ESKOM HOLDINGS LTD

KRIEL POWER STATION
ASH DAM 4 – SITE 10
CONCEPT DESIGN UPDATE

Report No.: JW044/16/E821 – Rev 0

April 2016

Jones & Wagener
Engineering & Environmental Consultants
59 Bevan Road PO Box 1434 Rivonia 2128 South Africa
Tel: 00 27 (0)11 519 0200 Fax: 00 27 (0)11 519 0201 email: post@jaws.co.za

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SYNOPSIS

Introduction

An investigation carried out by Jones & Wagener in 2006 (report JW127/06/A407) showed that the existing ash dams at Kriel had very limited capacity remaining. The study concluded that the most feasible option for providing additional deposition capacity was to construct a new ash dam, referred to as Dam 4.

Site Location and Constraints

Various alternative sites for Dam 4 have been identified within a 10km radius of the power station. The shortlist of preferred sites included Site 10 and Site 16N as described in the site selection report (J&W71/10/A407). Site 10 is located south of the existing ash complex, partly overlying the backfilled Pit 1 of Kriel Colliery. Site 16N is located on the farms Roodebloem and Roodepoort, approximately 12km north-east of the power station.

Site 10 is the selected site for the concept design of Dam 4 for this report. The boundaries of the site is defined by the existing ash dams north, Cut 2 void south, property boundary east and access road west. The Cut 2 void is seen as a boundary, because the cut is deep and the earthworks and liner works in this area will be costly.

Construction over the backfilled pits is unavoidable and the impact that differential settlement will have on the different design aspects of the dam needs to be considered carefully.

Ash Tonnages

Ash Dam 4 (AD4) was designed to accommodate average monthly tonnages of ash of 220 000 tpm. The total ash stream to be accommodated on the existing and new dams from 2016 to 2045 is 71.5Mtons.

Life Assessment

An updated life assessment was conducted prior to the concept design to establish the time that is available between switch over from the existing to the new dam. The existing Ash Dams will reach a limiting Rate of Rise of 3.0 m/year by end June 2021. At this stage Ash Dam 4 will have to be commissioned. An ash density of 1 t/m3 was assumed for the life assessment. AD4 will be built in three separate phases, referred to as Ash Dams 4.1, 4.2 and 4.3 (AD4.1, AD4.2, AD4.3), respectively. A period of 2 years was allowed for between the construction of each phase in order to defer large immediate capital costs.

Layout and operations

It was assumed that deposition on the existing dams will continue for 4 years after the commissioning of the first phase of Ash Dam 4, in order to minimise the footprint and final height of the new dams. The maximum heights of the final dams are 64m, 61m and 62m for AD4.1, AD4.2 and AD4.3, respectively.

AD4.1 and AD4.2 will be situated on natural ground, while AD4.3 that will overlay the backfilled pit. AD4.3 will only be required in 2023 thereby postponing development on backfill to the maximum.

AD4 will be constructed in the upstream direction using the daywall paddocking method and slurry deposited using a ring main delivery system, which consists of a ring main delivery line with multiple deposition points around the dam. Currently Pulverised Fuel Ash (PFA) is deposited at a single point (via conveyor) where it is slurried or sluiced and allowed to gravitate along the daywall, whereas the Boiler Bottom Ash (BBA) is pumped and deposited by open-ended deliveries at selected positions directly into the basin of the dam.

Waste Classification and Liner Requirements

The waste of three other Eskom dams were classified according to the Waste Act Regulations (GN634 and 635, 2013) as a Type 3 waste requiring a Class C liner, in accordance with GN636 (2013). It was assumed that the Kriel Power Station ash will have the same classification, but the classification will need to be confirmed prior to the basic design.

Liner System

A Class C liner (GN636, 2013) will be used as the lining system for the new ash dam. The liner comprises of a subsoil drain overlain by 300mm of compacted clay and a 1.5mm thick geomembrane. A drainage layer of BBA will be provided on top of the liner to promote drainage.

A liner will not be installed where the new dam abuts against the side slopes of the existing dams. A drainage layer of BBA, however, will be placed against the side slopes to form a preferential flow path at the interface between the new and existing dams. A Class C liner (GN636, 2013) with a drainage collection system will be provided on the larger benches of the existing dams. The water will then report to the under drainage system of AD4 at the toe of the existing dams.

Deposition

Only conventional wet slurry deposition was considered as part of this concept study.

Pre-deposition Works

Significant reshaping of the site, especially within the void of Cut 1, will be required to provide adequate drainage on top of the liner.

The height of the starter walls is a function of the rate of rise and as a result the amount of ash tonnages reporting to the dam. An acceptable rate of rise of 3.5m/year was assumed for operability as opposed to 3m/year for stability. If all the ash is diverted to the new ash dam at once, a substantial starter wall will be required. It is therefore recommended that deposition initially be split between the new and existing dams, to reduce the rate of rise and the ash tonnages reporting to each dam.

A maximum starter wall height of 11m on AD4.1, 11m on AD4.2 and 17m for AD4.3 will be needed. The starter wall height of AD4.2 was determined assuming it only receives 25% of the slurry stream upon commissioning until the RoR reaches 3.5m/year. In the same manner the height of the starter walls for AD4.1 and AD4.3 was determined assuming a slurry stream of 30% and 35%, respectively, upon commissioning until their RoR reaches 3.5m/year.

Decant System

Storm and supernatant water are to be decanted off the basin of the dam by means of a gravity penstock. A decant system will be required for each phase of development to be able to decant off the water.

Due to differential settlement concerns, the main/permanent penstock for each compartment of AD4 will be placed on either only natural ground (AD4.1 & 4.2) or only backfill (AD4.3).

In general the maximum height a penstock can reach before crushing and down-drag becomes a problem is approximately 25-30m and elevated penstocks will be required for AD4 in the future. This is however not believed to be a problem as the ash due to its cementing properties provides a strong stable base for the elevated penstocks.

Return Water handling

Since AD4.1 will be constructed over the existing Return Water Dam (RWD), a new Ash Water Return (AWR) dam will be required. The new AWR was sized to accommodate the 1:50 year, 24 hour storm for AD4 (i.e. the existing dams are assumed to be rehabilitated when all three phases of AD4 have been commissioned). A 330 000m^3/310Mℓ AWR will be required for this scenario.

The new proposed AWR was positioned between the East Wing Dam of the existing RWD and the pit boundary. The new AWR will be located at the toe of AD4.1. This site was selected as it is situated on natural ground, as opposed to backfill material, and because it is in the direct vicinity of the existing pumping station.

An additional dam is required for the run-off, leachate seepage, decant and process water from AD4.2, due to the low point of AD4.2 being above the new proposed AWR. The additional dam, referred to as the Transfer Dam, will require a pumping system in order to pump water into the AWR. The transfer dam is nominally sized to fit into the available space between AD4.3 and the backfill. The transfer dam should furthermore pump process and storm water to the AWR dam from AD4.2 and later on AD4.3.

A pump sump will be provided at the toe of AD4.1 to the West of the new proposed AWR, as the natural ground elevations and the underdrainage outlet elevations are below the full supply level of the new AWR. Seepage water will need to be pumped from the sump into the solution trench running into the AWR.

A Class C liner (GN636, 2013) is specified for the AWR and Transfer Dam with a concrete protection layer to help reduce uplift and possible damage to the geomembrane when the dams are cleaned. On the side slopes a reinforced geogrid is provided under the concrete filled geocells.

Closure Requirements

Concurrent rehabilitation of the outer slopes of the ash dam is recommended to aid in dust suppression. The capping of the ash consists of a 300mm topsoil layer which will be vegetated.

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Jones & Wagener
Engineering & Environmental Consultants
59 Bevan Road PO Box 1434 Rivonia 2128 South Africa
Tel: 00 27 (0)11 519 0200 Fax: 00 27 (0)11 519 0201 email: post@jaws.co.za

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ASH DAM 4
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1. INTRODUCTION

1.1 Background

The existing ash dam complex is located 1,5km south of the power station, comprising three abutting ash dams. The complex is bounded on the south and west by rehabilitated open cast coal mining areas of the Kriel Colliery.

An investigation carried out by Jones & Wagener in 2006 (report JW127/06/A407) showed that the existing ash dams at Kriel had very limited capacity remaining. The study concluded that the most feasible option for providing additional deposition capacity was to construct a new ash dam, referred to as Ash Dam 4 (AD4).

Various alternative sites for AD4 have been identified within a 10km radius of the power station. The shortlist of preferred sites included Site 10 and Site 16N as described in the site selection report (J&W71/10/A407). Site 10 is located south and east of the existing ash complex, partly overlying the backfilled Pit 1 of Kriel Colliery. Site 16N is located on the farms Roodebloem and Roodepoort, approximately 12km north-east of the power station.

An extensive geotechnical investigation was undertaken by J&W during 2010/11, resulting in report JW196/11/C779, issued in May 2012. The report was focused on establishing the founding conditions of Site 10 (i.e. the preferred site based on distance from the plant) and evaluating the depth of the backfilled pit. The report furthermore also recommended a large scale Monitoring Trial Embankment (MTE) be constructed to calibrate the geotechnical design parameters derived from the investigation. The purpose of the MTE is also to verify, by direct measurement, whether the new ash dam can be successfully constructed with the incorporation of a liner, as required by the authorities, over the backfilled pit.

The specification for the MTE has been completed and approval for its construction is still required. The results of the MTE with regard to the expected settlement will be used to assess the lining system requirements and best suited deposition methods of AD4.

However, the preferred deposition methodology based on cost, operability, environmental impact and adaptability to existing conditions and operations on site needs to be established first. This investigation was carried out in 2013 (JW164/13/D379) and concluded that wet disposal was preferred over dry disposal.

Eskom has since then indicated that an initial development on the natural ground portions of Site 10 is preferred to allow more time to investigate ash deposition on backfill and possibly to avoid this altogether. This refinement is the subject of the concept design.

1.2 Previous Investigations

Previous investigations done for Kriel Power Station specifically relating to this report includes:-

- 2006: Initial Concept Study – J&W Report no. JW127/06/A407;
- 2010: Site Selection Inputs – J&W Report no. JW71/10/A407;
- 2011: Geotechnical Site Investigation – J&W Report no. JW196/11/C779;
- 2013: Concept Study (Wet vs. Dry) – J&W Report no. JW164/13/D379;
- 2014: Step-In & Go Higher Geotechnical Investigation and Stability Assessment – Preliminary Report – J&W Report no. JW129/15/F015.

1.3 Purpose of Report

The purpose of this report is to update the ash dam concept design (Report no. JW044/16/E821), taking into consideration wet deposition on natural ground first followed by development on backfill. This report only considers Site 10 and a life cycle cost is calculated based on the concept design.

1.4 Definitions and Abbreviations

1.4.1 Project

J&W	**J**ones **&** **W**agener (Pty) Ltd
KPS	**K**riel **P**ower **S**tation
DWAF	**D**epartment of **W**ater **A**ffairs and **F**orestry - Replaced by **DWA** and **DEA**
DWA	**D**epartment: **W**ater **A**ffairs
DEA	**D**epartment: **E**nvironmental **A**ffairs
GRI	**G**eosynthetic **R**esearch **I**nstitute

1.4.2 Technical

Survey & Coordinates:

mamsl	**m**etres **a**bove **m**ean **s**ea **l**evel
NGL	**N**atural **G**round **L**evel

Design:

ADx	**A**sh **D**am (**x** specifies the ash dam under consideration)
AWR	**A**sh **W**ater **R**eturn
BBA	**B**oiler **B**ottom **A**sh
FoS	**F**actor **o**f **S**afety
GLF	**G**enerating **L**oad **F**actor
HDPE	**H**igh **D**ensity **P**oly**E**thylene
LCC	**L**ife **C**ycle **C**ost
LoPP	**L**ife **o**f **P**ower **P**lant
PFA	**P**ulverised **F**uel **A**sh
PSD	**P**article **S**ize **D**istribution
MAP	**M**ean **A**nnual **P**recipitation
MAE	**M**ean **A**nnual **E**vaporation
MCR	**M**aximum **C**ontinuous **R**ating
MTE	**M**onitoring **T**rial **E**mbankment

NB	**N**ominal **B**ore
RoM	**R**un **o**f **M**ine
RoR	**R**ate **o**f **R**ise
RWD	**R**eturn **W**ater **D**am
ZoI	**Z**one **o**f **I**nfluence

Symbols

ρ	Density, kg/m^3
γ	Unit weight, kN/m^3
ϕ'	Effective stress angle of internal friction, degrees
c'	Cohesion in terms of effective stresses, kPa

1.4.3 Definitions

Coal ash is a by-product of combustion and collectively refers to:-

- **Pulverised Fuel Ash (PFA):** PFA rises with the furnace gasses and is collected by electrostatic precipitators in, or, before the stacks or chimneys of the power station. The ash is therefore captured dry and is commonly referred to as fly ash. The ash can be conditioned by adding small amounts of moisture to ease handling by mechanical means and to reduce dust before it is transported to the deposition facility usually by troughed conveyors. PFA constitutes approximately 80% to 90% of the coal ash.

- **Boiler Bottom Ash (BBA):** BBA is the larger ash particles that cannot rise and falls down into a pan below the boiler where it is quenched in water. The ash is therefore captured wet. The ash and water forming a slurry can be thickened to an optimal density before it is transported to site by means of pumping. BBA constitutes approximately 10-20% of the coal ash.

1.5 Reference Documents

The following references were consulted in preparing the scope of the investigation:

(i) **Adamson, PT.** 1981. *Southern African Storm Rainfall.* Department of Environmental Affairs, Directorate of Water Affairs. Technical Report No. TR102.

(ii) **DWAF.** 1998. *Minimum Requirements for Waste Disposal by Landfill.*

(iii) **ECsoft.** 2009. *Kriel Power Station: Ash Dam Pump Station Design Report*. Report No. 10559.

(iv) **Government Notice 432**. 2011. Department of Environmental Affairs, National Environmental Management: Waste Act, Act 59 of 2008.

(v) **Government Notice 634**. 2013. Department of Environmental Affairs, National Environmental Management: Waste Act, Act 59 of 2008: Section 69(1)(a), (b), (g), (h), (m), (q), (r), (s), (dd) and (ee): *Waste Classification and Management Regulations.*

(vi) **Government Notice 635**. 2013. Department of Environmental Affairs, National Environmental Management: Waste Act, Act 59 of 2008: Section 7(1)(c): *National Norms and standards for the assessment of waste for landfill disposal.*

(vii) **Government Notice 636**. 2013. Department of Environmental Affairs, National Environmental Management: Waste Act, Act 59 of 2008: Section 7(1)(c): *National Norms and standards for disposal of waste to landfill.*

(viii) **Government Notice 704**. 1999. National Water Act, Act 36 of 1998.

(ix) **GRI.** 2005. GRI Report #30. *Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces*.

(x) **Robinson, B. 2005.** *Tailings Disposal Course*. Fraser Alexander Tailings.

(xi) **Slope/W.** 2012. GeoStudio® 2012. GeoSlope International (Ltd). Canada.

(xii) **Sherard, J.L. & Dunnigan, L.P.** 1989. *Critical Filters for impervious soils.* ASCE Journal of Geotechnical Engineering.

(xiii) **Warman International Ltd.** 2000. *Warman Slurry Pumping Handbook*.Warman International Ltd.

(xiv) **J&W Report no. JW127/06/A407**. Initial Concept Study. 2006

(xv) **J&W Report no. JW71/10/A407**. Site Selection Inputs. 2010

(xvi) **J&W Report no. JW196/11/C779**. Geotechnical Site Investigation. 2011

(xvii) J**&W Report no. JW164/13/D379**. Concept Study (Wet vs. Dry). 2013

(xviii) **J&W Report no. JW129/15/F015**. Step-In & Go Higher Geotechnical Investigation and Stability Assessment – Preliminary Report. 2014

(xix) **323-09**. Eskom economic Evaluation Parameters. July 2015

(xx) **Paterson & Cooke**. Large Scale Loop and Pump Tests. Document ESK-1559 R01 Rev 1. 2013

2. SITE DESCRIPTION

2.1 Site Locality

The site selected for the concept design of the new ash dam in this report is Site 10, which is located west of the town Kriel in the Mpumalanga Province. The site is south east of the Kriel Power Station and is the closest site to the plant. Matla Ash dam is located south west of Site 10.

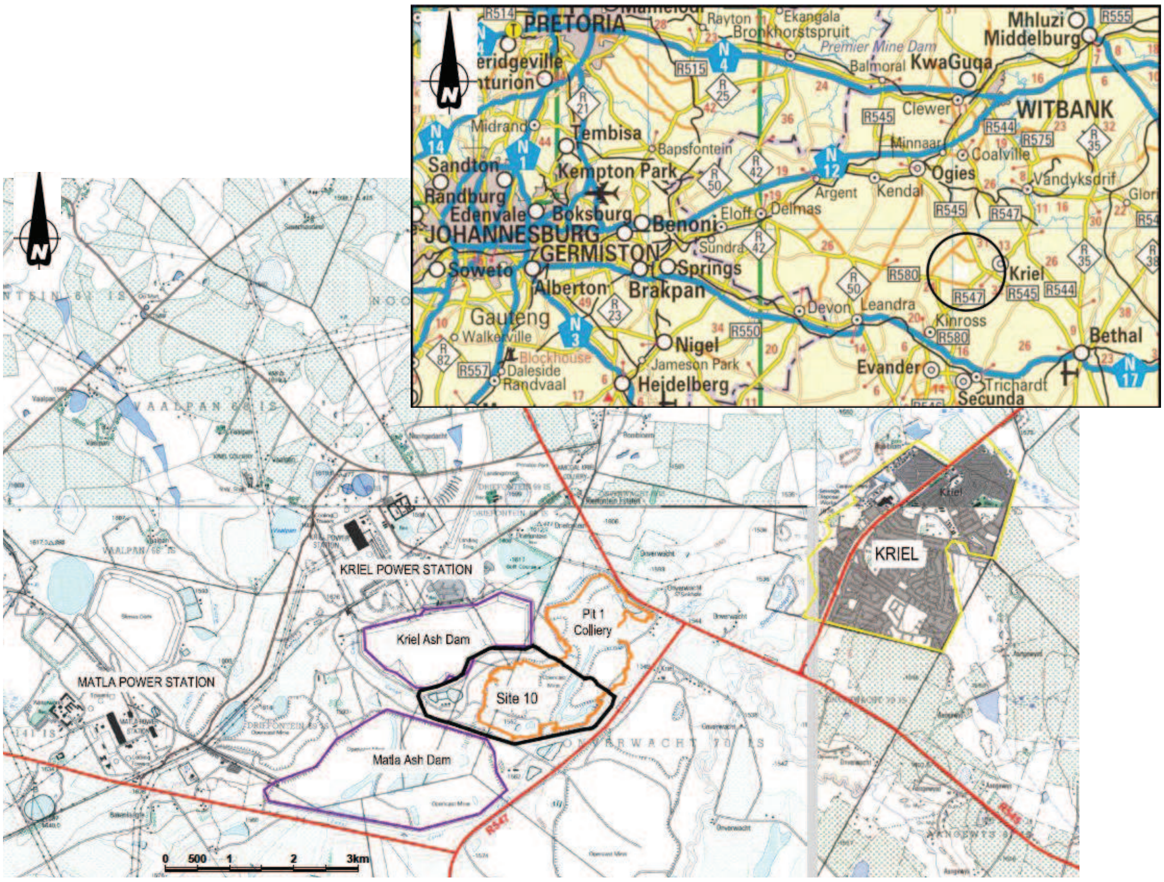


Figure 2-1: Locality Plan

2.2 Regional Geology

The following section is an extract from the geotechnical investigation report (J&W Report no. JW196/11/C779-Rev.0) for completeness.

The regional geology is dominated by the Karoo Supergroup sediments and Kriel Coalfield. Topographically the region is undulating with the main Dwars-in-die-Weg/Steenkoolspruit valley more or less bisecting the coalfield. The few extensive flat areas within the coalfield are largely confined to the crests of ridges and to the floor of the main valley. Apart from a prominent scarp on the north of the Steenkoolspruit and a few pans and marsh areas, there are no distinctive topographic features. Prominent hills, composed of pre-Karoo felsite and granites, occur immediately to the north of the area. Data from some 500 exploratory boreholes and surface observations have provided a detailed knowledge of the stratigraphy and geology of the area. The coalfield is underlain by a thin sequence of Dwyka and Middle Ecca strata resting on an undulating floor composed of felsite, granite and diabase associated with the Bushveld Complex and older Vaalian formations. The Karoo strata have been scoured in the north by the Steenkoolspruit. Where prominent pre-Karoo hills are present, the lower strata of the Karoo System, including the main coal seams, have not deposited at all.

The stratigraphic units are virtually identical to those found in the Witbank Coalfield, which is situated immediately to the north of the area. The stratigraphy throughout the Kriel Coalfield is remarkably uniform. The topography of the No 4 Seam is generally flat to gently undulating. Where a dolerite sill has cut through, the seam is faulted and in some areas slightly tilted along the margins of the fault. The coal is generally burnt in these fault-margin areas. There are two main dolerite sills in the coalfield, the thickest occurring in the south-west and a very extensive thinner sill, 1.5 to 15m thick. The latter underlies the No. 4 Seam in almost half the coalfield, and overlies the seam in the remainder. In some areas, the sill has cut through the seam in dome-form fashion. The vertical throw on the fault caused by these intrusions varies from 6m to 25m. Very few dykes have been located on the surface, probably because they are not very numerous and because the generally thick cover of soil renders them difficult to detect either on aerial photographs or by magnetometer surveys.

The geology around Kriel Power Station and Site 10 is shown in Figure 2-2.

