipate to have the side slopes of the RWD dam to be exposed to any aggressive leachate or water that can contribute to chemical degradation of the geomembrane over the 25 years required service life.
2. We assume and state that the site climatic maximum temperature the sides slopes of the facility with an exposed HDPE geomembrane 1.5mm or 2mm HDPE to be between 24 degrees and 34 degrees Celsius. This is the worst case scenario to which the HDPE geomembrane would be exposed to.
3. The most reliable and ideal method of determining the service life of the geomembranes would be from exposure under the actual field conditions, this is not presently feasible due to the length of time that would be required to obtain such results. As such testing would need to be run beyond 7 years.
4. Stage A, that of antioxidant depletion for HDPE geomembranes as reported by Rowe and Sangram for an HDPE immersed in water at a temperature of 35 degrees Celsius similar to what can be anticipated at Wesizwe Platinum Project was estimated at 35 years, we take this as the similar

condition to which we can apply our comparison to. We therefore anticipate the total service life of HDPE geomembrane at the stated temperature ranges to be considerably longer than that presented above because of the additional time in Stages B (induction time) and C (time required for the degradation of engineering properties of interest). Literature and research does not provided any case study in the quantification of stages B and C at this temperature, as this is ongoing research. We can however estimate the service life of the HDPE geomembrane at Wesizwe Platinum Project based on extrapolation of literature to be much greater than 35 years.

5. The time-temperature-superposition methodology of determining service life of the geomembrane is deemed valid based on current research work in the geosynthetics industry, and a summary of that work is highlighted in this report. This assumes that all polymer degradation mechanisms are proportionate to temperature with higher values being more aggressive and lower values less so, in a uniform (but not necessarily linear) manner. The incubations to which Table 6 was derived was done at 50, 60 and 70 degrees C which is below the range of the service temperature anticipated at Wesizwe Platinum Project and thus justifies its stated service life extrapolation.
6. The laboratory experiments conducted to infer the depletion rates of service life of the geomembrane, assumed the leachate strength was essentially constant in strength and application because it was regularly replaced and so the exposure was essentially constant over the testing period (Rowe and Sangram). At Wesizwe Platinum Project, there is no leachate to be considered as the runoff and water in the returnwater dam can be considered to be close to inert. And therefore the rate of antioxidant depletion can be expected to decrease however much lower than that measured in the laboratory by Rowe and Sangram.
7. Different exposure conditions of the geomembrane to site temperatures is likely to underestimate the depletion time of the actual site conditions, as these cannot be stimulated under any laboratory condition. However it is anticipated that the rate of antioxidant depletion will decrease and be lower if a higher geomembrane is used both in thickness and stress crack resistance values. This geomembrane will be higher in mechanical properties as well as UV resistance as per antioxidant package that is used in its manufacturing process. We therefore propose a 2mm HDPE thickness geomembrane for the returnwater dam with the following values as per attached table in the report annexure.

f. <u>Conclusion</u>

The international research work leads to the evaluation of service life times of > 100 years for basal liner systems with 2.5 mm thick HDPE in landfill applications at 40°C and of > 100 years (Werner Müller, HDPE-geomembranes in Geotechnics German Federal Agency for Material Testing). In applications with moderate temperatures (20- 30°) research led to estimated service life of > 100 years for 1.5 mm thick liners (Robert M. Koerner, designing with geosynthetics). The specific research of effects of acidic mine drainage does indicate a service life of 136 years in a tailings impoundment (Gulec, Edil & Benson). The extrapolation of service life of these international research works as above is based on Arrhenius-modelling as described in ISO 11346.
The service life of exposed geomembranes in similar conditions to Wesizwe Platinum Project can be estimated to be over 35 years. This is as per estimation and summary of various industry professions that have done work in this area and reference reports attached to annexure of this report.

The key findings of the work reported by various research fellows in this field conclude that the service lives of the HDPE geomembranes are essentially controlled by the antioxidants in the material and the service temperature. However, we acknowledge that there is the potential for debate regarding the property (s) to be assessed with respect to the degree of polymer breakdown and the level used as the failure level.
On our part we commit to having the initial mechanical properties of the geomembrane assessed and thereafter cut out samples in the slope exposed areas of the facilities to have them assessed every five years over the 25 years site life of the facilities.
This report has provided the closest simulation of geomembrane exposure conditions at the project site in relation to laboratory simulations done on a landfills and other mine containment facilities as published to date, and as a consequence the estimated antioxidant depletion times are expected to provide the most realistic estimate of the likely depletion time of antioxidants at Wesizwe Platinum Project.