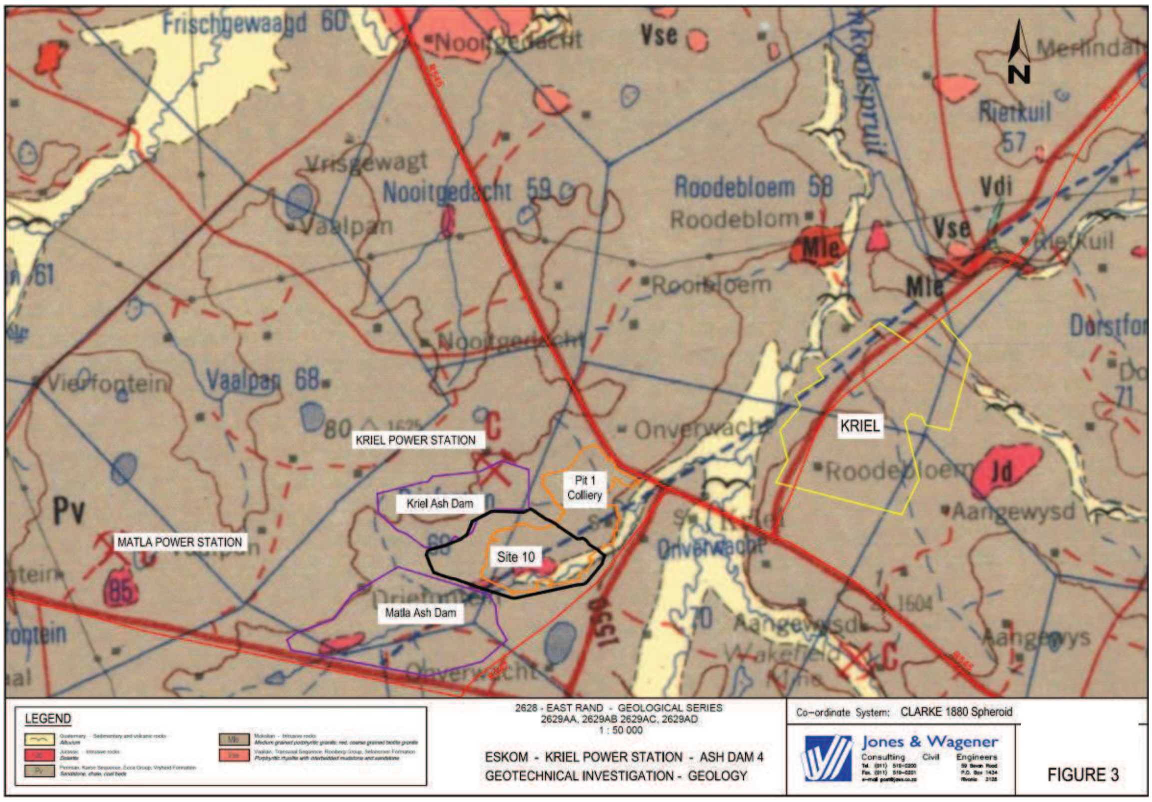


Figure 2-2: Regional Geology

2.3 Site Geology

The No.4 Coal Seam

The No. 4 Seam, mined in Pit 1, occurs through most of the area and is unworkable only where it has been affected by weathering or burnt by dolerite intrusives. The seam varies in thickness from 3m to 6m, but is remarkably consistent at about 4.9m in thickness. The workable coal is contained in the lower two-thirds of the seam (usually below a marker shale), except in a few isolated areas where a thick parting occurs near the seam floor. In these exceptional cases, the workable section extends down to the top of the parting. The coal is generally dull and dull-lustrous, with a few bright bands. The top third of the seam is composed of inferior coal/carbonaceous shale.

Kriel Pit 1

Kriel Pit 1 (Block 4) was mined using a typical dragline long wall opencast operation. The coal was mined from a long, narrow and parallel advancing box cut. The sequence of mining was as follows:

- Strip and stockpile the surface overburden soils that can be used for other purposes including haul road construction, rehabilitation, top-soiling etc.
- Drill and blast the overburden rock above the coal seam.
- Strip overburden spoils to expose the coal seam by dragline.
- Dump overburden spoils in windrow heaps on the opposite side of the box cut using the dragline.
- Drill and blast the coal seam and remove low grade or contaminated coal.
- Load coal by face shovel or front-end-loader and transport by dump trucks to the Run of Mine (RoM) tip.

As the pit progresses laterally, a series of parallel windrows of spoils heaps, up to 20m above the original ground level, are left behind. Subsequently the windrows are flattened by dozing their peaks into the intervening valleys. The resulting gently undulating landscape is then top-soiled and rehabilitated. A limited area of un-rehabilitated spoil heaps remain till this day around the flooded Ramp 2 and Cut 2 areas of Pit 1. These are clearly visible in the topographical rendering presented in Figure 2-3 overleaf.

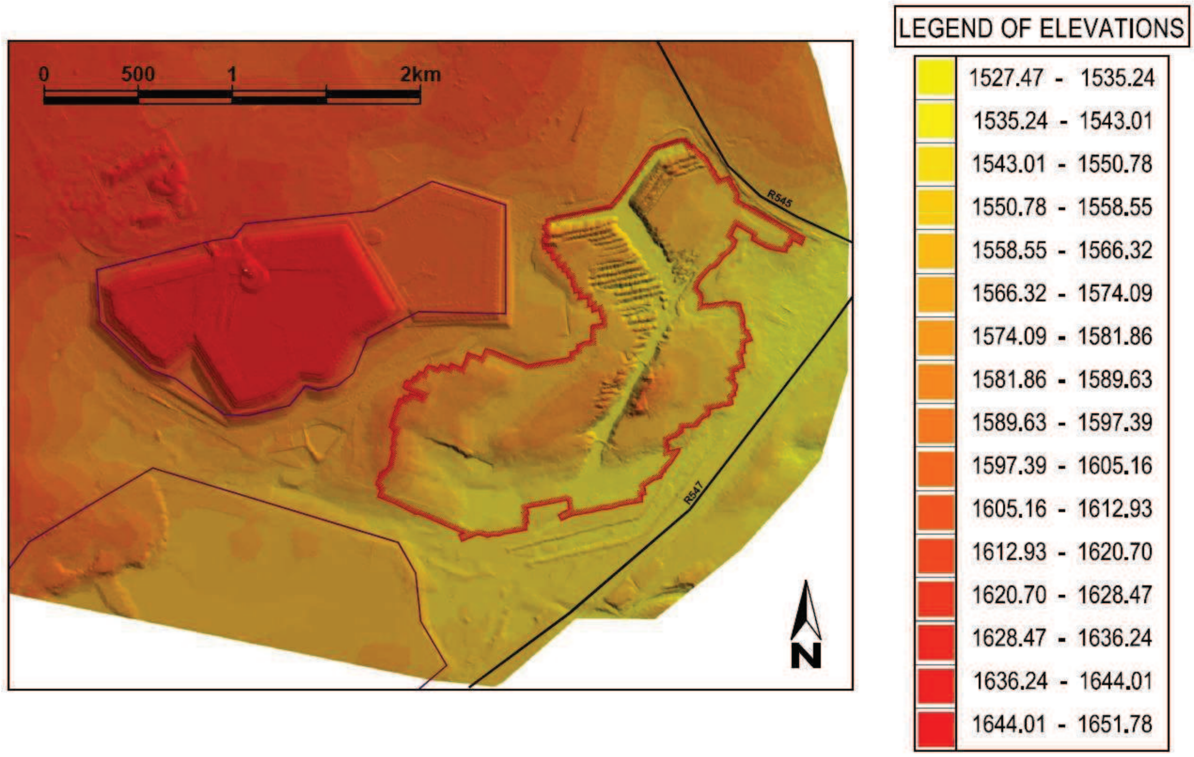


Figure 2-3: Heatmap of current topography

Compared to truck-and-shovel and scraper operations, dragline operations typically result in opencast backfill with a higher void ratio and more cavities and resultant higher potential for settlement.

Expected Ground Conditions in Backfill

Because of the general methods of working, opencast mining operations tend to leave areas where there may be relatively loose fill materials of considerable thickness, which can undergo significant settlement. This is exacerbated by the fact that the backfill is normally of a heterogeneous nature, composed of a wide range of materials including silty sandy & clayey soils, fragments of sandstone, siltstone, mudstone, shale and coal debris. To complicate the situation further, there is considerable variation in the dimensions of the contained fragments, from clay-sized up to several meters across. Replacement of these spoils in the excavation, even with controlled compaction, produces the potential for large differential settlement.

A benefit of dragline operations is the well-mixed nature of the cast spoils. End-tipping operations typically result in segregation as the larger particles roll to the toe of the heap. As a result of the nature of the methods of operation and because of the double handling and weathering effects between initial excavation and final rehabilitation, the siltstone/mudstone/shale components in the spoils backfill can break down and behave as a cohesive material. The harder sandstone components are more durable and remain as cohesionless gravel and boulder inclusions.

2.4 Topography

The topography of the site is somewhat variable due to the nature of the mining activities and the subsequent rehabilitation that has taken place. Figure 2-3 shows the latest topographical survey information in a thematic render. The entire area to the east and south of the complex has been disturbed, either by the mining and rehabilitation activities, or by the construction of the existing ash dams. Where the pit has been rehabilitated the topography is gently undulating, however, there are areas where the dragline heaps still form steep cones of spoil. The general fall of ground is in a south-easterly direction by as much as 50m.

2.5 Drainage

Drainage in the area of interest is dominated by the Bakenlaagtespruit and its tributaries, which drain in an easterly direction towards the Steenkoolspruit and eventually to the Olifants River.

The rivers and streams in this area of the Highveld are moderately incised with narrow well defined floodplains and few meanders, emphasising the dominance of downward erosion on the slope of the original erosion surface. Tributary gulleys provide subordinate drainage in a broad pattern of dendritic drainage. Streams and rivers are classified as intermittent to perennial, but water is available to farmers throughout the year due to numerous natural springs and farm dams. Oval shaped pans and localised depressions occur on the planar surfaces in the region. These pans are considered to be deranged drainage features, remnant from senile rivers superimposed on the African erosion surface.

These pans have localised centripetal drainage patterns and limited catchments. The water levels in the pans vary seasonally and the pans may become dry in the winter months. The regional water table in the Matla/Kriel area lies at a depth of about 30m to 50m below ground surface. The water table is closer to surface in topographic lows and approaches ground level within the major drainage channels. The regional trend of groundwater flow is in a north-westerly to south-easterly direction. However, this regional trend has been disrupted as a result of the mining activities. Groundwater flow in the relatively low permeability Karoo sedimentary rocks is largely confined to structural features such as bedding planes and joints. The No.4 and No.5 coal seams are also known to be fractured and serve as relatively permeable strata in the profile.

Perched water table conditions develop generally within the residual soils at the interface with bedrock and in some cases in pedogenic soils of high permeability. According to farmers in the area (one resident since 1913) the springs that emerge where the perched water tables intersect the topography, have never run dry.

The Site 10 area used to be drained by a system of smaller tributaries, refer Figure 2-4. In 1976, parts of tributaries (red) were diverted and canalised (green) in preparation for the mining of Pit 1 and construction of the ash dam complex. Other parts (purple) were covered by Ash Dam 3 and mined out in Pit 1. Today there are several artificial water bodies (dirty or contaminated water) on the site including the remains of Ramp 2 and the final void of Cut 2 in Pit 1. These filled with water as the groundwater table was restored after mining ceased. Other water bodies include small dams used by Roshcon (ash dam operator and contractor) to circulate the contaminated water back to the ash return water dams.

The current water table in the Cut 2 access ramp is at around 1549 to 1550mamsl. It is controlled by the decant point on the eastern boundary of the site and water bodies.

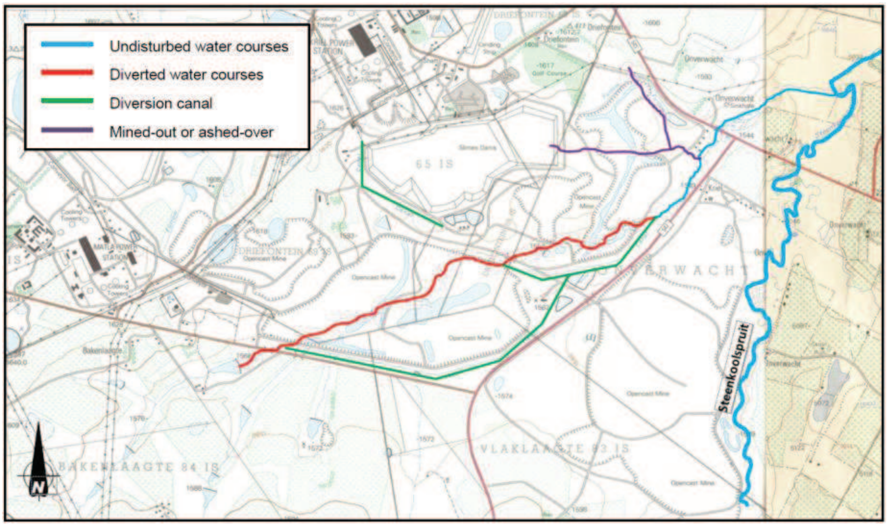


Figure 2-4: Main water courses near Kriel Power Station

2.6 Ash Characteristics

2.6.1 Waste Classification

The waste was classified according to the Waste Act Regulations (GN634 and 635, 2013). Power station ash classifies as a Type 3 waste requiring a Class C liner, in accordance with GN636 (2013). This has recently been confirmed for a number of Eskom power stations including Medupi, Kusile and Hendrina. The ash classification is a result of the process of combustion, but also the source of the coal. As all ash classified to date yields the same outcome it is deemed sufficient for this study to adopt this classification.

2.6.2 Particle Size Distribution

Foundation indicators were done on five ash samples, the sampling positions are shown below:-

Figure 2-5: Ash Sampling Positions

1. C1: Coarse Ash Sample – Near outlet (fines appear to be washed out);
2. C2: Coarse Ash Sample – week old;
3. C3: Coarse Ash Sample – Placed by trucks;
4. F1: Fine Ash Sample – Wet sample; and
5. F2: Fine Ash Sample – "Dry" Sample.

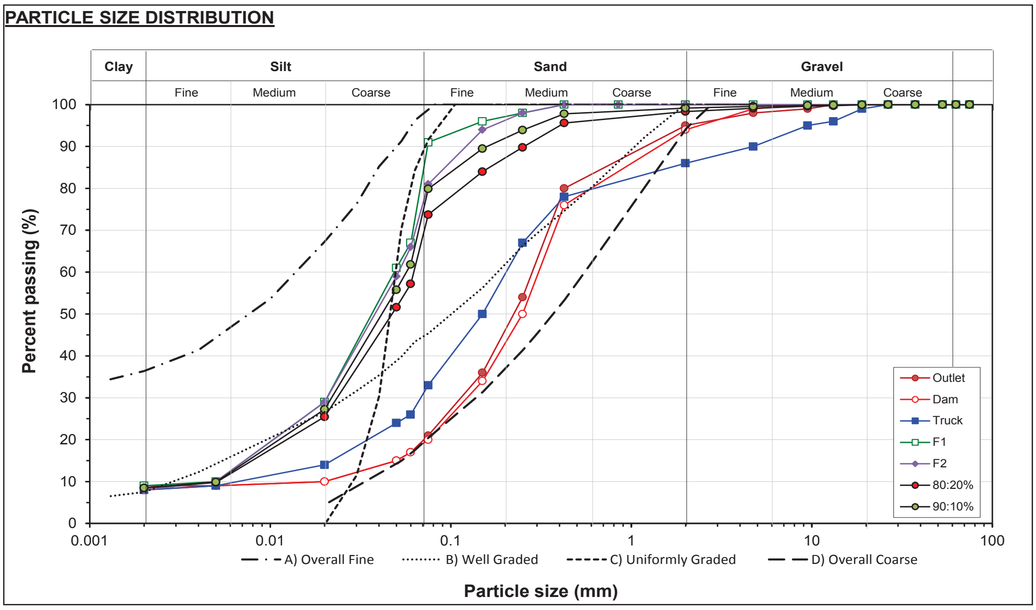


Figure 2-6: Particle Size Distributions

BBA and PFA are produced in a ratio that varies from 20%:80% to 10%:90%. The Particle Size Distribution (PSD) of the combined stream was calculated and is also presented above.

In addition to the ash PSDs, those of typical tailings materials are also shown. These typical tailings materials are used by Robinson (2005) to provide an indication of whether the material is suitable for 'low-effort' spigotting or daywall paddocking in contrast there to.

The figure is, therefore, quite useful to illustrate that the fine ash and combined total stream of ash is best suited to be developed by daywall paddocking. Of higher importance, however, is to note that the BBA has a PSD similar to materials that has been spigotted and hence can probably be deposited on its own.

2.6.3 Permeability

The results from the flexible wall permeabilities carried out in the tri-axial cell on the same ash samples as given in the previous section were:-

Table 2-1: Permeability of ash

Material	Permeability (cm/s)			(m/year)
	Minimum	Maximum	Average	Average
C1	4.8E-05	9.7E-05	8.1E-05	26
C2	8.2E-05	9.2E-05	9.0E-05	28
C3	5.7E-05	7.6E-05	6.3E-05	20
F1	1.2E-05	1.5E-05	1.4E-05	4
F2	1.5E-05	1.9E-05	1.7E-05	5

The table above confirms that the coarse ash is more permeable than the fine ash. The factor that was measured here varies between 4 and 7.

3. CONCEPTUAL DESIGN

3.1 Ash Production and Life of Power Plant

The ash production rate until the end of the design Life of the Power Plant (LoPP) is shown in Table 3-1 overleaf. Values are based on an 80% Generating Load Factor (GLF) and 1 Unit of the power plant being decommissioned every year from the year 2036. The maximum ash production used was provided by Andre Kreuiter for a Maximum Load Factor (MCR) of 100%.

An average ash dry density of 1 t/m^3 was assumed. It should be noted that if the ash dry density was assumed to be 0.9 t/m^3, the remaining life of the existing dams will only be until the year 2019 and not 2021.

A 5 year contingency has been allowed for, thus it has been assumed that the Power Plant will be operated for an additional 5 years at full load from 2036 to 2040, thereby pushing the decommissioning dates out from 2041 to 2045.

Note in Table 3-1 that the planned ash production reduces from the year 2036 due to the decommissioning of generating units as mentioned above. However, a full design ash production with all units operational was used for an additional 5 years from the year 2036 to account for a 5 year contingency.

3.2 Development and Deposition Method

The most common upstream method in South Africa for constructing a conventional wet ash dam, which is also current operations on site, is referred to as the daywall method.

The daywall method comprises separating the dam into two areas. One dedicated to day deposition which is along the perimeter of the dam and the other to night deposition directly into the basin of the dam. The daywall allows for construction of paddocks to contain the ash and build freeboard thereby impounding the ash deposited during the night.

At present the PFA is deposited at a single point (via conveyor) where it is slurried or sluiced and allowed to gravitate along the daywall, whereas the BBA is pumped and deposited by open-ended deliveries at selected positions directly into the basin of the dam. Only PFA is deposited on the daywalls as BBA has a higher permeability than PFA and will cause horizontal preferential seepage paths. These seepage paths could result in seepage daylighting on the slopes and could cause sloughing and possibly instability.

A portion of the BBA is also hauled to site, the origin of this ash could not be established, but is believed to be from operations to remove clinkers from the boiler.

Table 3-1: Design Ash Production

Maximum Power Station Ash Production				3 700 000	tonnes/year	
No. of Units				6	Unit	
Maximum Unit Ash Production				616 667	tonnes/year/unit	
Fly Ash (80%)				2 960 000	tonnes/year	
BBA (20%)				740 000	tonnes/year	
Fly Ash Sold				329 000	tonnes/year	
BBA Sold (uncertain)				0	tonnes/year	
Plan year	**Calendar Year**	**Units**	**Generation Load Factor**	**Planned Ash Production [tonnes/year]**	**Contingency [tonnes/year]**	**Design Ash Production (rounded) [tonnes/year]**
1	2015	6	0.8	2 631 000	-	2 631 000
2	2016	6	0.8	2 631 000	-	2 631 000
3	2017	6	0.8	2 631 000	-	2 631 000
4	2018	6	0.8	2 631 000	-	2 631 000
5	2019	6	0.8	2 631 000	-	2 631 000
6	2020	6	0.8	2 631 000	-	2 631 000
7	2021	6	0.8	2 631 000	-	2 631 000
8	2022	6	0.8	2 631 000	-	2 631 000
9	2023	6	0.8	2 631 000	-	2 631 000
10	2024	6	0.8	2 631 000	-	2 631 000
11	2025	6	0.8	2 631 000	-	2 631 000
12	2026	6	0.8	2 631 000	-	2 631 000
13	2027	6	0.8	2 631 000	-	2 631 000
14	2028	6	0.8	2 631 000	-	2 631 000
15	2029	6	0.8	2 631 000	-	2 631 000
16	2030	6	0.8	2 631 000	-	2 631 000
17	2031	6	0.8	2 631 000	-	2 631 000
18	2032	6	0.8	2 631 000	-	2 631 000
19	2033	6	0.8	2 631 000	-	2 631 000
20	2034	6	0.8	2 631 000	-	2 631 000
21	2035	6	0.8	2 631 000	-	2 631 000
22	2036	5	0.8	2 137 667	2 631 000	2 631 000
23	2037	4	0.8	1 644 333	2 631 000	2 631 000
24	2038	3	0.8	1 151 000	2 631 000	2 631 000
25	2039	2	0.8	657 667	2 631 000	2 631 000
26	2040	1	0.8	164 333	2 631 000	2 631 000
27	2041	0	0.8	0	-	2 138 000
28	2042	0	0.8	0	-	1 645 000
29	2043	0	0.8	0	-	1 151 000
30	2044	0	0.8	0	-	658 000
31	2045	0	0.8	0	-	165 000

3.3 Height and Rate of Rise Limits

The final allowable height and rate of rise are key aspects that need to be considered in order to develop the footprint area and slope geometry of AD4. A general discussion of these limits are presented in this section as input to the conceptual design.

3.3.1 Height Limitations

The final heights of the various Ash Dams are limited by:-

(a) Ensuring stability of the ash dam and foundations.

(b) Maximum safe and sustainable rate of rise that increases as the dam height increases and the available deposition area decreases (i.e. for downstream development).

(c) Impacts including visibility from the surrounding environment and dust. These impacts should be assessed as part of the Environmental Impact Assessment (EIA).

(d) The size of the potential area or Zone of Influence (ZoI) around the dam that would be influenced should the dam fail. The area should be defined by a dam breach analyses and an assessment of the potential loss of life, land use and infrastructure.

(e) Arbitrary or time limits that are specified in the environmental or water use licensing. It is understood that no current limit is specified in the licences of the existing dams.

3.3.2 Rate of Rise (RoR) Limitations

The safe and sustainable RoR is defined here as the minimum of either the stability limit or the operability limit.

The stability limit in turn is influenced by:-

(a) Strength characteristics of the tailings.

(b) Consolidation and drainage characteristics including drying by wind and sun (desiccation) of the ash to control and reduce de-stabilizing pore pressures.

(c) Available drainage directions (i.e. up, down and lateral).

The operability limit is defined as RoR where the dam rises too fast to allow drying and subsequent repacking of the daywalls. In simple terms it is the RoR where the dam is simply too wet to access by machine or by labourers.

The maximum rate of rise is specified as part of this project as 3m/year for stability and 3.5m/year for operability. The operability limit is applied to determine the height of starter walls and the stability limit for final height. These limits are empirical and are based on ash dams that are effectively managed within South Africa.

Note that the RoR limit for the Infill area was specified as 4.5 m/year. This is higher than the sustainable RoR of 3.5 m/year but is deemed acceptable as the Infill area is small and tonnages can be diverted during wet summer periods when the ash is too wet to pack.

3.4 Development Phases

As discussed in Section 1.3, the purpose of this concept development was to delay development over backfilled open pit areas. This is achieved by developing Ash Dam 4 in three compartments as follows:-

- Ash Dam 4.1: Overlying natural ground south of Ash Dams 1-3.
- Ash Dam 4.2: Overlying natural ground east of Ash Dam 3.

- Ash Dam 4.3: Overlying pit backfill south of Ash Dams 3, 4.1, 4.2.

In addition to the above, AD4.1 and 4.2 serve as stability buttresses for the east and south sides of AD1-3 that have stability concerns. Of particular importance is to note that AD4.1 is extended over the existing AWR dams to maximise the buttress of AD1.

The footprint area for the three compartments are shown in Figure 3-1 below.

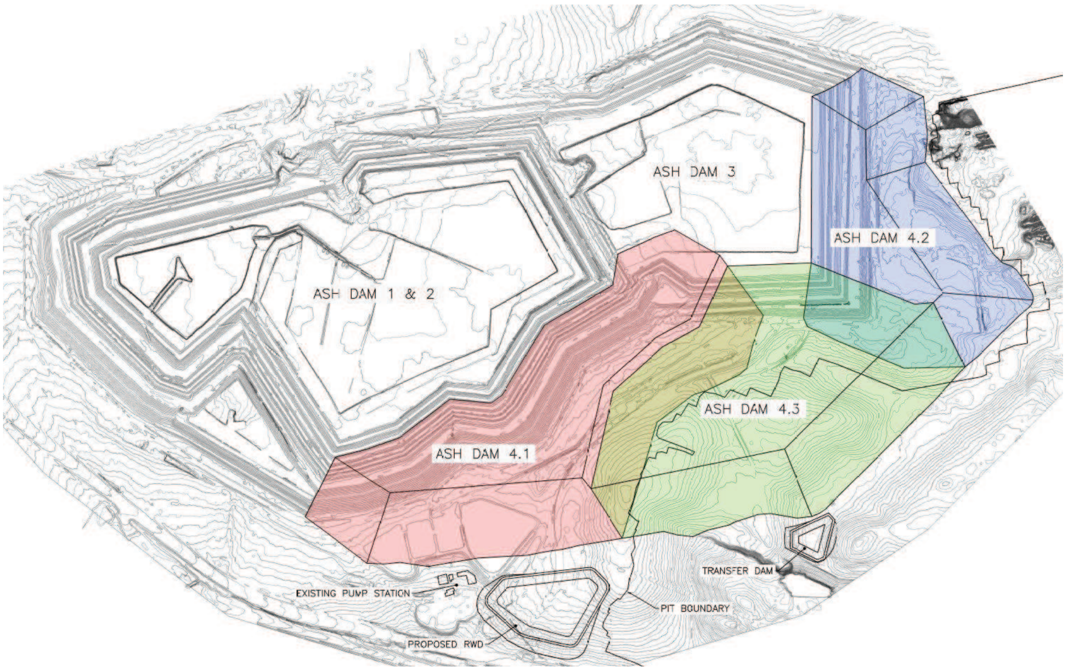


Figure 3-1: Ash Dam 4 Concept

3.4.1 Design Development Sequence

This development sequence allows for deposition space for the design ash load (i.e. 80% GLF) and the decommissioning of the existing AD1-3 once all phases of AD4 are operational.

Deposition is distributed between the dams to optimise the remaining life. When all the existing dams reach the limiting Rate of Rise (RoR) of 3m/year, the available life is maximised, and AD4 should be commissioned.

It was determined that the existing dams will reach their limiting RoR in July 2021. At this stage AD4.2 will have to be commissioned.

Deposition was split between the existing and new dams in order to reduce the height of the preliminary starter walls, as well as the final height of the new dams. It was assumed that deposition on the existing dams will continue for 4 years after the commissioning of the first phase of AD4 (i.e. until the final phase of AD4 is commissioned). Once AD4.1, 4.2 and 4.3 are operational the existing dams will be decommissioned, and rehabilitated.

A period of 2 years was allowed for between the construction phases of AD4 in order to defer large immediate capital costs. Thus after AD4.2 is commissioned in July 2021, AD4.1 will be commissioned in July 2023 and subsequently AD4.3 in July 2025. The RoR, Tonnages, Height and Elevation in time is shown in Figure 3-2 overleaf.

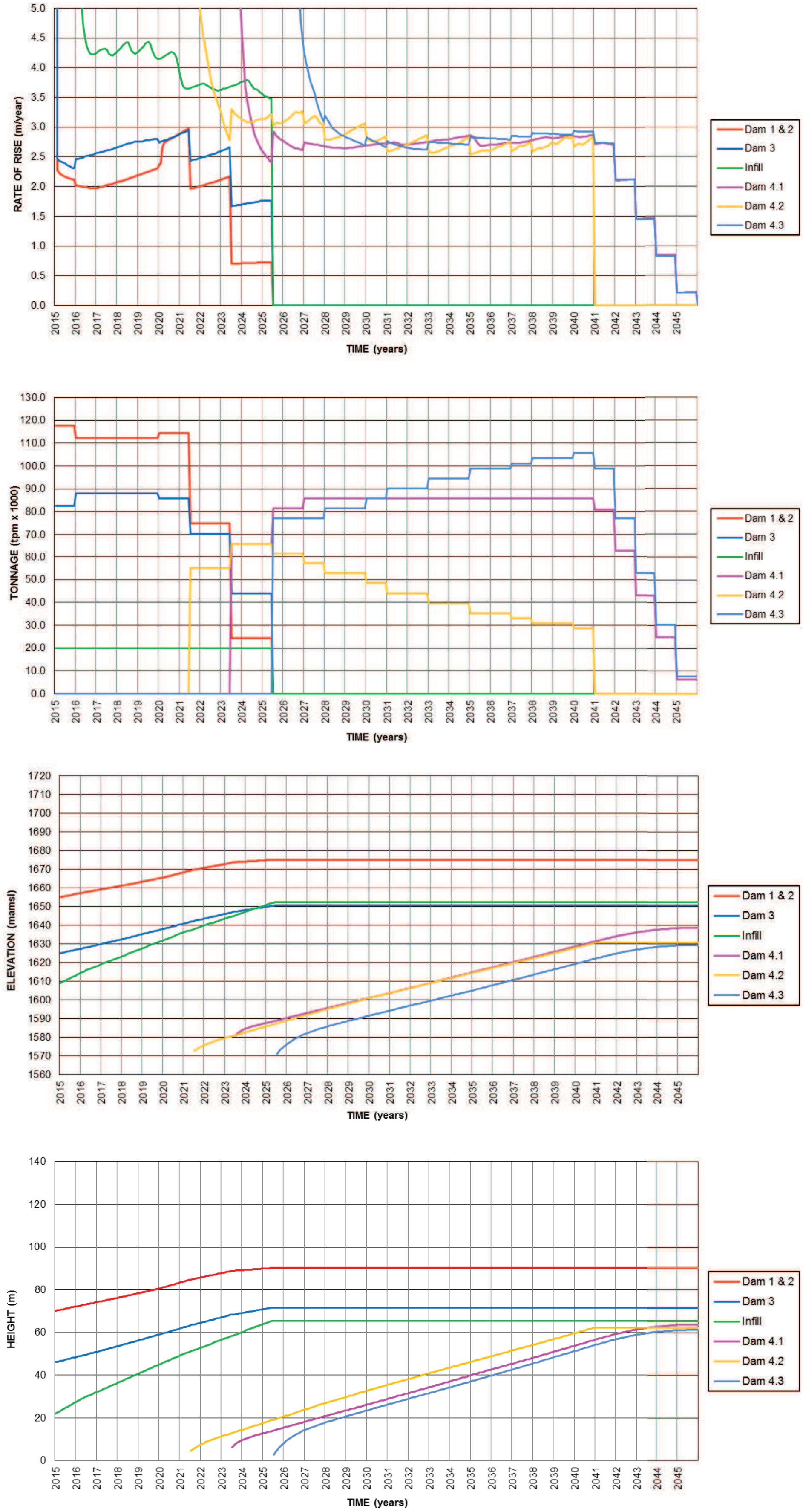


Figure 3-2: Stage-Capacity Relation – Design Ash Load

The following table summarises the properties of all the deposition areas at the end of each stage:-

Table 3-2: Stage-Capacity Summary – Design Ash Load

Deposition Stage	*Start date*	*End date*	*Deposition Area*	*Height (m)*	*Elevation (mamsl)*	*RoR (m/year)*
Deposition onto Existing Dams until AD4.2 is commissioned	January 2015	July 2021	Ash Dams 1 & 2	85	1670	1.96
			Ash Dam 3	63	1642	2.43
			Infill Area	51	1638	3.66
Deposition onto Existing Dams and AD4.2 until AD4.1 is commissioned	July 2021	July 2023	Ash Dams 1 & 2	89	1674	0.70
			Ash Dam 3	68	1647	1.67
			Infill Area	58	1646	3.69
			Ash Dam 4.2	13	1581	3.30
Deposition onto Existing Dams, AD4.2 and AD4.1 until AD4.3 is commissioned	July 2023	July 2025	Ash Dams 1 & 2	90	1675	
			Ash Dam 3	72	1651	
			Infill Area	66	1653	
			Ash Dam 4.2	19	1588	3.02
			Ash Dam 4.1	14	1589	2.93
Deposition onto only AD4 until the end of the Life of Power Plant	July 2025	December 2045	Ash Dam 4.2	61	1630	
			Ash Dam 4.1	64	1639	
			Ash Dam 4.3	62	1630	

3.4.2 Alternative Development Sequence

This development sequence allows for deposition space for the current ash load that is produced (i.e. 70% GLF) while maintaining AD1-3.

This option has the major benefit that AD4.3, which overlies the pit backfill does not need to be constructed.

Approval by the authorities will be required in order to continue the operation of the existing dams. This matter should be discussed as a priority, in addition to the overall stability of the facility, as it influences the design.

Table 3-3: Stage-Capacity Summary – Alternative Development Plan

Deposition Stage	*Start date*	*End date*	*Deposition Area*	*Height (m)*	*Elevation (mamsl)*	*RoR (m/year)*
Deposition onto Existing Dams until AD4.2 is commissioned	January 2015	July 2021	Ash Dams 1 & 2	82	1667	1.58
			Ash Dam 3	61	1640	1.99
			Infill Area	48	1635	3.66
Deposition onto Existing Dams and AD4.2 until AD4.1 is commissioned	July 2021	July 2023	Ash Dams 1 & 2	85	1670	0.55
			Ash Dam 3	65	1644	1.34
			Infill Area	54	1641	3.18
			Ash Dam 4.2	12	1580	3.04
Deposition onto Existing Dams, AD4.2 and AD4.1 until the end of the Life of Power Plant	July 2023	December 2045	Ash Dams 1 & 2	105	1690	
			Ash Dam 3	92	1671	
			Infill Area	88	1675	
			Ash Dam 4.2	62	1630	
			Ash Dam 4.1	64	1638	

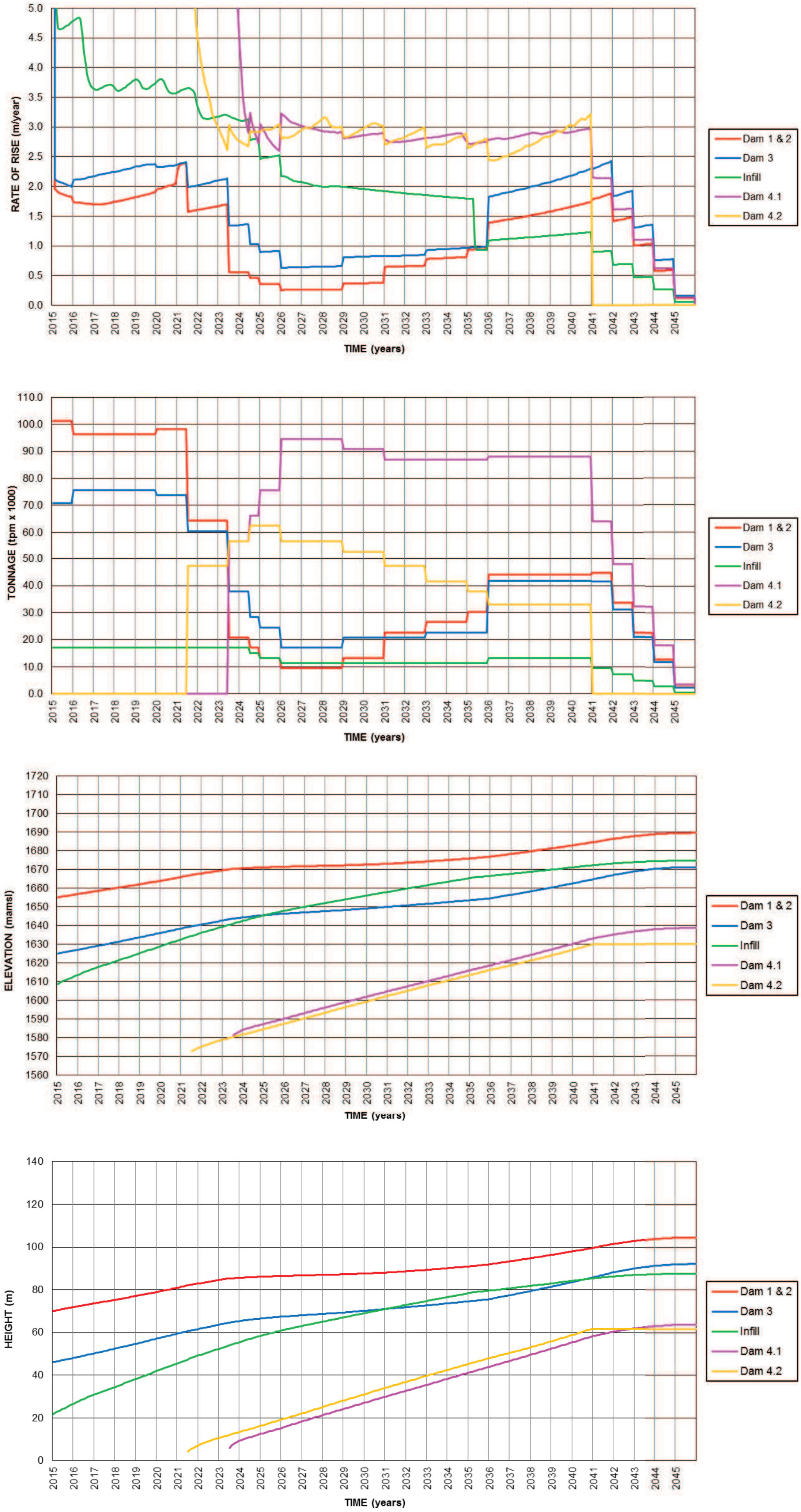


Figure 3-3: Stage-Capacity Relation – Alternative Ash Load

3.5 Pre-Deposition Works

3.5.1 Description

The works required before operations can start on AD4 is discussed in this section and includes the reshaping of the basin to allow proper drainage, construction of starter walls, lining the site and providing the necessary drainage boundaries between clean and dirty water systems.

3.5.2 Outer Wall Profile

Upstream Slopes

The upstream slopes must be specified as steep as possible whilst ensuring that the impermeable layer can be constructed successfully. The following practical limits is specified:

- For clay layers placed in horizontal layers, a slope of 1(V):1.5(H) is acceptable as the surface can be prepared and made smooth to accept a HDPE geomembrane liner.
- For clay layers placed parallel to the slope a 1(V):3(H) slope is required to allow earthmoving equipment such as bulldozers and compactors to move on the slope safely without shearing the clay layers beyond repair. Some shear cracks from vehicle loads will however occur.

Downstream Slopes

The downstream slopes must be specified as steep as possible whilst ensuring that:

- Closure requirements in terms of erosion and soil loss are acceptable.
- Visual impact is acceptable.
- Vegetation requirements can be achieved.
- The slope remains stable.

The average (overall) slope of Ash Dam 4 is specified as 1(V):4(H). This slope will be achieved by constructing the dam wall as follows (see Figure 3-4):

- Vertical lifts of 10m with an average minimum side slope of 1(V):3.5(H);
- Benches 10m wide between vertical lifts.

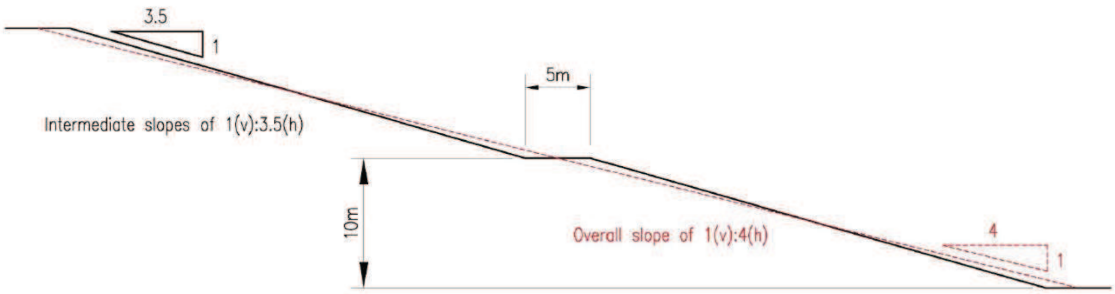


Figure 3-4: Typical Wall Section

3.5.3 Differential Settlement Mitigation

(a) Re-shaping and overfill

AD4.3 will be developed on backfilled pits. The backfill is highly compressible relative to natural ground which will result in overall and probably differential settlement. The footprint areas of the new ash dams must therefore be re-shaped to ensure that drainage will be maintained towards the perimeter or toe areas of the outer slopes. The perimeter is not expected to settle substantially as no additional overburden load is applied as opposed to the centre of the footprint where the ash is at maximum height imposing a substantial additional load on the foundations. The inner part of the dam will therefore settle more than the outer perimeter and must be over-steepened to ensure continuous drainage. In order to ensure a final 1:200 slope once the dam is fully developed with the associated load conditions, a 1(v):63(h) basin slope must be constructed as part of the pre-deposition works.

It should be noted that the majority of AD4.3 has steep slopes in excess of 1:63 and only reshaping of the Pit 1 ash fill as part of the east portion of the AD4.3 footprint needs re-shaping.

(b) Routing of Drainage and Seepage

The liner system will provide drains above and below the liner as described in Section 3.5.6. The drain pipes will be placed both sides of the backfilled pit high wall where the maximum differential settlement and associated risk of liner damage is expected. By this measure seepage water is kept away from these areas.

3.5.4 Topography and Drainage

There are several areas that are too flat to allow drainage that will need to be reshaped as part of the site preparation for the basin of AD4. A minimum slope of 1(v):200(h) and maximum of 1:5 should be provided to allow free drainage and easy liner installation across the site.

The basin areas within the backfill that are underneath the side slopes of AD4 will require a minimum slope of 1:63 to take into account the effects of differential settlement. An increase in the amount of earthworks can be expected if a steeper slope is required than the one calculated. The feasibility study should include an update of the minimum slope required based on the findings of the Monitoring Trial Embankment (MTE).

The main areas that will require reshaping are shown in Figure 3-5. The volumes have not been optimised and the site topography should be considered carefully during the feasibility design to include the impact of the amount of expected settlement over the backfilled pit. The dozing of the MTE and reshaping of the existing return water dam area is not shown on the figure below and should also be included in the site preparation works.

The site predominantly drains in a southerly direction on the western side of the site and easterly on the eastern side.

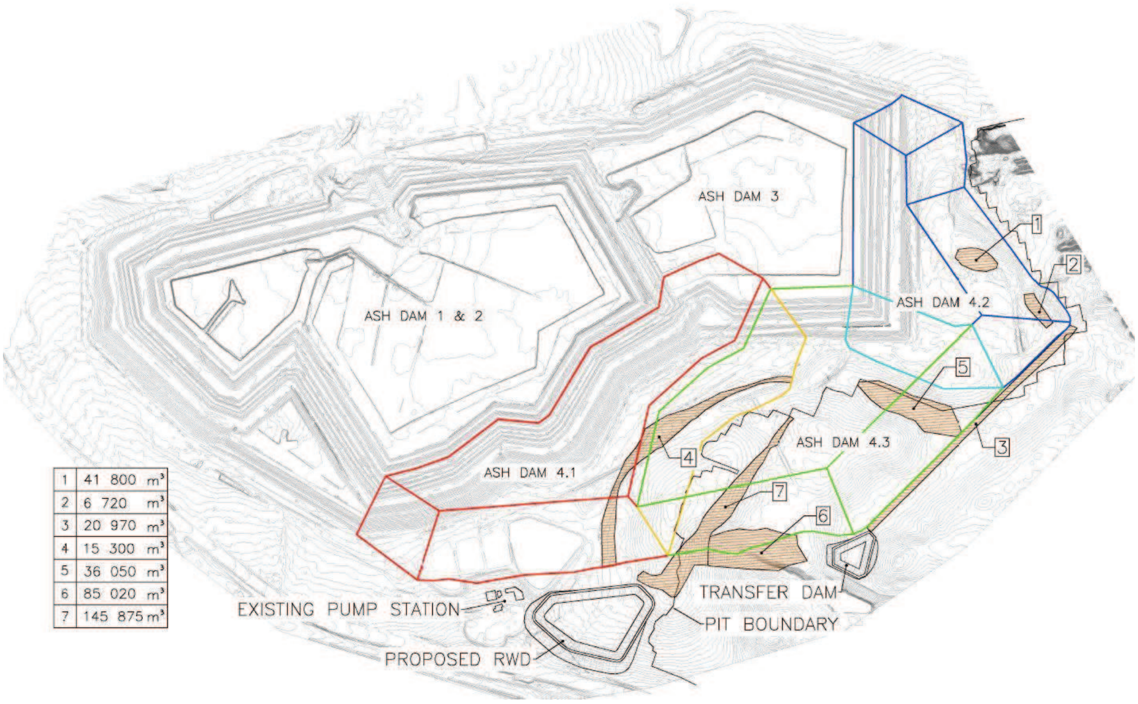


Figure 3-5: Earthworks – Reshaped basin

3.5.5 Starter Walls

The toe of the dam is defined on site by starter walls. The starter walls contain the initial deposition, before construction of the ash dam in an upstream direction starts.