We are aware that additional testing is required in general to allow similar estimates of Stages B and C of the service life. It is noted that the service life of the geomembrane at Wesizwe Platinum Project will depend on the resin and antioxidant package used and may vary from one geomembrane to another, we intend to use a 1.5mm and 2mm HDPE geomembrane with the specified mechanical properties included in the attached annexures. And if required this additional information on resin and antioxidant package can be provided to further motivate the estimates made above.

5. Construction Quality Assurance

As per the National Norms and Standards for Disposal of Waste to Landfill, Regulation 636, dated 23 August 2013. In particular clauses of the Regulation R636 section 3 items (2) (g), every waste disposal facility being designed and permitted should include a construction quality assurance (CQA) plan. This plan describes the tasks involved with the construction quality assurance (CQA) for the waste containment facilities. **Refer to attached document.**

In summary the purpose of the CQA Plan is to address the CQA procedures and monitoring requirements for construction of the project. The CQA Plan is intended to:

(i) define the responsibilities of parties involved with the construction of facilities; (ii) provide guidance in the proper construction of the major components of the project; (iii) establish testing protocols; (iv) establish guidelines for construction documentation; and (v) provide the means for assuring that the project is constructed in conformance to the Technical Specifications, permit conditions, applicable regulatory requirements, and Construction Drawings.

Knight Piesold will under strict quality assurance provide full time construction supervision during the construction phase of the waste disposal facility and associated infrastructure to ensure the facility is constructed as per design and perform intent in line with all activities outlined in the project specifications.

6. Technical Specifications

The publications below form part of the specifications relating to the liner design of the Tailings dam and returnwater dam;

- Project specifications related the construction of the waste containment facilities.

- South African National Standard (SANS 10409: 2005 Edition 1) - Design, selection and installation of geomembranes
- South African National Standard (SANS 1526: 2015 Edition 3) - Thermoplastics sheeting for use as a geomembrane
- American Society for Testing and Materials (ASTM)
    - D 1004 Test Method for Initial Tear Resistance of Plastic Film and Sheeting
    - D 1238 Standard Test Method for Flow Rates of Thermoplastics by Extrusion Plastometer
    - D 1505 Test Method for Density of Plastics by the Density-Gradient Technique
    - D 1603 Test Method for Carbon Black in Olefin Plastics
    - D 3895 Standard Test Method for Oxidative-Induction Time of Polyolefins by Differential Scanning Calorimetry
    - D 4218 Standard Test Method for Determination of Carbon Black in Polyethylene Compounds
    - D 4833 Standard Test Method for Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products
    - D 5199 Standard Test Method for Measuring Nominal Thickness of Geotextiles and Geomembranes
    - D 5397 Standard Test Method for Evaluation of Stress Crack Resistance of Polyolefin Geomembranes Using Notched Constant Tensile Load Test
    - D 5596 Standard Test Method for Microscopic Evaluation of the Dispersion of Carbon Black in Polyolefin Geosynthetics
    - D 5994 Standard Test Method for Measuring Core Thickness of Textured Geomembranes
    - D 6392 Standard Test Method for Determining the Integrity of Nonreinforced Geomembrane Seams Produced Using Thermo-Fusion Methods
    - D 6693 Standard Test Method for Determining Tensile Properties of Nonreinforced Polyethylene and Nonreinforced Flexible Polypropylene Geomembranes
    - D 7240 Standard Practice for Leak Location using Geomembranes with an Insulating Layer in Intimate Contact with a Conductive Layer via Electrical Capacitance Technique (Conductive Geomembrane Spark Test).

GRI Standards:

- "Test Methods, Test Properties and Testing Frequency for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes, GRI Test Method GM13". Latest edition at dated on tender enquiry.

- "Seam Strength and Related Properties of Thermally Bonded Polyolefin Geomembranes, GRI Test Method GM19." Latest edition at dated on tender enquiry.