The height of the starter walls is a function of the rate of rise and as a result the amount of ash tonnages reporting to the dam. The elevation required for the starter walls will be where the rate of rise is low enough to allow most of the excess pore pressures to dissipate and the ash to be packed.

An acceptable rate of rise of 3.5m/year was specified. If all the ash is diverted to the new ash dams at once, large starter walls will be required. It is therefore recommended that deposition initially be split between the new and existing dams, to reduce the rate of rise and the ash tonnages reporting to each dam.

To reduce the size of the starter walls required it was assumed that AD4.1, AD4.2 and AD4.3 only receives 30%, 25% and 35% of the total slurry stream, respectively, upon commissioning until their RoR reached 3.5m/year.

For the abovementioned assumption a maximum starter wall height of 11m on AD4.1, 11m on AD4.2 and 17m on AD4.3 will be needed, see Figure 3-6.

The starter wall parameters are as follows:-

Minimum height	:	1.5m (nominal starter wall)
Crest width	:	7m for main starter walls and 3m for nominal starter walls
Inside slope	:	1 (v) : 1.5 (h)
Outside slope	:	1 (v) : 3 (h)

The starter walls should be properly compacted, e.g. 95% Standard Proctor at -2% to 2% OMC, to provide a solid structure, should settlement occur.

For clay layers placed in horizontal layers, a slope of 1(V):1.5(H) is acceptable as the surface can be prepared and made smooth to accept a HDPE geomembrane liner.

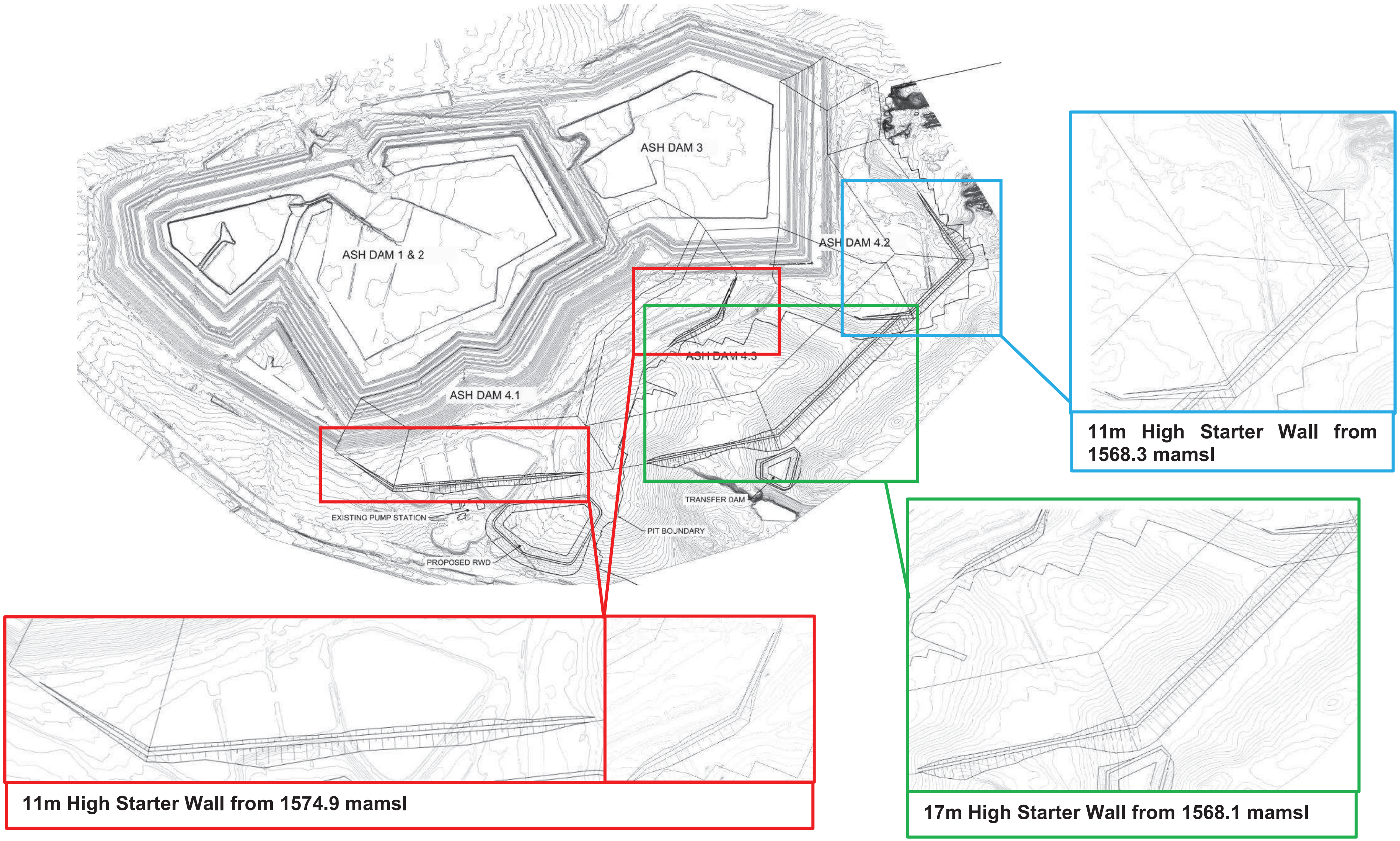


Figure 3-6: Starter Walls for New Dams

3.5.6 Liner System

As discussed under Section 2.6.1, ash classifies as a Type 3 waste, which requires a Class C liner in accordance to GN636 (2013). The regulatory liner and liner component specification are shown below.

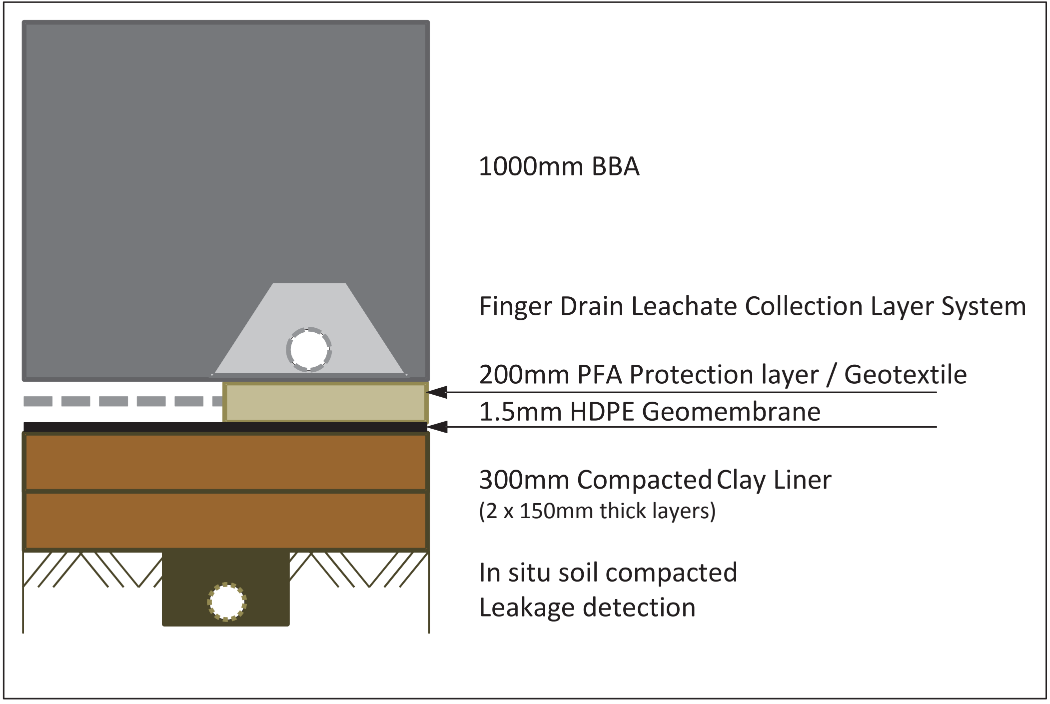


Figure 3-7: Class C landfill liner

Each layer of the liner system is described below starting from the excavation level:-

(a) In situ soil

The footprint will need to be cleared and grubbed of all vegetation prior to start of construction of each phase of development. The upper 200mm of topsoil will also need to be stripped and stockpiled in a designated area to be used later for the rehabilitation of the outer slopes and final surface (J&W Report no. JW196/11/C779). The resulting level after shaping earthworks of the facility basin to provide minimum slopes of 1:200.

(b) Subsoil drain

The drain will act as partial seepage detection and groundwater control. The system will consist of drains directly under the leachate collection drains.

The subsoil finger drains will consist of:-

NB of primary pipe	:	200mm
NB of secondary pipes	:	160mm
Media for pipes	:	Thick walled HDPE (SDR 7.4)
Spacing of branching pipes	:	100m c/c

Clean sand, not BBA, must be used as it is below the liner system and in direct contact with clean soils

(c) Composite Liner

The liner will consist of a 300mm (i.e. 2 x 150mm) thick layer of compacted clay in direct and continuous contact with a 1.5mm double-textured HDPE geomembrane. All geomembranes under the outer walls will be double-textured for stability purposes.

(d) Leachate Collection System – Dam Basin

In terms of regulatory requirements only finger drains are required for a Class C barrier to form the leachate collection system and not a continuous drainage layer over the whole footprint of the dam that is required for a Class A and B barrier.

The purpose of leachate collection is to control and limit the leachate head that acts on the liner, thereby reducing the hydraulic driving head and associated infiltration volume through the barrier. Proper drainage over the liner also aids stability as pore pressure build-up is reduced.

As part of this concept a continuous 1000mm thick layer of BBA is provided, which will be placed mechanically in addition to the finger drains, to act as a drainage layer to reduce pore pressure build up on the liner as the deposit is wet and stability must be assured.

The finger drains will consist of:-

NB of primary drainage pipes	:	300mm
NB of secondary or branching pipes	:	160mm
Pipe material	:	Thick walled HDPE (SDR 7.4)
Spacing of branching pipes	:	100m c/c

The BBA will need to be deposited in a designated area in advance of construction to create a stockpile that can be re-mined, hauled to site and placed mechanically. The infill area as well as the deposition points on Dam 3 against Dams 1 & 2 could serve as stockpile areas. Haulage from Dam 3 was provided for in this study as it is an existing BBA deposition point that can be accessed without compromising the integrity of the existing dams.

As a general rule, drainage material is specified to have a permeability of at least 10 times or one order of magnitude higher than the material that is to be drained (i.e. overlying PFA). The BBA is coarser and 4 to 7 times more permeable than the PFA (Refer Table 2-1). Even though this permeability difference between BBA and PFA is not 10 times the BBA will serve the purpose of drainage if the thickness and ultimate drainage capacity is correctly specified. In terms of this study a 1000mm thick BBA layer is deemed sufficient and probably conservative.

The BBA contains 20% gravel sized particles that will puncture the liner and a thin protection or cushion layer of PFA will be required to be placed directly on the HDPE geomembrane and under the BBA. A 200mm thick mechanically placed layer is provided overlain by a separation geotextile to reduce dust generation during construction and to reduce the risk of displacing the PFA during placement of the BBA.

As part of the following phase of basic and/or detail engineering the system can possibly be optimised by investigating:-

- Hydraulic placement of the BBA as part of operations as opposed to expensive mechanical placement. Hydraulic placement will wash away the PFA cushion and if this option is to be considered in-line screening of the slurry stream of BBA will have to be investigated to allow the omission of the PFA layer and placement of BBA directly on the liner.

- Hydraulic placement without screening in paddocks to cover the liner as the ash level rises. The paddocks can be constructed over the PFA protection and geotextile cover layer. The effect of the paddocks will be to reduce flow over the PFA and geotextile and reduce the risk of PFA wash-away. Fines can also be decanted off the paddock to completed lower areas that has already been filled with BBA.

- Cycloning of the BBA to dewater the BBA and to allow easy hydraulic placement followed by dozing to place and to extract the fine fraction that inhibits the high permeability potential of the larger fraction of the BBA. BBA is highly abrasive and abrasion resistant cyclones will be required that may require, in the extreme, expensive ceramic lining. A bottom-up construction sequence will be required to create a low-lying area where fine cyclone overflow can be deposited while the leachate collection layer is developed in new higher lying areas. The initial low-lying area will have to be formed by mechanical placement of BBA.

The theoretical permeability of the BBA for various fractions of fines removal was calculated as proposed by Hazen and reproduced by Craig, 2004 in Table 3-4 below. From the table it is evident that the permeability will increase by three to four orders of magnitude if cycloning is considered.

Table 3-4: Permeability of coarse ash with percentage of fines removed

% fines removed	D_{10}	k
	mm	cm/s
0%	-	1.5e-05*
25%	0.14	1.9E-02
50%	0.27	7.3E-02
75%	0.42	1.8E-01

* Average measured as per Table 2-1

A filter compatibility check was carried out in accordance to the criteria set by Sherard et al (1989) for granular filters. Note that the cycloned BBA will be a suitable filter for the total stream, PFA or BBA deposited separately.

(e) Leachate Collection System – Against Ash Dams 1-3

A drainage layer of BBA will be placed against the side slopes of the existing dams, 5m wide, to form a drainage layer which will drain water at the interface between the new and existing dams. The seepage will report to the under drainage system of AD4 at the toe of the existing dams, refer Figure 3-8. The 5m width is specified conservatively thick due to the fact that this layer will form part of routine deposition and could potentially be as thick as the proportion of BBA that is deposited, which varied between 10% and 20% of the total ash load. No screening is required as the BBA will not be in contact with a liner system and large particles causing damage is not a concern.

Note that a Class C liner will be provided on the wide horizontal bench approximately mid-height of the existing ash dam outer walls, to collect leachate from the upper slope areas. This is deemed a sufficient barrier as the side slopes are steep providing drainage towards the drain pipes at the toe and bench.

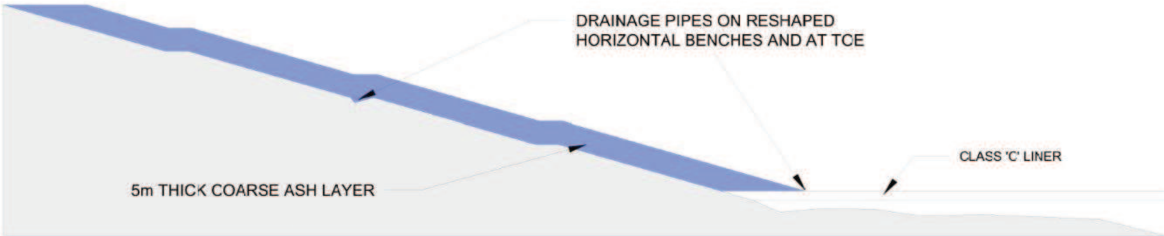


Figure 3-8: Detail of drainage layer placed between existing dams and AD4

3.5.7 Solution Trenches and Storm Water Diversion Berms

The solution trenches at the toe of the ash dams are needed to, firstly, prevent run-off from the side slopes of the dam spilling into the environment and, secondly, convey the leachate collected via the underdrains and decant from the penstock to the Ash Water Return (AWR) dam.

The storm water diversion berms will act as the boundary between the clean and contaminated water and will be constructed from the cut material of the solution trenches, see illustration below.

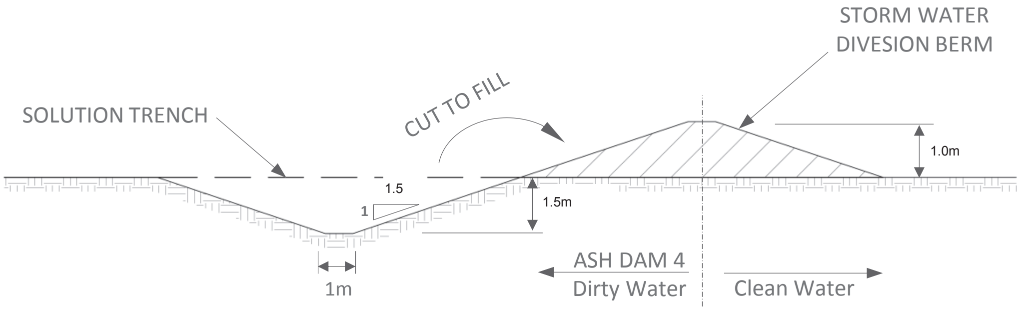


Figure 3-9: Typical section of a solution trench with a storm water diversion berm

The solution trenches need to accommodate a 1:50 year, 24 hour storm.

The solution trench parameters are as follows:-

Width of basin of trench	:	1m
Depth of trench	:	1.5m
Side slopes	:	1 (vertical) : 1.5 (horizontal)
Liner	:	100mm thick, 25MPa concrete filled geocells underlain by a 1.5mm thick geomembrane.

The storm water diversion berms parameters are as follows:-

Material	:	Cut material from solution trench
Width of crest	:	3m
Height of berm	:	1m
Side slopes	:	1 (vertical) : 1.5 (horizontal)
Compaction specification	:	Compact in 150mm layers (1km freehaul) 95% Std. Proctor and moisture content at -2% to +2%.

3.6 Slurry Delivery System

3.6.1 Deposition Method

The ash dam is to be developed as a ring dyke with the outer walls raised continuously using machine packed day walls.

Deposition will be on a planned cycle so that:

- The rate of rise of the outer wall exceeds or is at least equal to that of the basin;
- The crest of the dam remains as level as practical within freeboard requirements;
- Sufficient deposition area is available at any time;
- The pool is always located at the penstock inlet.

In total there are six deposition areas, which comprise of the following:

1. Ash Dams 1 and 2;
2. Ash Dam 3;
3. Infill Area;
4. Ash Dam 4.1
5. Ash Dam 4.2
6. Ash Dam 4.3

The PFA and BBA will be distributed around the dams by a combination of pumping and gravity flow as follows:

- All BBA will be pumped from the current slurry booster pump station and deposited into the basin of AD1-3 and later on in AD 4.1 and 4.2 to form the leachate collection system (Refer Section 3.5.6). To form the leachate collection system BBA must be pumped without mixing to AD4 as it serves as a drainage layer. Mixing BBA with fine ash yields a product with a permeability that is too low to serve as a drainage layer.
- All PFA will need to be pumped to allow deposition at specific delivery stations whilst keeping areas of inactive delivery stations dry. This is required for stability purposes. The ash dam daywalls rely on sun and wind drying to cause desiccation, associated over-consolidation of the outer wall and reduction of pore pressures that ultimately provides the environment to develop stable outer walls. Note that current practice is to sluice all PFA from the distribution point at Dam 1. This has the effect that the areas between the deposition point and the sluice point remains wet and prevents desiccation, feeds water and causes pore pressure build-up in the outer walls.

3.6.2 Required Velocity to Limit Settlement

The pumping requirements were assessed based on pipe friction losses and limiting settling velocities as per Paterson & Cooke (2013).

The BBA was assumed to be delivered through the existing 300mm steel pipes. Operations allows for ashing of all six units in one 8 hour shift and pumping all ash away in 45min per unit. Slurry density was adjusted to assure a settling velocity of 2.48m/s that corresponds to a solids concentration of 15% by weight or a slurry density of 1.096t/m3.

The PFA was assumed to be sluiced to a solids concentration of 20% by weight or a slurry density of 1.132t/ m3. In order to reduce friction losses to allow the existing pumps to be re-used 2 x 400mm pipes are required. The settling velocity of this configuration is 1.03m/s while the actual flowrate will be 1.99m/s providing for 40% additional capacity (i.e. the ash stream is pumped away in 14.4hours in a 24 hour cycle.

3.6.3 Deposition Lines

The delivery lines will be similar to the current system and consist of the following:-

Permanent main line for BBA	:	1 x duty 300mm NB steel pipes
Deposition stations	:	Every 300m
	:	Open-ended deliveries
Permanent main line for PFA	:	2 x duty 450mm NB HDPE pipes Class 10 (PE100)
Deposition stations	:	Every 300m
	:	Open-ended deliveries

3.6.4 Slurry Pump Station

Eskom has constructed a slurry booster pump station at the sluice point on Ash Dam 1. The pump station consist of 4 Metso HG250C3 slurry pumps installed in parallel according to the design report, ECsoft (2009).

Three pumps & three pipelines will be required to provide the duty point (1 x BBA and 2 X PFA). One pump is therefore available for standby. The pump station has a double inlet sump that serves two pumps simultaneously. The pump station therefore seems suitable to pump the PFA as two pumps will be duty and two will be standby.

The definitive assessment of the pump station is beyond the scope of this report but it is the opinion of J&W that a modification will be required to pump the BBA. Eskom can also consider to deliver all BBA to AD3 as the height is less and the existing pumps at the generating may be able to deliver to AD3.

3.7 Decant System

3.7.1 Description

Storm and supernatant water are to be decanted off the basin of the dam by means of a gravity penstock. The penstock will consist of vertical stacked concrete ring towers that is raised as the dam rises and a sub-horizontal thin walled steel outlet pipe that is encased in concrete to drain decant water to the toe of the dam. At the toe of the dam the pipe discharges into a solution trench that drains towards the AWR dam.

Access to the inlet will be provided by means of a conventional pool wall, timber catwalk and timber platform. A decant system will be required for each phase of development to be able to decant off the water. Wing walls will also need to be constructed to assist in maintaining the pool at the inlet.

Note the following:-

- The AD4.1 penstock will drain into the solution trench that drains towards the AWR dam. The penstock will be provided with two inlet positions, the first will serve the dam initially when the outer walls are low and the second will serve the dam once the outer walls have risen and the centre position of the dam has shifted due to the sloping outer walls.

- The AD4.2 penstock will drain under gravity into a transfer dam south of AD4.3. The decant water will have to be pumped to the AWR from the transfer dam. It should be noted that the opportunity exists to drain AD4.2 directly into AD4.1 provided that AD4.2 is commissioned before AD4.1 and that the crest of AD4.2 remain above the AD4.1 crest. AD 4.1 will eventually rise above AD4.2 as AD4.2 is small and needs to be terminated to avoid an active area that is too small to operate properly and maintain pool control. If AD4.3 is constructed the AD4.2 penstock will discharge onto AD4.3 hence eliminating the need for the transfer dam altogether.
- The AD4.3 penstock will drain under gravity directly into the silt trap of the AWR dam. The pipe alignment route will however require substantial excavations to flatten spoils and provide the free draining gradient.
- During construction of AD4.1, the AD1 penstock will be diverted temporarily into the newly constructed AWR dam and later on permanently into the solution trench of AD4.1 on the west. The penstock of AD3 will need to be extended to discharge into the solution trench on the east.

3.7.2 Summary of Properties of Each Penstock

The properties of each penstock are as follows:-

Table 3-5: Penstock Properties

	Ash Dam 4.1	Ash Dam 4.2	Ash Dam 4.3
Design flood	1:50 year, 24 hour storm		
Inlet pipe material	Precast concrete rings		
Inlet pipe diameter	510mm	510mm	510mm
Number of inlets	4	4	4
Minimum slope (V:H)	200	200	200
Outfall pipe material	Steel	Steel	Steel
Outfall pipe diameter	750mm	700mm	650mm
Number outfall pipes	1	1	1
Time to decant (days)	3	3	3

The capacity of a penstock system is the minimum of either inlet or outlet control and both must therefore be accounted for to determine the actual capacity of the system.

4. WATER BALANCE

4.1 Introduction

Jones & Wagener (J&W) performed a water balance in order to assess the Ash Water Return Dam (AWR) capacity requirements of the existing and new AWR as part of the concept design of the future Ash Dam(s).

The input parameters and assumptions discussed below should be verified in order to size the final AWR.

The AWR dam has to be large enough to hold storm and process water and only spill once in every 50 years as per GN704. Implicit to this requirement is to ensure that the AWR is large enough so that water does not need to be stored on the ash dams as is currently the case. This is of particular importance as the current facilities are very high and stability investigations point to marginal stability of the ash dam complex.

The water balance provides an assessment and understanding of the inflows, outflows and resulting storage requirements for the Ash Dams.

Inflows – Outflows = Δ Storage (change in storage)

The inflows and outflows were simulated using daily time steps.

Inflows considered:

- Historical daily rainfall data;
- Ash slurry water pumped onto the ash dams.

Losses considered:

- Evaporation off the deposit. Differentiation is made between wet (i.e. pool) and dry (i.e. beach) areas in the basin, higher evaporation is experienced in wetter areas;
- Lock-in of pore water. Each unit volume of ash that is deposited has voids in between the actual particles (i.e. pores) that are partly filled with water. The pore water drains slowly as the Ash Dam consolidates and the available pore volume decreases. This process of consolidation stops for all practical purposes at some point in time and some of the pore water remains in place and does not drain out. There is therefore a proportion of the water sent out to the dam that gets locked up in the pores of the in-situ ash;
- Average expected seepage through the composite liner.

4.2 Climatic and Geographic Data

The site is situated in quaternary catchments B11D and B11E in the WR2000 series of books.

The Langsloot-Secunda rainfall station (0478330W) was used for daily rainfall. Langsloot-Secunda was used, instead of Kriel rainfall station, despite being further from the site because it has a longer (combined) reliable length of record, good representation of extreme events, better represents the published MAP for the quaternary sub-catchment and is consistent with work done for previous studies. (MAP is 717 mm versus the published MAP of 671 and 682 mm for B11D and BD11E respectively).

Evaporation data published for B11D and B11E was used (MAE of 1600mm).

Evaporation off the tailings pool was factored by 0.9 to account for salinity. Evaporation off the wet and dry beaches was factored by 0.7 and 0.4 respectively, (Blight, 2010)[1].

The permeability to calculate seepage to groundwater was assumed to be 1 x 10^{-6} cm/s for the existing facility and 1 x 10^{-12} cm/s for the new Ash Dam(s), since the new dam(s) will be lined.

4.3 Slurry and Return Water Inputs

The Life of Power Plant (LoPP) tonnages were based on a target 80% Generating Load Factor (GLF) of the Maximum Continuous Rating (MCR) of the power station and monthly ash sales as discussed in the concept design criteria.

Typical slurry properties are shown below. These properties should be confirmed by Eskom.

Table 4-1: Slurry Properties

Slurry Density	1.1 t/m^3
Specific Gravity (solids)	2.4
Specific Grevity (liquids)	1.0
% Saturation	100%
Dry Density	1.0 t/m^3

It was assumed that 100% of water from slurry is pumped from the AWR as return water. This should, however, be confirmed as historic photographs shows the AWR to be at or near capacity constantly. The amount of water pumped as return water will have a significant effect on the dam sizing. This should be confirmed.

4.4 Survey Data

The 25 March 2015 LiDAR survey was used to determine the catchments.

The existing AWR capacity was obtained from the Kriel Ash Dam Operations and Maintenance Manual (Revision 0). The capacity of the silt traps were not considered.

4.5 Catchment

Three scenarios were analysed as follows:

- Scenario 1: Current site (Existing ash dams and AWR);
- Scenario 2: Only new ash dam(s) area as the catchment. Existing ash dams assumed to be rehabilitated;
- Scenario 3: Existing and new dam(s) operated concurrently.

Note that based on the topography of the site only rainfall on the ash dams and directly on the AWR were considered in the water balance. Clean water is assumed to be redirected away from the facility. Clean water cut-offs should be clarified.

[1] Blight G. E. 2010. **Geotechnical Engineering for Mine Waste Storage Facilities**. University of Witwatersrand.

4.6 Water Balance Findings

4.6.1 Scenario 1 (Current situation)

From the analysis it is evident that the existing AWR has insufficient capacity for the current ash dams. The results in Figure 4-1 show that the dam spills during several storm events based on the 95 year rain record used.

The existing ash dams require a facility with approximately 380 Mℓ of storage capacity based on the abovementioned assumptions, opposed to the current storage capacity of approximately 110 Mℓ.

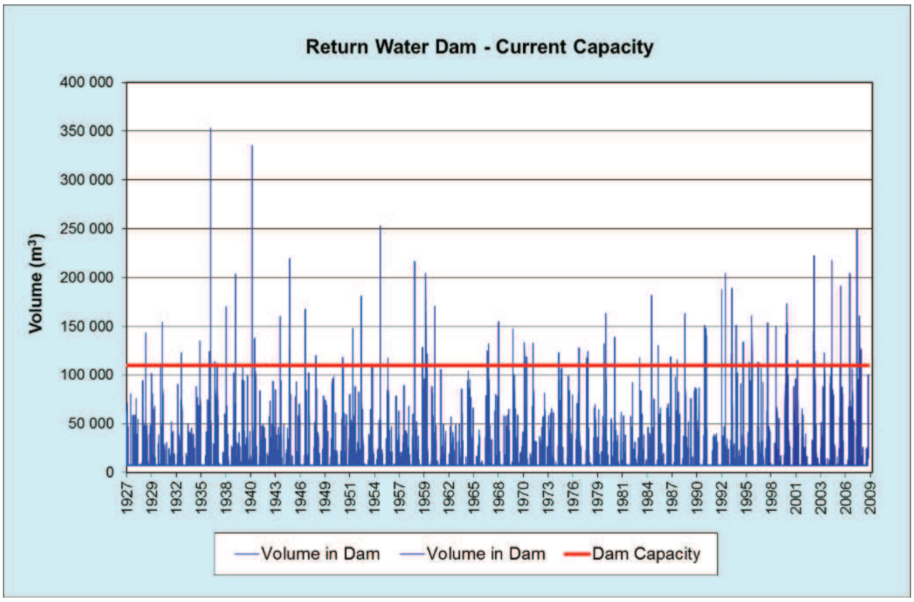


Figure 4-1: Results of Scenario 1 (current facility)

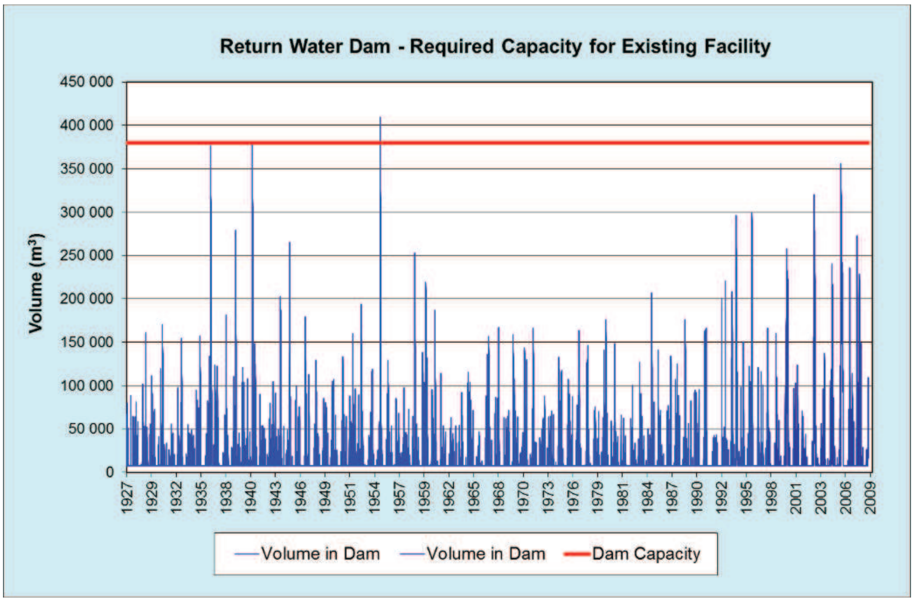


Figure 4-2: Results of Scenario 1 (required facility)

4.6.2 Scenario 2 (Only new ash dam(s) considered)

This analysis was conducted assuming that the catchment will only consist of the footprint area of the new ash dam(s). This implies that the existing ash dams will be rehabilitated and clean water falling on the existing ash dams will be redirected away from the AWR. The results for this analysis can be seen below, and indicates that an AWR with a capacity of approximately 330 Mℓ will be required.

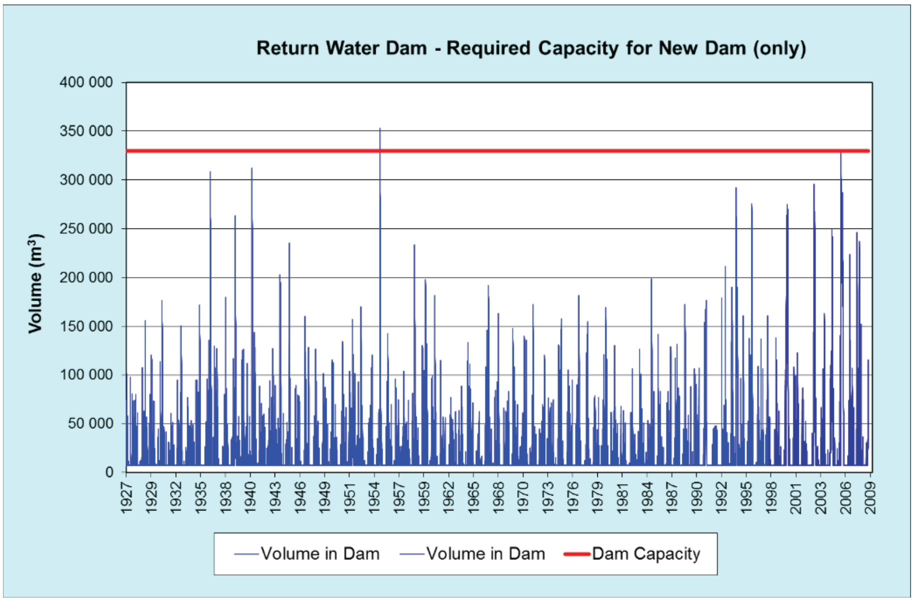


Figure 4-3: Results of Scenario 2

4.6.3 Scenario 3 (Existing and new ash dam(s))

This scenario is effectively a combination of Scenarios 1 and 2. The depth of the new AWR that will have to be constructed was assumed to be 4m in order to determine a capacity of the required AWR. The results for this analysis can be seen below, and indicates that an AWR with a capacity of approximately 710 Mℓ will be required if the new ash dam(s) are constructed.

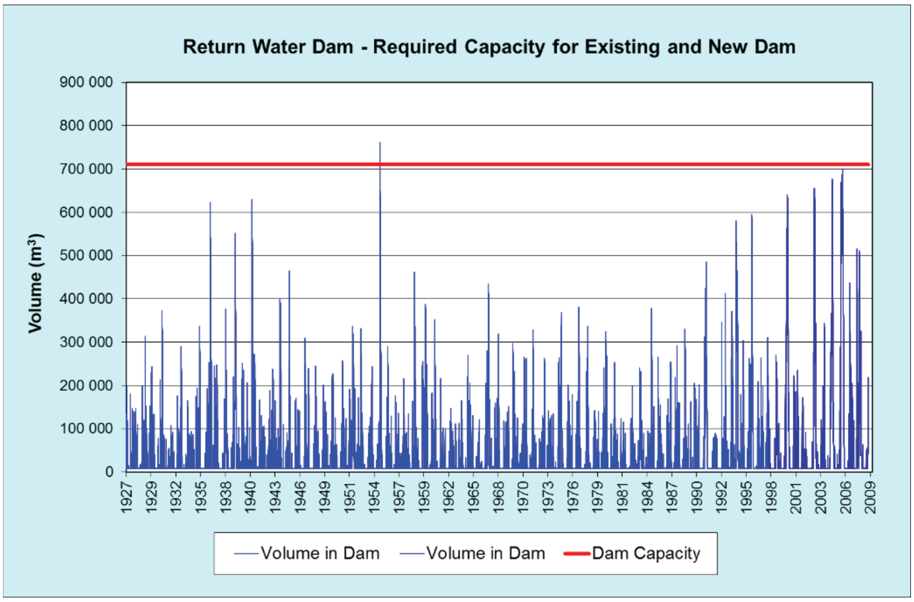


Figure 4-4: Results of Scenario 3

4.6.4 Summary

The required capacities for the Ash Water Return dam is summarised in the table below.

Table 4-2: Required Capacity of AWR

Scenario	Description	Dam Size To Spill Only Once In 95 Year Record (m^3)
1	Current	380 000
2	Only new dam(s) considered. Existing dams assumed rehabilitated	330 000
3	Existing and new dam(s) operated concurrently	710 000

Note that the capacities required for a new AWR is based on the concept design of the new ash dam(s).

Another scenario was conducted for the current ash load produced (i.e. 70% GLF) as discussed in Section 3.4.2. Thus, only AD4.1 & AD4.2 were considered to operate concurrently with AD1-3. This scenario showed that an AWR with a capacity of approximately 550 000m^3 (550 Mℓ) will be required, which will require the AWR embankment to be lifted by 3m. This dam configuration remains feasible but will require that the eastern underdrains be pumped in addition to the western underdrains (refer to Section 5.2).

5. RETURN WATER SYSTEMS

5.1 Ash Water Return Dam

Based on the preliminary findings of the water balance a new AWR will be required to accommodate the new ash dam(s) to ensure the site does not discharge to the environment more than once in 50 years (GN704).

The location next to the East Wing Dam of the existing AWR is a suitable location for the construction of a new AWR or extension of the existing dam due to the fact that the dam is:-

- Situated on natural ground as opposed to backfill thereby reducing the risk of large settlements and possible cracking and failure of the water retaining embankments.
- At one of the low points on site thereby maximising gravity flow of drain, decant and dirty storm water. Note that the whole site cannot drain to this point as discussed in the next section.
- Near the existing AWR pump station allowing re-use of the facility.

Should this site be chosen for the location of a new AWR, the site can also be used as a borrow pit for material to construct the starter walls of the new ash dam(s) to create additional dam capacity as part of excavation.

The figure below shows the location and concept layout of the proposed new AWR.

This site is a suitable option as it is situated on natural ground, as opposed to backfill material, and is in the direct vicinity of the existing pumping station.

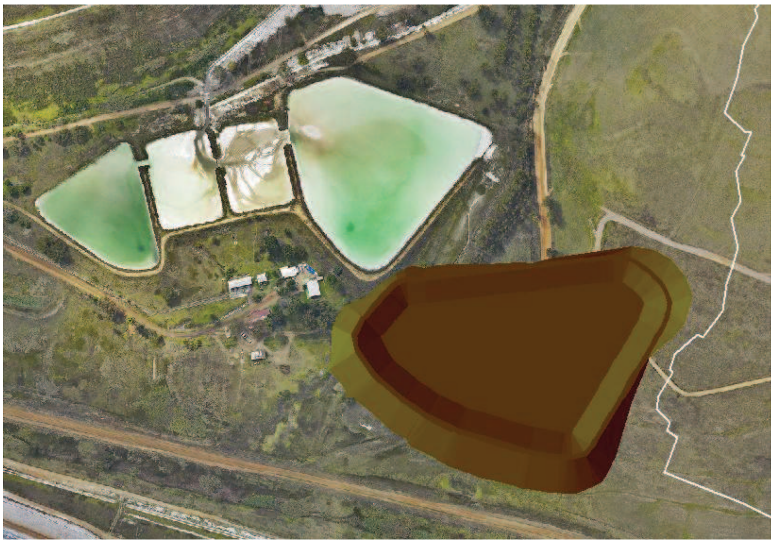


Figure 5-1: Proposed Location of New AWR

Key parameters of the new AWR dam are as follows:-

Crest Width	:	7m
Downstream slope	:	1 (vertical) : 3 (horizontal)
Upstream slope	:	1 (vertical) : 3.5 (horizontal)
Outer Wall height	:	17.2m at 1565 mamsl
Dry Freeboard	:	1.5m
Basin Liner	:	As given under Section 5.3
Volume	:	330 000m^3

A dam with a safety risk is defined as a dam with a wall height exceeding 5m that can store more than 50 000m^3. If either one of the conditions are not met the dam is not considered to be a dam with a safety risk and is not required to be registered with the Dam Safety office of Department of Water Affairs. The new AWR dam will meet both of these requirements and is considered to be a dam with a safety risk.

Dams with a safety risk are categorized in terms of their size and hazard potential as a Category I, II or III. The new AWR Dam will need to be classified to determine the level of control over the safety of the structure that is applicable in terms of GN32543 (2009).

5.2 Transfer Dam, Seepage Sumps and elevated Solution Trenches

The whole site cannot drain under gravity to the AWR dam the following is required to ensure that all impacted water can report to the AWR dam:-

- A transfer dam is provided south of AD4.3 to collect all water from AD4.2, decant water from AD3 and a portion of seepage water from AD4.1. The transfer dam is excavated into the Cut 1 ash fill adjacent to the original starter wall.
- The solution trench at the toe of AD4.1 west of the AWR is elevated to allow part of the storm water from AD4.1 and the decant from Ash Dams 1 and 2 to flow under gravity into the AWR.
- The AD4.1 underdrains east of the AWR will report to a concrete manhole that will be provided with a pump to pump the water to the solution trench mentioned above.

Note that the transfer dams and manholes do not require capacity to ensure compliance but instead need capacity to allow pumping of transfer water to the AWR dam. The transfer dam was provisionally sized to store inflow from the penstock for 24 hrs without pumping. It should be noted that seepage and storm water were not considered in the sizing of the transfer dam. It is recommended that a bund wall is constructed downstream of the dam to prevent a spill by overtopping of the original Cut 1 ash back fill starter wall.

Nominally sized silt traps are provided, the dam will also be provided with internal division walls to allow operation of compartments whilst another is cleaned.

Key parameters of the transfer dam are as follows:-

Crest Width	:	7m
Downstream slope	:	Excavated into backfill – no downstream slope
Upstream slope	:	1 (vertical) : 3.5 (horizontal)
Wall height	:	0m at 1567 mamsl
Dry Freeboard	:	0.8m
Basin Liner	:	As given under Section 5.3
Volume	:	34 000m^3

5.3 Liner System

A single composite liner similar to that of the ash dam is specified for all dams and silt traps with a concrete protection layer consisting of 150mm thick concrete filled geocells to help reduce uplift and possible damage to the geomembrane when the dams are cleaned.

Note that all geomembranes are textured on the side slopes to provide stability.

5.4 Return Water Line and Pump Station

The ash water is pumped to the high level ash water return reservoirs by six pumps. Ash water return pumps 1-4 discharge into a discharge manifold while pumps 5 & 6 have no manifold on the discharge side. The discharge lines from 5 & 6 have 550mm diameter lines compared to that of 305mm of the smaller lines. From the 1-4 pump manifold a 400mm line runs to the high level effluent dams (for emergency use). Under normal conditions either pump 5 or 6 is in service while 3 of the smaller pumps are running to pump water to the Slurry plant / Distribution Box.

The technical data below was assessed and the system will be able to serve AD4 without any modification.

Technical Data

Pumps 1 - 4

Motor	:	261 kW
Amps	:	58.5 (full load rating)
Speed	:	1 400 r/min
Supply	:	3.3 kV
Pump type	:	Centrifugal
Manufacturer	:	APE
Drive	:	Direct coupling
Discharge capacity	:	9 m^3/min
Discharge pressure	:	1050 kPa

Pumps 5 & 6

Motor	:	484 kW
Amps	:	96 (full-load rating)
Speed	:	1482 r/min
Supply	:	3.3 kV
Pump type	:	Centrifugal
Manufacturer	:	Mather and Platt
Drive	:	Direct coupling
Discharge capacity	:	22.5 m^3/min
Discharge pressure	:	900 kPa

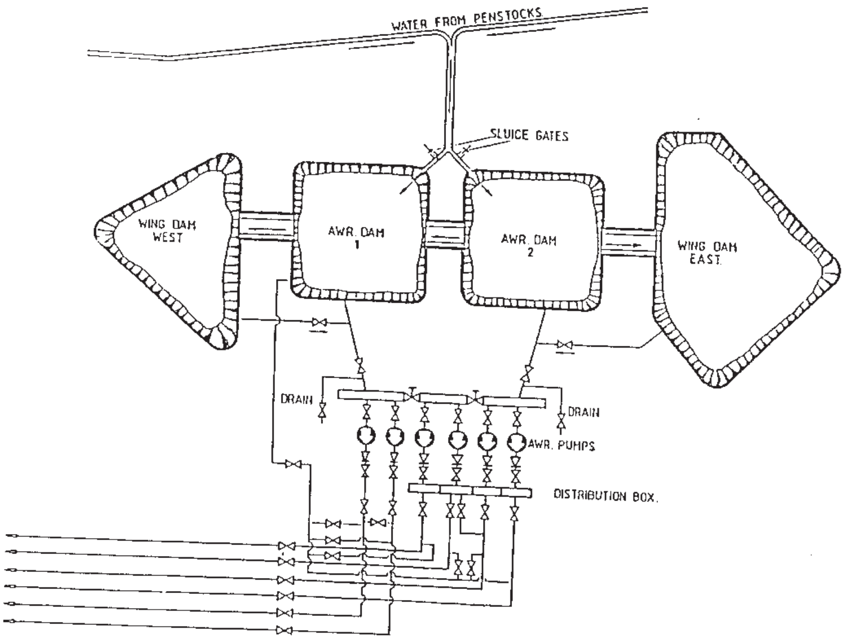


Figure 5-2: Existing Return Water System

5.5 Transfer Dam Pump Station and Pipeline

The transfer dam is nominally sized to fit into the available space between AD4.3 and the backfill. The transfer dam should furthermore pump process and storm water to the AWR dam from AD4.2 and later on AD4.3.