7. Specifications for hdpe Geomembranes for wesizwe tailings dam and returnwater dam

The geomembrane shall be of high quality formulation polyethylene material, resistant to ultraviolet rays, manufactured of new, first-quality products, containing no plasticizers, fillers or extenders, and designed and manufactured specifically for the purpose of intended application and use.

Tailings dam geomembrane specification

The 1.5mm smooth HDPE geomembrane shall comply with the latest revision GRI-GM13, however with the following variations to GRI-GM13 in order to further improve the quality and longevity of the geomembrane for project specific requirements.

- Thickness to be Minimum not Nominal
- Standard OIT to be greater than 150 minutes instead of 100minutes
- High Pressure OIT to be greater than 500 minutes instead of 400minutes

Smooth geomembrane shall have good appearance qualities, and shall be free from such defects that would affect the specified properties.

Returnwater dam geomembrane specification.

For a 2mm thickness HDPE smooth geomembrane to be used on the returnwater dam; Shall be as per the relevant test methods in GM 13.

- Thickness to be Minimum not Nominal
- Standard OIT to be greater than 150 minutes instead of 100minutes
- High Pressure OIT to be greater than 500 minutes instead of 400minutes

Tested Property	Test Method	Frequency	Value
			2.00 mm
Thickness, mm Lowest individual reading	ASTM D 5199	every roll	2.00 1.80
Density, g/cm^3	ASTM D 1505	90,000 kg	0.94
Tensile Properties (each direction) Strength at Break, N/mm Strength at Yield, N/mm Elongation at Break, % Elongation at Yield, %	ASTM D 6693, Type IV Dumbbell, 50 mm/min G.L. 50 mm G.L. 33 mm	9,000 kg	 57 31 800 13
Tear Resistance, N	ASTM D 1004	20,000 kg	257
Puncture Resistance, N	ASTM D 4833	20,000 kg	711
Multi-axial Break Resistance, %	ASTM D 5617	per formulation	30
Carbon Black Content, % (Range)	ASTM D 1603*/4218	9,000 kg	2.0 - 3.0
Carbon Black Dispersion	ASTM D 5596	20,000 kg	Note[(1)]
Notched Constant Tensile Load, hr	ASTM D 5397, Appendix	90,000 kg	1,000
Oxidative Induction Time, mins	ASTM D 3895, 200°C; O_2, 1 atm	90,000 kg	>160
High Pressure Oxidative Induction Time, mins	ASTM D 5885, 150°C; O_2, 3.4 MPa	per formulation	>500
Oven aging at 85°C High Pressure OIT (min. avg.) - % retained after 90 days	ASTM D 5721 ASTM D 5885	per formulation	80
UV Resistance High Pressure OIT (min. avg.) - % retained after 1,600 hours	GM 11 ASTM D 5885	per formulation	80

8. Disclaimer

This report has been prepared from technical literature review of all the papers quoted in this report. Nothing mentioned in this paper is new work carried out by the author of this report.

APPENDIX D

CONSTRUCTION QUALITY ASSURANCE (CQA) PLAN

Bakubung Minerals (Pty) Ltd

WESIZWE PLATINUM MINE
DWS COMPLIANCE
CONSTRUCTION QUALITY ASSURANCE PLAN FOR THE CONSTRUCTION OF THE DWS COMPLIANCE

PREPARED FOR:

Bakubung Minerals (Pty) Ltd.

PREPARED BY:

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Report Number: 301-00050/02/R1

Rev 0
Date: January 2016

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Knight Piésold CONSULTING

WESIZWE PLATINUM MINE

DWS COMPLIANCE

CONSTRUCTION QUALITY ASSURANCE PLAN FOR THE CONSTRUCTION OF THE DWS COMPLIANCE

REPORT 301-0050/02/R1

REVISION 0

JANUARY 2016

Prepared by : ______________________________

(Name)

(Status)

Reviewed by : ______________________________

(Name)

(Status)

WESIZWE PLATINUM MINE

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1. INTRODUCTION

1.1 TERMS OF REFERENCE

Knight Piésold (Pty) Ltd (KP) has prepared this Construction Quality Assurance (CQA) for the construction of Wesizwe Platinum TSF. The TSF has been design to conform to the latest regulations regarding waste classification and barrier systems by the Department of Water and Sanitation (DWS).