The Transfer dam will require the following pumping arrangement:-

Flow: 480m3/hr

Static head: 20m

Pipe length: 850m

For these requirements 350mm diameter steel pipeline was selected. The associated pipe velocity is 1.5m/s generating a total head of 27m.

6. CLOSURE REQUIREMENTS

6.1 General Description

Concurrent rehabilitation of the outer slopes of the ash dam is recommended to aid in dust suppression. The capping of the ash will consist of a 300mm topsoil layer which will be vegetated.

Topsoil should be stockpiled near AD4, preferably within freehaul distance (i.e. 1km) and it is recommended that the topsoil be placed at a maximum height of 3m and vegetated to maintain some of the soil characteristics that are favourable for plant growth.

All storm water will be diverted off the facility by penstocks in order to prevent ash silt build up over vegetated areas. These bench penstocks provide the opportunity to split clean and dirty run-off but will require two separate sets of penstocks. For this concept it is assumed that all run-off from rehabilitated and un-rehabilitated areas will report to the solution trenches. Post closure the penstock outfalls can be diverted from the solution trenches to the environment.

6.2 Operations

The AWR and transfer dams will remain in passive operation to collect seepage water under gravity. The pump sump that pumps the western AD4.1 underdrain water to the AWR will need to be decommissioned and the underdrain pipes extended into the AWR. As the sumps are 1m lower than the AWR, seepage water will back-up in the finger drains to the level of the AWR dam crest level. This elevation is below the starter wall height and hence no sidewall seepage, associated sloughing or stability concerns are expected.

7. LIFE CYCLE COSTS

7.1 Life Cycle Cost Components

The Life Cycle Costs (LCC) were calculated accounting for:-

(a) Pre-deposition Works

The works include for all civil and mechanical infrastructure to allow first ashing to take place.

(b) Operational Costs

These costs include water, power consumption and provision of an operating contractor to develop the ash dam.

(c) Closure costs

These costs include for provision of topsoil, grassing and bench penstocks to convey clean water to the environment.

Note that all the components are not described here, refer to the summary of costs in Section 7.3, which serves as a list of the actual items that were allowed for each option.

7.2 Assumptions

The following assumptions were made:-

Capital Costs:-

- The construction of the facility will start in 2019 to allow deposition in 2021;
- A nominal discount rate for Eskom of 10.4% (323-09:Eskom economic Evaluation Parameters (July 2015)) was used;
- All civil works and mechanical plant rates were taken from previous projects and are deemed suitable for the study accuracy of 30%;
- Electrical works for the return water pumping and irrigation for grassing and dust suppression are not measured;
- P&G costs were assumed to be 35% of the works, and are based on previous projects.

Closure Costs:-

- Concurrent rehabilitation by topsoil and grassing will take place and was split proportionally over the life of the ash dam.

Operating Costs:-

- Cost of power is 23 c/kWh direct from power station;
- Cost of water is R8/kl from Vaal River; and
- Cost for operating contractor on ash dam is R13/ton.

7.3 Implementation Timelines

The following table summarises the completion dates:-

Table 7-1: Year of completion for each phase

Construction Phase	*Year of Completion*
Ash Dam 4.1	2023
Ash Dam 4.2	2021
Ash Dam 4.3	2025

7.4 Cost Summary

The overall project costs are summarized overleaf. The detailed cost sheets and cash flow are attached in Appendix B.

Table 7-2: Cost Summary

		Present Day Cost	PV at 10,4%
1. CAPITAL COSTS			
Ash Dam 4			
1	PRELIMINARY & GENERAL	R 144 152 006.67	R 90 157 200.56
2	SITE CLEARANCE	R 14 129 140.00	R 8 502 550.89
3	EARTHWORKS	R 52 101 265.00	R 30 196 562.60
4	SURFACE DRAINAGE	R 15 511 040.00	R 10 388 252.34
5	LINING SYSTEM	R 138 710 216.44	R 83 530 743.06
6	LEACHATE COLLECTION	R 117 611 417.00	R 70 856 873.99
7	DECANT SYSTEM	R 24 261 880.29	R 14 684 386.74
8	ASH DEPOSITION SYSTEM	R 47 500 360.00	R 38 194 244.55
9	ROADS AND ACCESS	R 2 037 557.50	R 1 238 387.40
	CONTINGENCY 10%	R 41 186 287.63	R 25 759 200.16
		R 597 201 170.52	R 373 508 402.29
Return Water Dam and Transfer Dam			
1	PRELIMINARY & GENERAL	R 48 476 517.25	R 36 340 347.71
2	SITE CLEARANCE	R 1 669 750.00	R 1 240 921.28
3	EARTHWORKS	R 13 338 090.00	R 9 912 573.56
4	LINING SYSTEM	R 115 842 095.00	R 86 987 489.22
5	OUTLET WORKS & PUMP SYSTEM	R 5 000 000.00	R 3 715 889.44
6	RETURN WATER PIPES	R 2 654 400.00	R 1 972 691.39
	CONTINGENCY 10%	R 13 850 433.50	R 14 016 991.26
		R 200 831 285.75	R 154 186 903.86
SUB-TOTAL		**R 798 032 456.27**	**R 527 695 306.16**

2. OPERATIONAL COSTS			
1	RESIDUE DELIVERY		
	(a) Power	R 22 948 266.36	
2	RESIDUE DISPOSAL		
	(a) Facility Operation & Maintenance	R 906 373 000.00	
	(b) Water Consumption	R 62 804 110.32	
3	RETURN WATER		
	(a) Power	R 2 036 977.31	
SUB-TOTAL		**R 994 162 353.99**	**R 371 696 604.27**

3. CLOSURE COSTS			
1	PRELIMINARY & GENERAL	R 21 126 140.41	R 5 287 533.53
2	CLADDING	R 49 878 279.19	R 12 483 731.93
3	GRASSING	R 10 482 121.98	R 2 623 506.72
	CONTINGENCY 10%	R 6 036 040.12	R 1 510 723.87
SUB-TOTAL		**R 87 522 581.70**	**R 21 905 496.06**

TOTAL		**R 1 879 717 391.97**	**R 921 297 406.49**

8. IMPLEMENTATION PROGRAMME

8.1 General Summary

An approximate generic programme of construction for a wet system is presented overleaf.

Note the following:-

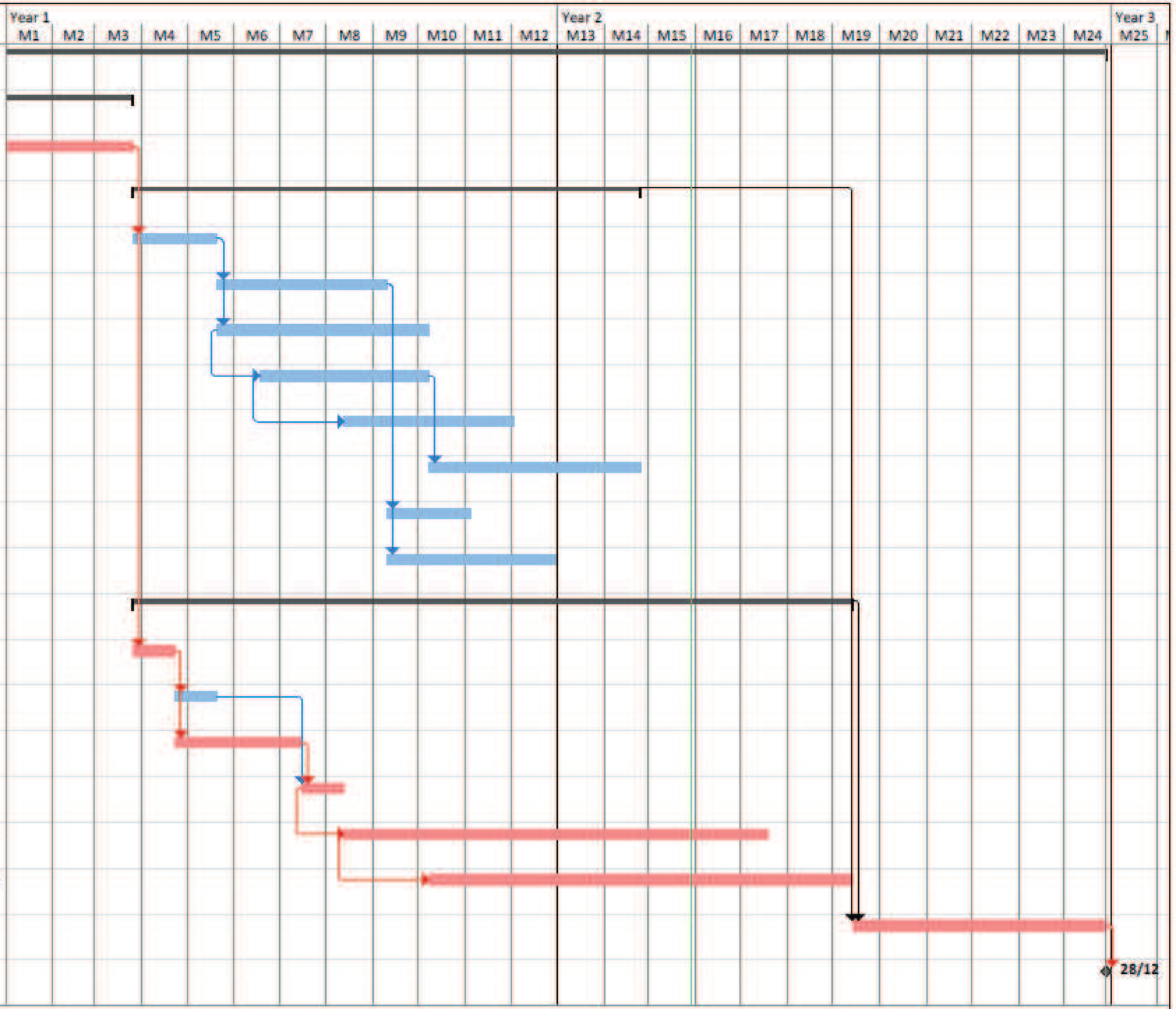
- AD1-3 will be stepped in and lifted higher to provide sufficient deposition space to allow AD4 to be constructed.
- AD4.2 needs to be commissioned in 2021 to reduce the deposition on AD1-3
- The AWR and transfer dam need to be commissioned with AD4.2 to receive the AD1-3 and AD4.2 return water
- Once the new AWR is commissioned the existing AWR can be decommissioned to allow construction of AD4.1, which overlies the existing AWR dams. Commissioning of AD4.1 is 2023
- AD4.3 must be commissioned in 2025 at which point in time AD1-3 will be decommissioned.

8.2 Design, Licence and Construction Time Requirements

The program overleaf shows the durations that should be allowed for the various technical activities that is required to develop the first phase of AD4 which consists of AD4.2, the AWR and the Transfer Dam. The program does not allow for Eskom processes that includes time for procurement, decision making and project stage approvals. Licencing requirements must also be defined by the various Subject Matter Experts.

Table 8-1: Implementation Programme: AD.2, AWR and Transfer Dam

ID		Task Mode	Task Name	Duration	Predecessors
1			**Construction AD 4.2 & AWR**	**26 mons**	
2			**General**	**3 mons**	
3			Establishment	3 mons	
4			**Ash Dam**	**12 mons**	
5			Site clearing	2 mons	3
6			Excavation and starterwalls	4 mons	5
7			Underdrains	5 mons	5
8			HDPE & clay liner	4 mons	7SS+1 mon
9			BBA cover layer	4 mons	8SS+2 mons
10			Leachate drains	5 mons	8
11			Delivery piping	2 mons	6
12			Canals and roads	4 mons	6
13			**AWR & Transfer Dam**	**17 mons**	
14			Site clearing	1 mon	3
15			Excavation	1 mon	14
16			Dam walls	3 mons	14
17			Underdrains	1 mon	15,16
18			HDPE & clay liner	10 mons	17SS+1 mon
19			Concrete cover layer	10 mons	18SS+2 mons
20			Delays (Rain and holidays)	6 mons	4,13
21			**Commisioning**	0 mons	20



9. CONCLUSIONS AND RECOMMENDATIONS

This report demonstrates that AD4 can be constructed in phases with the following benefits:-

- Initial Phases AD4.1 and 4.2 will be on natural ground thereby postponing construction activities on backfill to start 2023 with the construction of AD4.3.
- The trial embankment that will be required to investigate pit backfill settlement is only needed for AD4.3 and hence some time becomes available to construct the embankment and monitor settlement while AD4.1 and AD4.2 is being developed.
- Avoiding development on backfill is possible provided that AD1-3 can be operated for the remainder of the life of the power station. This will require approval by the authorities and it is recommended that this aspect be discussed as a matter of priority.
- A new AWR can be constructed while maintaining the AWR pump system.
- AWR pump system is adequate and will serve AD4 without modification.
- A new Transfer Dam will be required to pump water from AD4.2 to the AWR complete with pump station and pipeline.
- The slurry booster pump station upgrade needs to be assessed. This report provides a preliminary assessment that the pump station may be sufficient for the PFA stream and an alternative arrangement is required for the BBA.
- Major optimizations that should be considered during basic engineering is a trade-off of the construction of a new AWR pump station at a lower position to avoid the construction of the transfer dam.
- The use of cycloning to dewater and de-silt the BBA should be considered in order to optimise the specified leachate collection layer.
- Current sluicing from a single point does not allow for desiccation of the ash as the areas near the sluice point are always wet. A complete ring main system with multiple deposition points will therefore be required to distribute the ash around the dam perimeter without wetting inactive areas.

Document source: C:\Alljobs\E821_Eskom_Kriel_Ash_Dam_4\REP\E821-00_REP-001_r0_JW044-16_KrielAshDam4.docx
Document template: Normal.dotm

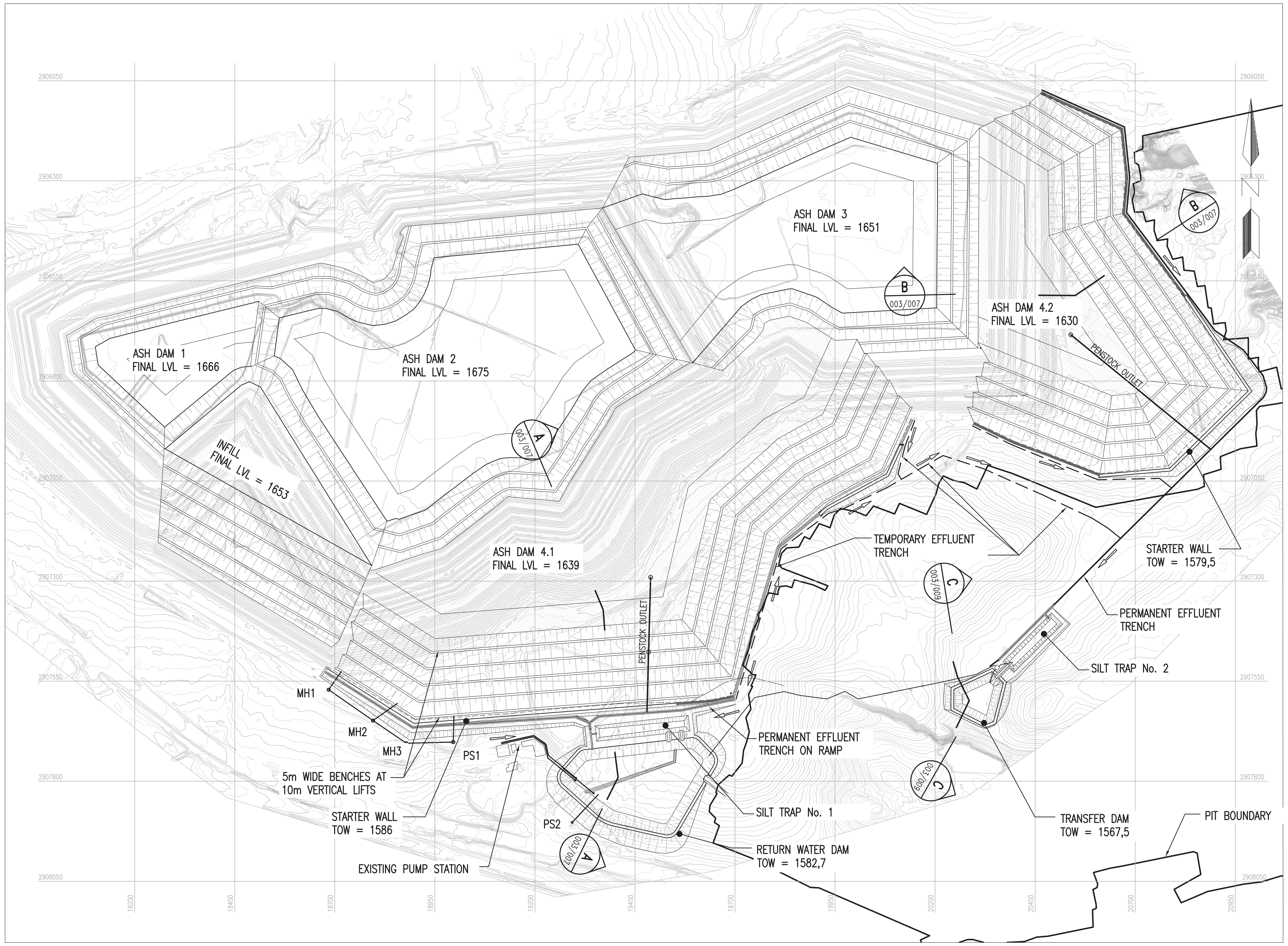
ESKOM HOLDINGS LTD

KRIEL POWER STATION
ASH DAM 4

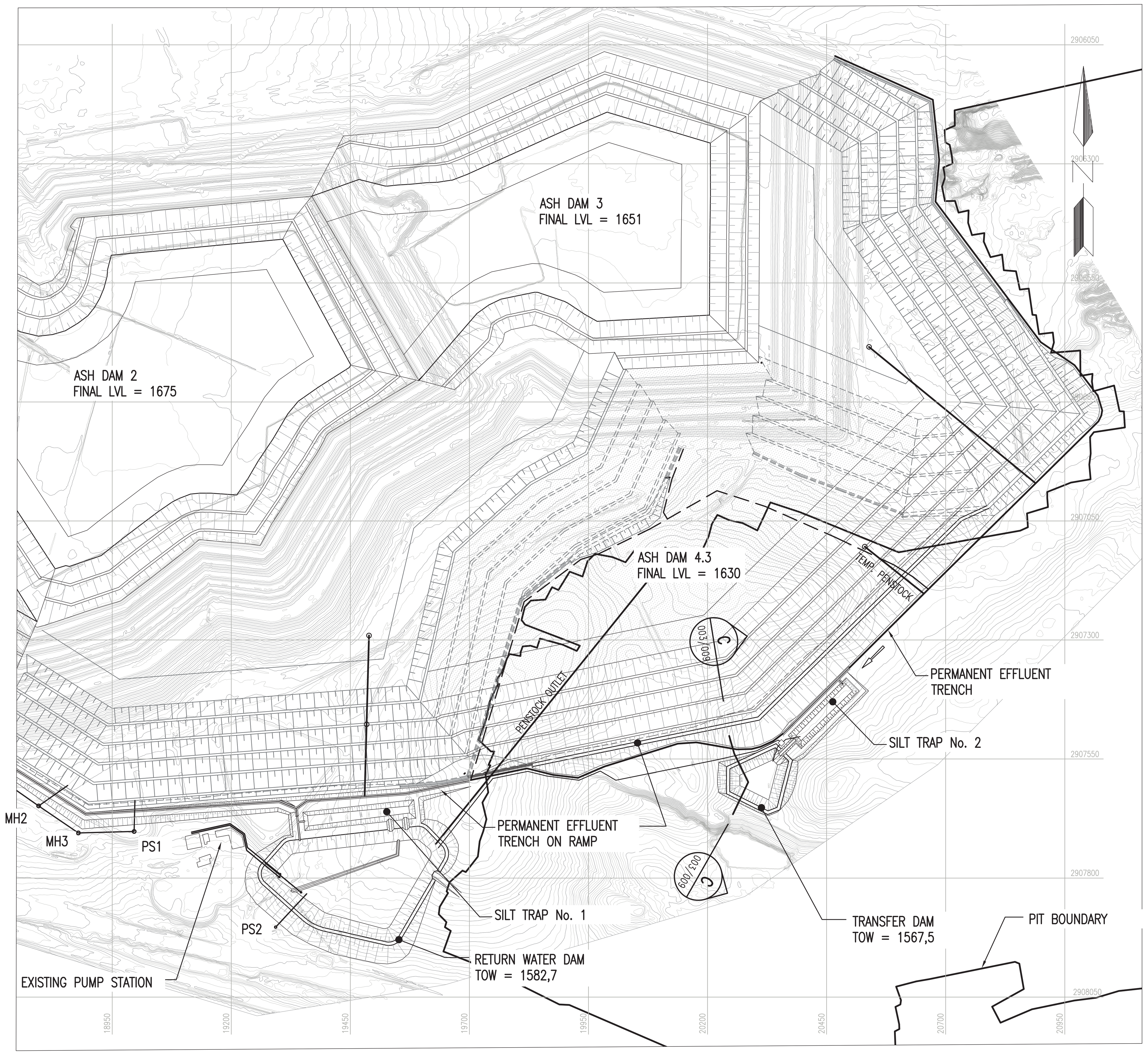
Report: JW044/16/E821 – Rev 0

APPENDIX A

DRAWINGS



LAYOUT AD4.1 & AD4.2
1:5000



LAYOUT AD4.3
1:5000

DRAWING LIST

E821-00-001 AD4.1, 4.2 & 4.3 – GENERAL ARRANGEMENT, FINAL ASH DAM (THIS DRAWING)
E821-00-002 AD4.1, 4.2 & 4.3 – GENERAL ARRANGEMENT, PRE DEPOSITION WORKS
E821-00-003 AD4.1, 4.2 & 4.3 – FINAL ASH DAM, TYPICAL SECTIONS & DETAILS
E821-00-004 AD4.1 – PRE DEPOSITION WORK , LAYOUT
E821-00-005 AD4.2 – PRE DEPOSITION WORKS, LAYOUT
E821-00-006 AD4.3 – PRE DEPOSITION WORKS, LAYOUT
E821-00-007 AD4.1, 4.2 & 4.3 – PRE DEPOSITION WORKS, TYPICAL SECTIONS & DETAILS
E821-00-008 RETURN WATER DAM – LAYOUT & DETAIL
E821-00-009 TRANSFER DAM – LAYOUT & DETAILS
E821-00-010 REHABILITATION DETAILS – BENCH PENSTOCK
E821-00-011 DELIVERY SYSTEM – ø300 STEEL PIPELINE
E821-00-012 DELIVERY SYSTEM – ø500 HDPE PIPELINE

Rev.	Revision	Name	Date	Approved
B	ISSUED FOR INFORMATION ONLY	MK	18.4.16	AB
A	ISSUED FOR APPROVAL	MK	17.3.16	AB

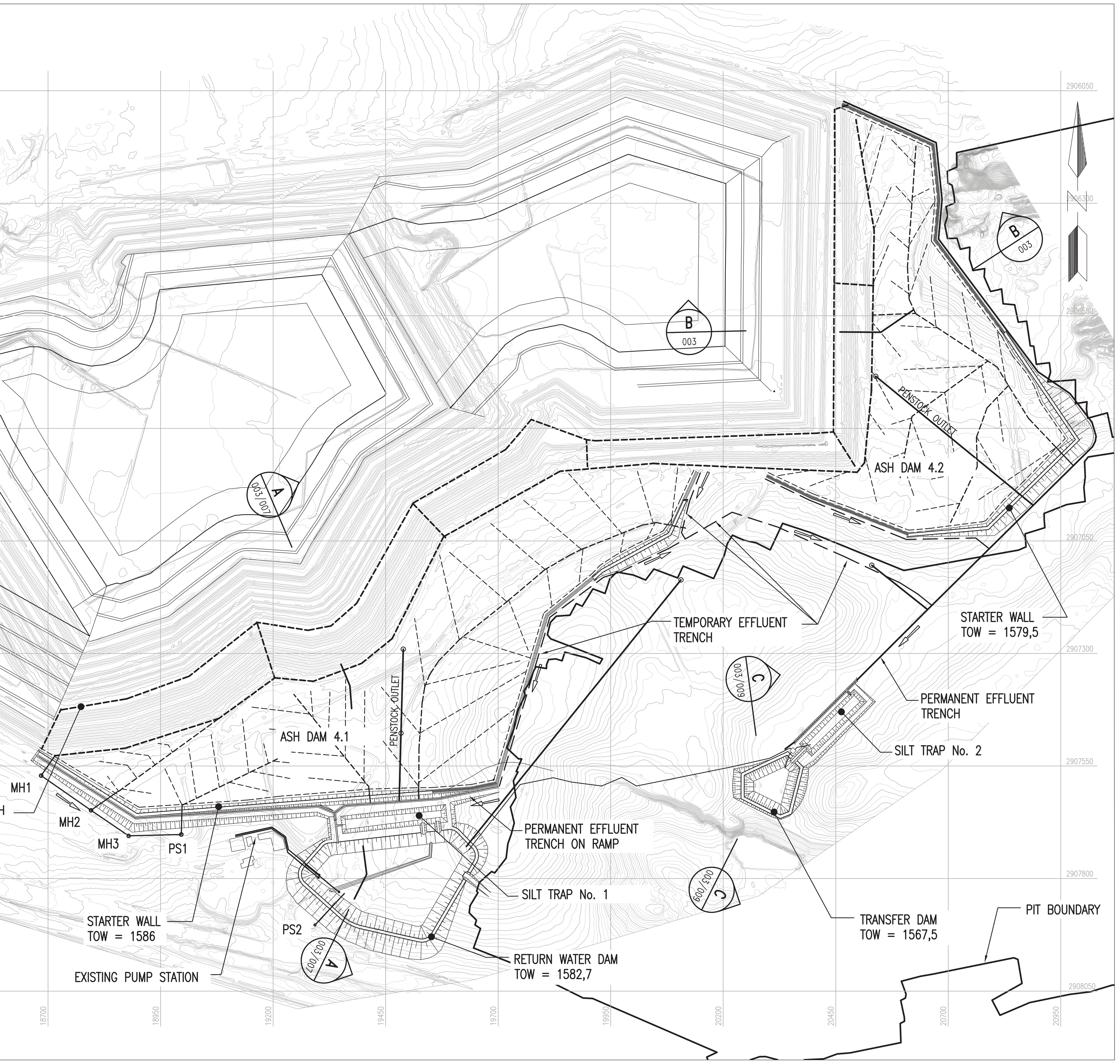
Reference Drawings

Jones & Wagener
Engineering & Environmental Consultants
Box 1434 Rivonia 2128 South Africa
Tel. (011) 519-0200
Fax. (011) 519-0201
Email post@jaws.co.za

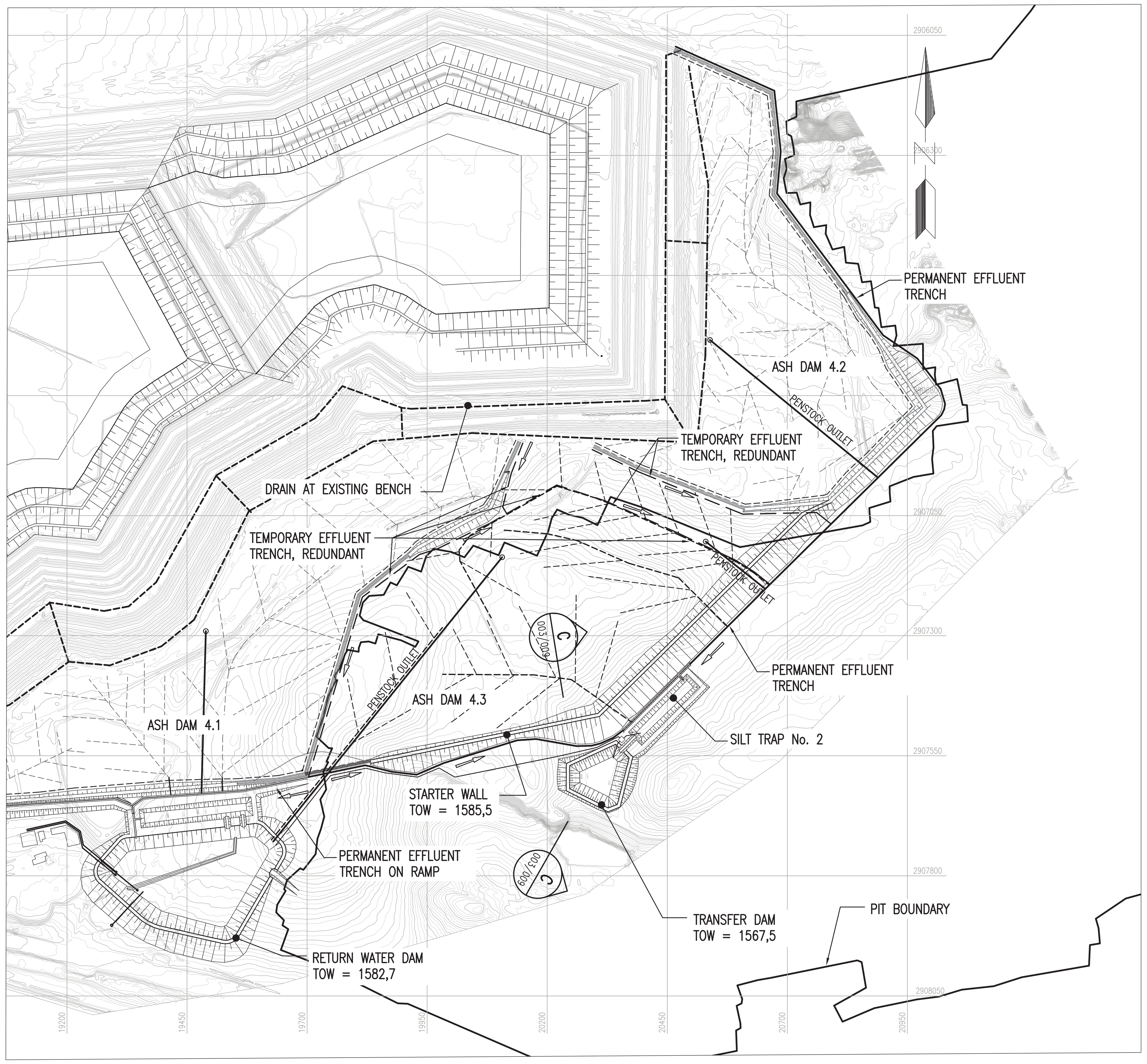
Drawn	M KAISER	26.2.16
Designed	D de WET	FEB 16
Checked	D de WET	15.3.16
Pr. Eng. Approval	A BAIN	17.3.16
Signature		

Client: ESKOM
Project: KRIEL POWER STATION
ASH DAM 4.1, 4.2 & 4.3
GENERAL ARRANGEMENT
FINAL ASH DAM

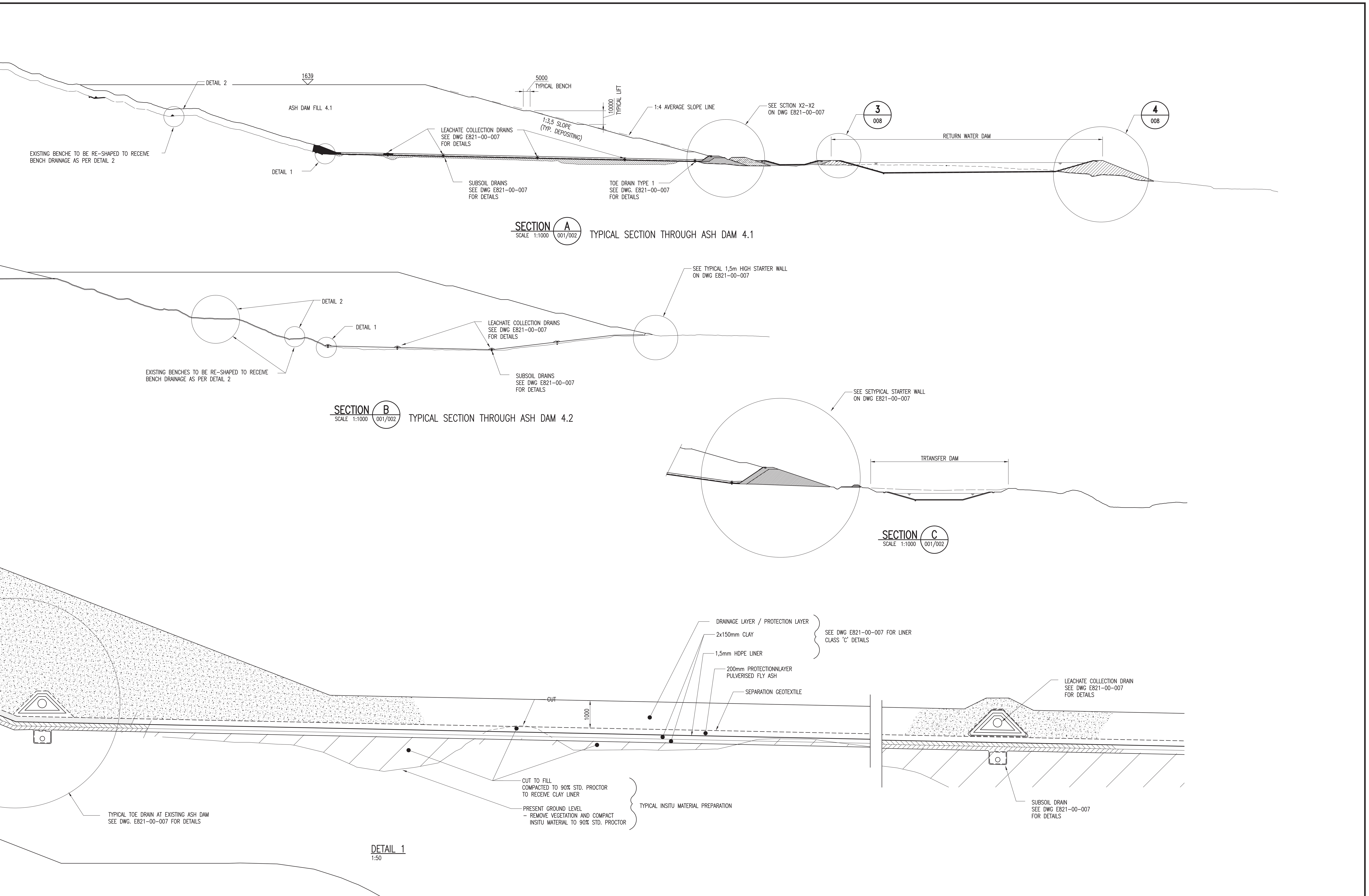
Scale: AS SHOWN — A0
Drawing No: E821-00-001
Revision: B

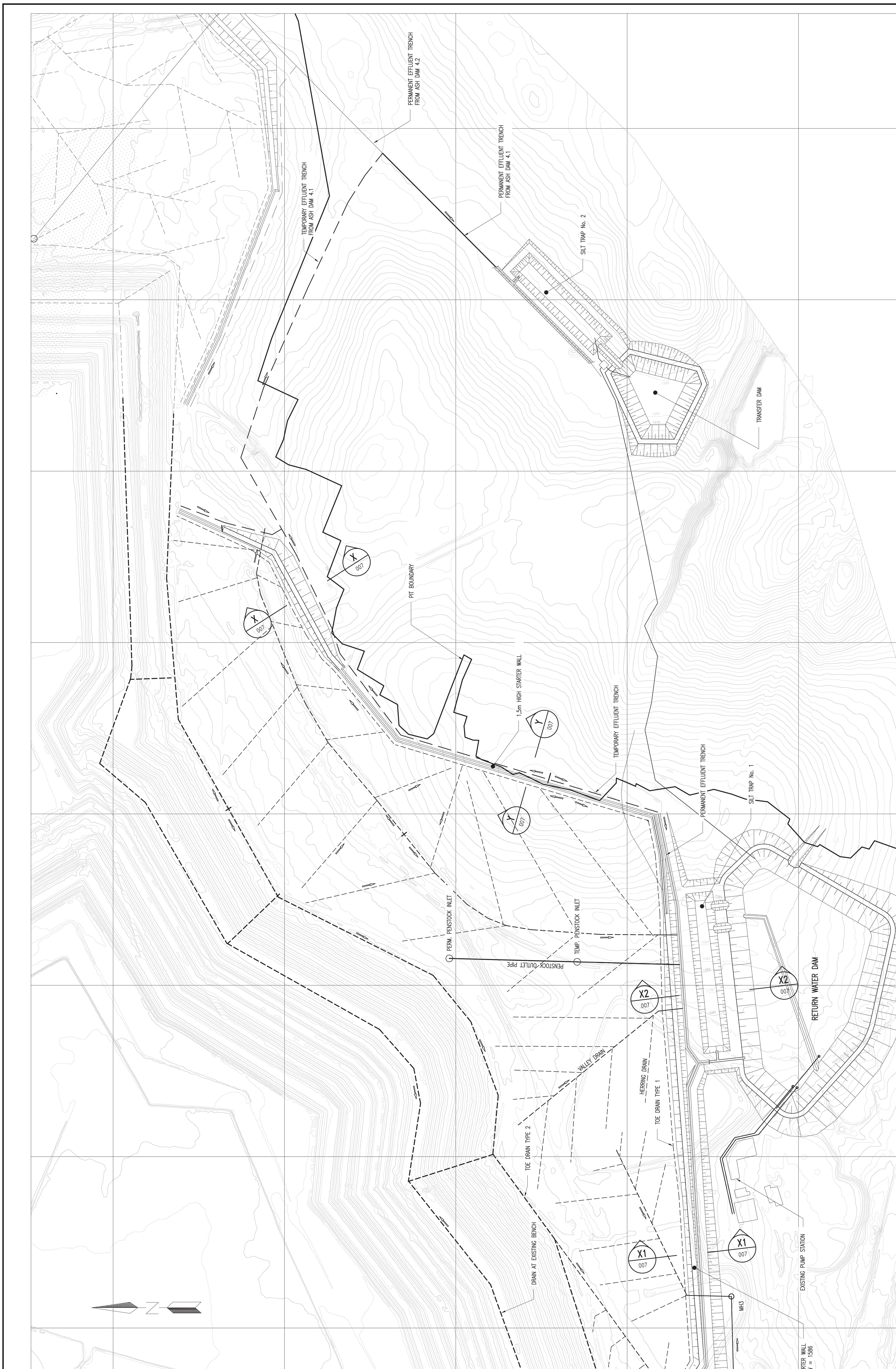


LAYOUT AD4.1 & AD4.2
1:5000

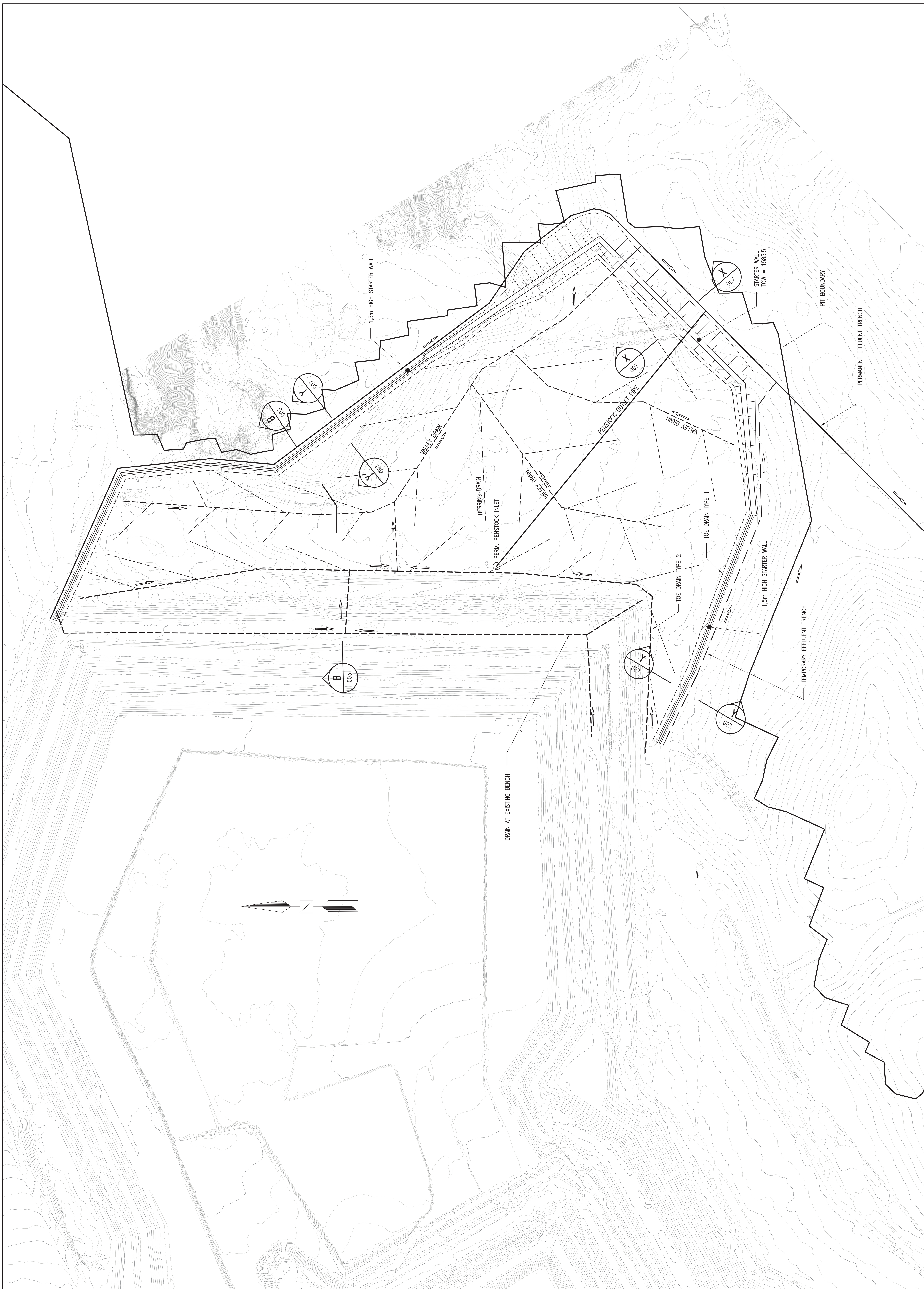


LAYOUT AD4.3
1:5000


DETAIL 2
1639
ASH DAM FILL 4.1
5000
TYPICAL BENCH
10000
TYPICAL LIFT
1:4 AVERAGE SLOPE LINE
SEE SCTION X2-X2
ON DWG E821-00-007
3
008
4
008
RETURN WATER DAM
1:3,5 SLOPE
(TYP. DEPOSITING)
LEACHATE COLLECTION DRAINS
SEE DWG E821-00-007
FOR DETAILS
EXISTING BENCHE TO BE RE-SHAPED TO RECEIVE
BENCH DRAINAGE AS PER DETAIL 2
DETAIL 1
SUBSOIL DRAINS
SEE DWG E821-00-007
FOR DETAILS
TOE DRAIN TYPE 1
SEE DWG. E821-00-007
FOR DETAILS
SECTION A
SCALE 1:1000
001/002
TYPICAL SECTION THROUGH ASH DAM 4.1
SEE TYPICAL 1,5m HIGH STARTER WALL
ON DWG E821-00-007
DETAIL 2
DETAIL 1
LEACHATE COLLECTION DRAINS
SEE DWG E821-00-007
FOR DETAILS
EXISTING BENCHES TO BE RE-SHAPED TO RECEIVE
BENCH DRAINAGE AS PER DETAIL 2
SUBSOIL DRAINS
SEE DWG E821-00-007
FOR DETAILS
SECTION B
SCALE 1:1000
001/002
TYPICAL SECTION THROUGH ASH DAM 4.2
SEE SETYPICAL STARTER WALL
ON DWG E821-00-007
TRTANSFER DAM
SECTION C
SCALE 1:1000
001/002
DRAINAGE LAYER / PROTECTION LAYER
2x150mm CLAY
1,5mm HDPE LINER
SEE DWG E821-00-007 FOR LINER
CLASS 'C' DETAILS
200mm PROTECTIONNLAYER
PULVERISED FLY ASH
SEPARATION GEOTEXTILE
LEACHATE COLLECTION DRAIN
SEE DWG E821-00-007
FOR DETAILS
CUT
1000
CUT TO FILL
COMPACTED TO 90% STD. PROCTOR
TO RECEIVE CLAY LINER
PRESENT GROUND LEVEL
- REMOVE VEGETATION AND COMPACT
INSITU MATERIAL TO 90% STD. PROCTOR
TYPICAL INSITU MATERIAL PREPARATION
SUBSOIL DRAIN
SEE DWG E821-00-007
FOR DETAILS
TYPICAL TOE DRAIN AT EXISTING ASH DAM
SEE DWG. E821-00-007 FOR DETAILS
DETAIL 1
1:50