1.2 PURPOSE AND SCOPE OF THE CONSTRUCTION QUALITY ASSURANCE PLAN

The purpose of the CQA Plan is to address the CQA procedures and monitoring requirements for construction of the project. The CQA Plan is intended to:

(i) define the responsibilities of parties involved with the construction;
(ii) provide guidance in the proper construction of the major components of the project;
(iii) establish testing protocols;
(iv) establish guidelines for construction documentation; and

(v) provide the means for assuring that the project is constructed in conformance to the *Technical Specifications*, permit conditions, applicable regulatory requirements, and *Construction Drawings*.

This CQA Plan addresses the soils and geosynthetic components of the barrier system for the project. The soils, geosynthetic, and appurtenant components include prepared subgrade, geomembrane and drainage aggregate. It should be emphasized that care and documentation are required in the placement of aggregate, and in the production and installation of the geosynthetic materials installed during construction. This CQA Plan delineates procedures to be followed for monitoring construction utilizing these materials.

The CQA monitoring activities associated with the selection, evaluation, and placement of drainage aggregate are included in the scope of this plan. The CQA protocols applicable to manufacturing, shipping, handling, and installing all geosynthetic materials are also included. However, this CQA Plan does not specifically address either installation specifications or specification of soils and geosynthetic materials as these requirements are addressed in the *Technical Specifications*.

1.3 REFERENCES

The CQA Plan includes references to test procedures in the latest editions of South African National Standards specification and the American Society for Testing and Materials (ASTM).

- GRI GM 13 latest edition specs
- SANS 1526
- SANS 10409
- SANS 1200 (Liner bedding tolerances for earthworks preparation to receive liner)

1.4 ORGANIZATION OF THE CONSTRUCTION QUALITY ASSURANCE PLAN

The remainder of the CQA Plan is organized as follows:

- Section 2 presents definitions relating to CQA;
- Section 3 describes the CQA personnel organization and duties;

- Section 4 describes site and project control requirements;
- Section 5 presents CQA documentation;
- Section 6 presents CQA of earthworks;
- Section 7 presents CQA of the drainage aggregates;
- Section 8 presents CQA of the pipe and fittings;
- Section 9 presents CQA of the geomembrane;
- Section 10 presents CQA of the geotextile;

2. DEFINITIONS RELATING TO CQA

This CQA Plan is devoted to Construction Quality Assurance. In the context of this document, Construction Quality Assurance and Construction Quality Control are defined as follows:

Construction Quality Assurance (CQA) - A planned and systematic pattern of means and actions designed to assure adequate confidence that materials and/or services meet contractual and regulatory requirements and will perform satisfactorily in service. CQA refers to means and actions employed by the CQA Consultant to assure conformity of the project "Work" with this CQA Plan, the *Drawings*, and the *Technical Specifications*. CQA testing of aggregate, pipe, and geosynthetic components is provided by the CQA Consultant.

Construction Quality Control (CQC) - Actions which provide a means to measure and regulate the characteristics of an item or service in relation to contractual and regulatory requirements. Construction Quality Control refers to those actions taken by the Contractor, Manufacturer, or Geosynthetic Installer to verify that the materials and the workmanship meet the requirements of this CQA Plan, the *Drawings*, and the *Technical Specifications*. In the case of the geosynthetic components and piping of the Work, CQC is provided by the Manufacturer, Geosynthetic Installer, and Contractor.

2.1 OWNER

The Owner of this project is Wesizwe Platinum Mine.

2.2 CONSTRUCTION MANAGER

Responsibilities

The Construction Manager is responsible for managing the construction and implementation of the Drawings, and Technical Specifications for the project work. The Construction Manager is selected/appointed by the Owner.

2.3 ENGINEER

Responsibilities

The Engineer is responsible for the design, Drawings, and Technical Specifications for the project work. In this CQA Plan, the term “Engineer” refers to Knight Piésold.

Qualifications

The Professional Engineer shall be a qualified engineer, registered with ECSA. The Engineer should have expertise, which demonstrates significant familiarity with piping, geosynthetics and soils, as appropriate, including design and construction experience related to liner systems.