PERMANENT EFFLUENT TRENCH FROM ASH DAM 4.2
PERMANENT EFFLUENT TRENCH FROM ASH DAM 4.1
TEMPORARY EFFLUENT TRENCH FROM ASH DAM 4.1
SILT TRAP No. 2
TRANSFER DAM
X 007
PIT BOUNDARY
1.5m HIGH STARTER WALL
Y 007
TEMPORARY EFFLUENT TRENCH
PERMANENT EFFLUENT TRENCH
SILT TRAP No. 1
PERM. PENSTOCK INLET
TEMP. PENSTOCK INLET
PENSTOCK OUTLET PIPE
X2 007
RETURN WATER DAM
VALLEY DRAIN
HERRING DRAIN
TOE DRAIN TYPE 1
TOE DRAIN TYPE 2
DRAIN AT EXISTING BENCH
X1 007
EXISTING PUMP STATION
MH3
N



1.5m HIGH STARTER WALL
STARTER WALL
TOW = 1585.5
PIT BOUNDARY
PERMANENT EFFLUENT TRENCH
X
007
Y
007
B
003
VALLEY DRAIN
PENSTOCK OUTLET PIPE
HERRING DRAIN
PERM. PENSTOCK INLET
TOE DRAIN TYPE 1
TOE DRAIN TYPE 2
1.5m HIGH STARTER WALL
TEMPORARY EFFLUENT TRENCH
DRAIN AT EXISTING BENCH
N

LAYOUT AD4.3
1:2000

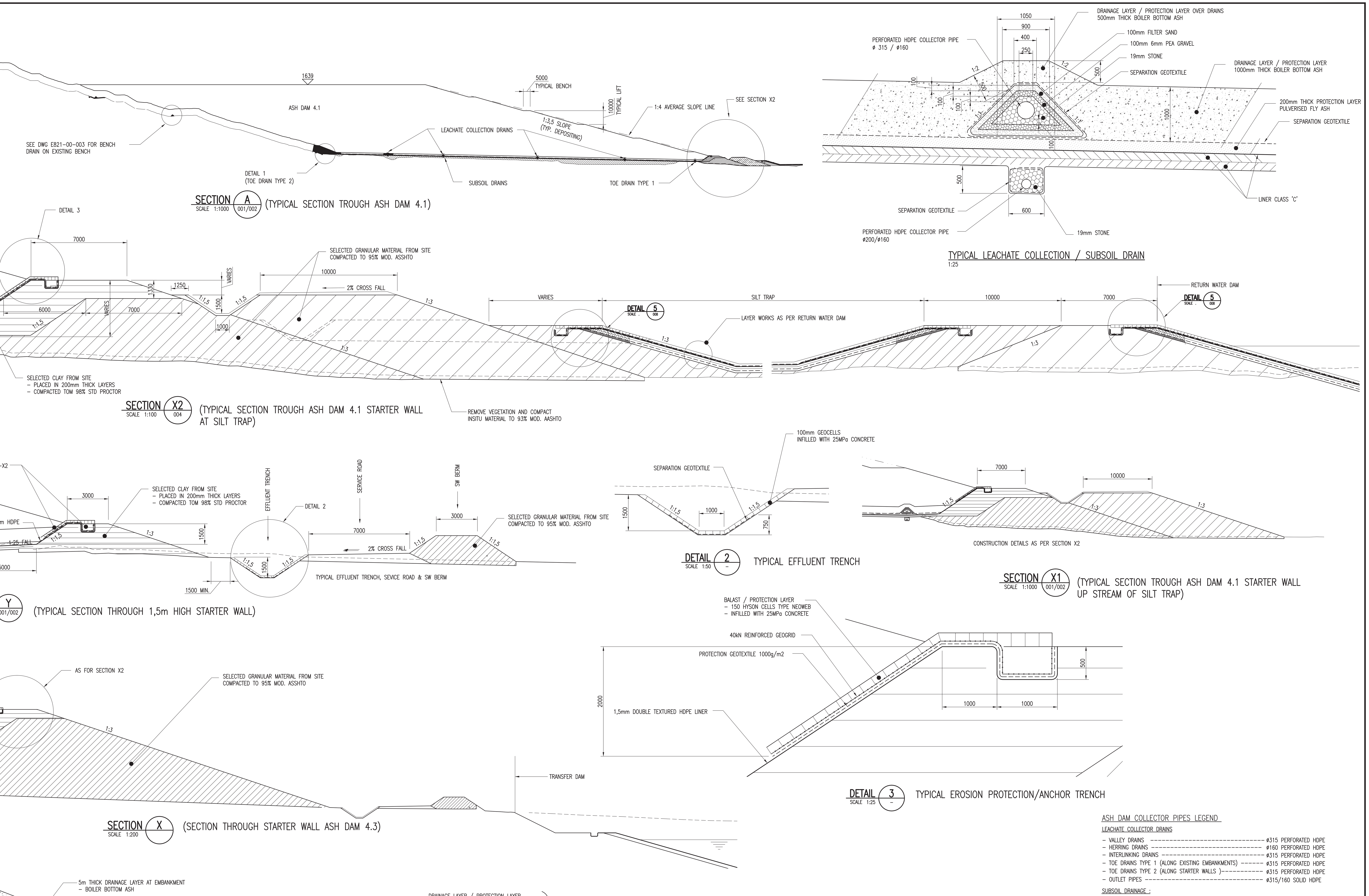
ASH DAM COLLECTOR PIPES LEGEND

LEACHATE COLLECTOR DRAINS

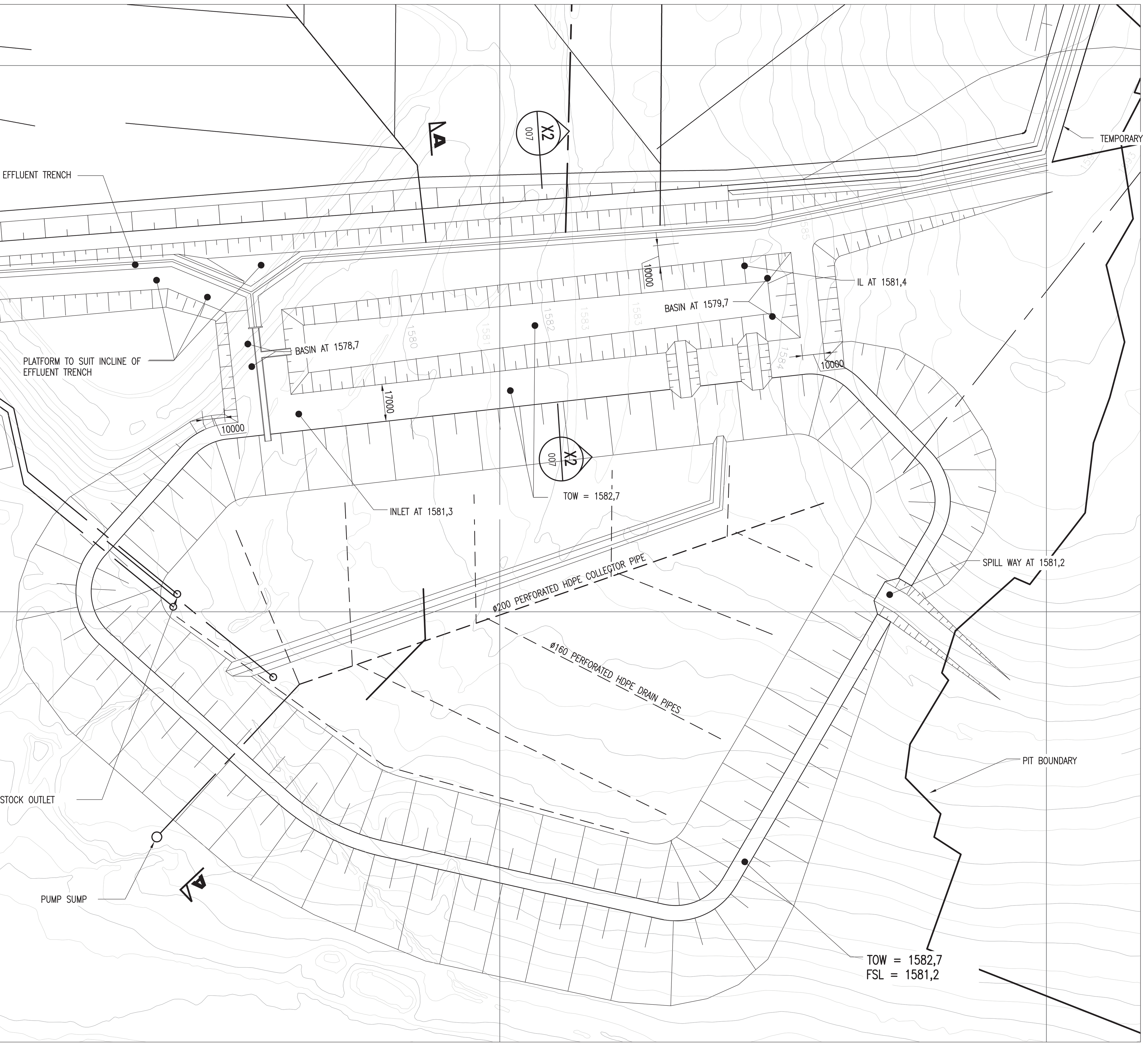
- VALLEY DRAINS ---------------------------------- ø315 PERFORATED HDPE
- HERRING DRAINS --------------------------------- ø160 PERFORATED HDPE
- INTERLINKING DRAINS ---------------------------- ø315 PERFORATED HDPE
- TOE DRAINS TYPE 1 (ALONG EXISTING EMBANKMENTS) ------- ø315 PERFORATED HDPE
- TOE DRAINS TYPE 2 (ALONG STARTER WALLS)------------ ø315 PERFORATED HDPE
- OUTLET PIPES ----------------------------------- ø315/160 SOLID HDPE

SUBSOIL DRAINAGE :

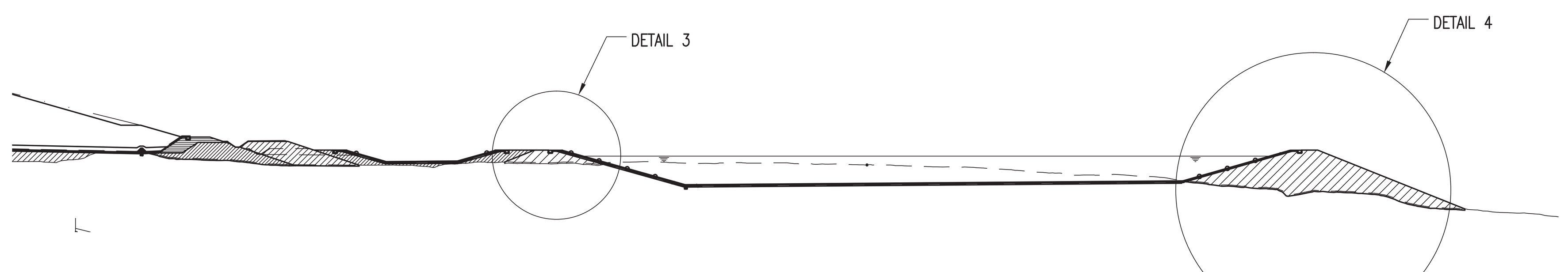
- VALLEY DRAINS ---------------------------------- ø200 PERFORATED HDPE
- HERRING DRAINS --------------------------------- ø160 PERFORATED HDPE
- INTERLINKING DRAINS ---------------------------- ø200 PERFORATED HDPE
- TOE DRAINS TYPE 1 (ALONG STARTER WALLS) ------- ø200 PERFORATED HDPE
- TOE DRAINS TYPE 2 (ALONG EXISTING EMBANKMENT) -------- ø200 PERFORATED HDPE
- OUTLET PIPES ----------------------------------- ø200/160 SOLID HDPE



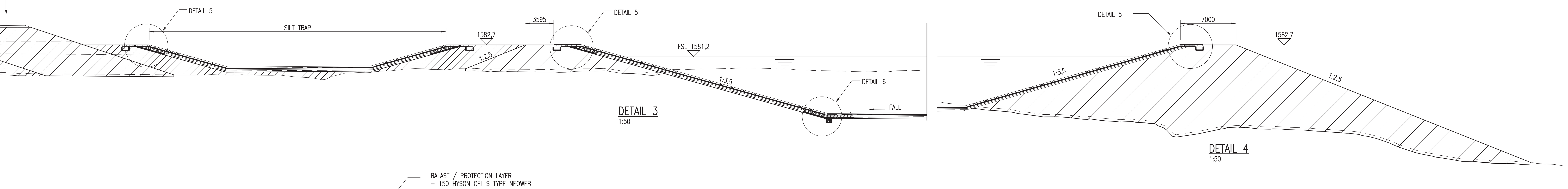
SEE DWG E821-00-003 FOR BENCH DRAIN ON EXISTING BENCH
ASH DAM 4.1
1639
5000 TYPICAL BENCH
10000 TYPICAL LIFT
1:3,5 SLOPE (TYP. DEPOSITING)
1:4 AVERAGE SLOPE LINE
SEE SECTION X2
LEACHATE COLLECTION DRAINS
DETAIL 1 (TOE DRAIN TYPE 2)
SUBSOIL DRAINS
TOE DRAIN TYPE 1
SECTION A SCALE 1:1000 001/002 (TYPICAL SECTION TROUGH ASH DAM 4.1)
PERFORATED HDPE COLLECTOR PIPE ø 315 / ø160
DRAINAGE LAYER / PROTECTION LAYER OVER DRAINS 500mm THICK BOILER BOTTOM ASH
100mm FILTER SAND
100mm 6mm PEA GRAVEL
19mm STONE
SEPARATION GEOTEXTILE
DRAINAGE LAYER / PROTECTION LAYER 1000mm THICK BOILER BOTTOM ASH
200mm THICK PROTECTION LAYER PULVERISED FLY ASH
SEPARATION GEOTEXTILE
LINER CLASS 'C'
SEPARATION GEOTEXTILE
PERFORATED HDPE COLLECTOR PIPE ø200/ø160
19mm STONE
TYPICAL LEACHATE COLLECTION / SUBSOIL DRAIN 1:25
DETAIL 3
SELECTED GRANULAR MATERIAL FROM SITE COMPACTED TO 95% MOD. ASSHTO
2% CROSS FALL
VARIES
SILT TRAP
RETURN WATER DAM
DETAIL 5
LAYER WORKS AS PER RETURN WATER DAM
SELECTED CLAY FROM SITE
– PLACED IN 200mm THICK LAYERS
– COMPACTED TOM 98% STD PROCTOR
SECTION X2 SCALE 1:100 004 (TYPICAL SECTION TROUGH ASH DAM 4.1 STARTER WALL AT SILT TRAP)
REMOVE VEGETATION AND COMPACT INSITU MATERIAL TO 93% MOD. AASHTO
100mm GEOCELLS INFILLED WITH 25MPa CONCRETE
SEPARATION GEOTEXTILE
DETAIL 2 SCALE 1:50 TYPICAL EFFLUENT TRENCH
CONSTRUCTION DETAILS AS PER SECTION X2
SECTION X1 SCALE 1:1000 001/002 (TYPICAL SECTION TROUGH ASH DAM 4.1 STARTER WALL UP STREAM OF SILT TRAP)
EFFLUENT TRENCH
SERVICE ROAD
SW BERM
DETAIL 2
SELECTED GRANULAR MATERIAL FROM SITE COMPACTED TO 95% MOD. ASSHTO
TYPICAL EFFLUENT TRENCH, SEVICE ROAD & SW BERM
1500 MIN.
Y 001/002 (TYPICAL SECTION THROUGH 1,5m HIGH STARTER WALL)
BALAST / PROTECTION LAYER
– 150 HYSON CELLS TYPE NEOWEB
– INFILLED WITH 25MPa CONCRETE
40kN REINFORCED GEOGRID
PROTECTION GEOTEXTILE 1000g/m2
1,5mm DOUBLE TEXTURED HDPE LINER
DETAIL 3 SCALE 1:25 TYPICAL EROSION PROTECTION/ANCHOR TRENCH
AS FOR SECTION X2
TRANSFER DAM
SECTION X SCALE 1:200 (SECTION THROUGH STARTER WALL ASH DAM 4.3)
5m THICK DRAINAGE LAYER AT EMBANKMENT
– BOILER BOTTOM ASH
ASH DAM COLLECTOR PIPES LEGEND
LEACHATE COLLECTOR DRAINS
– VALLEY DRAINS ø315 PERFORATED HDPE
– HERRING DRAINS ø160 PERFORATED HDPE
– INTERLINKING DRAINS ø315 PERFORATED HDPE
– TOE DRAINS TYPE 1 (ALONG EXISTING EMBANKMENTS) ø315 PERFORATED HDPE
– TOE DRAINS TYPE 2 (ALONG STARTER WALLS) ø315 PERFORATED HDPE
– OUTLET PIPES ø315/160 SOLID HDPE
SUBSOIL DRAINAGE :



LAYOUT 1:1000



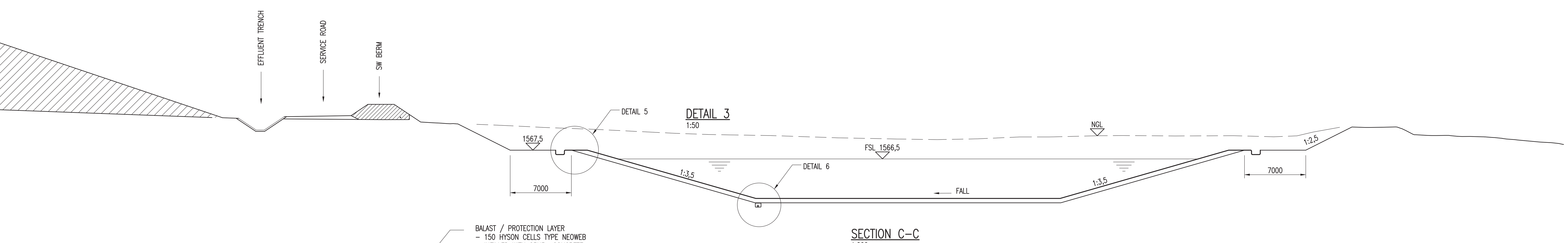
SECTION A–A
1:1000



DETAIL 3
1:50

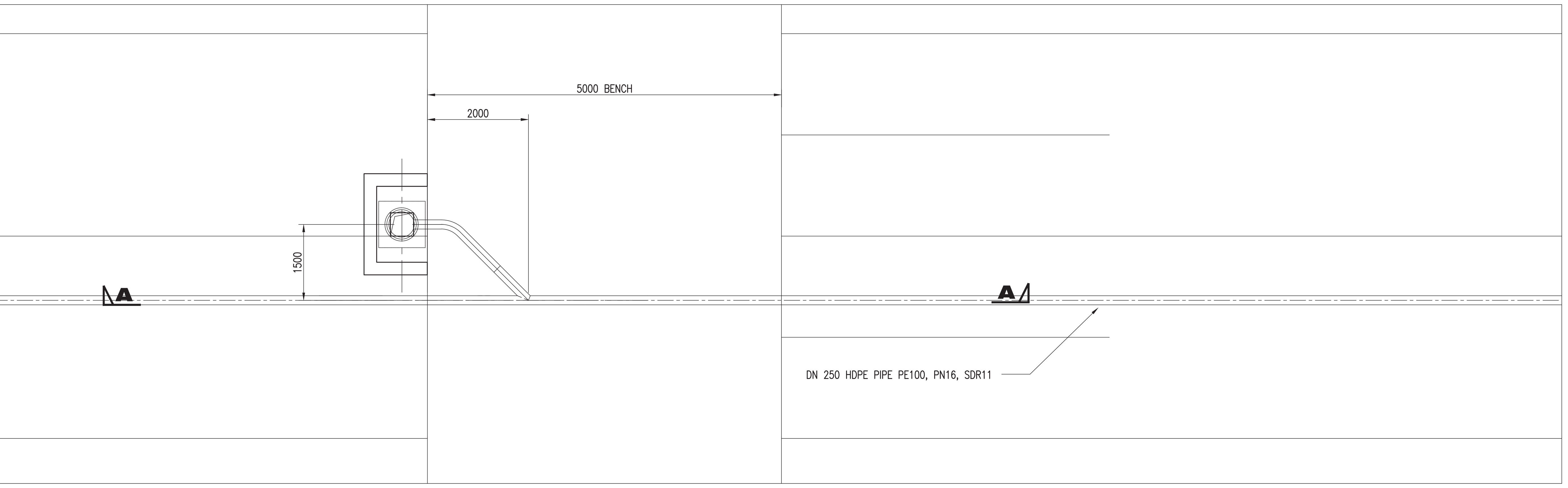
DETAIL 4
1:50

LAYOUT 1:1000

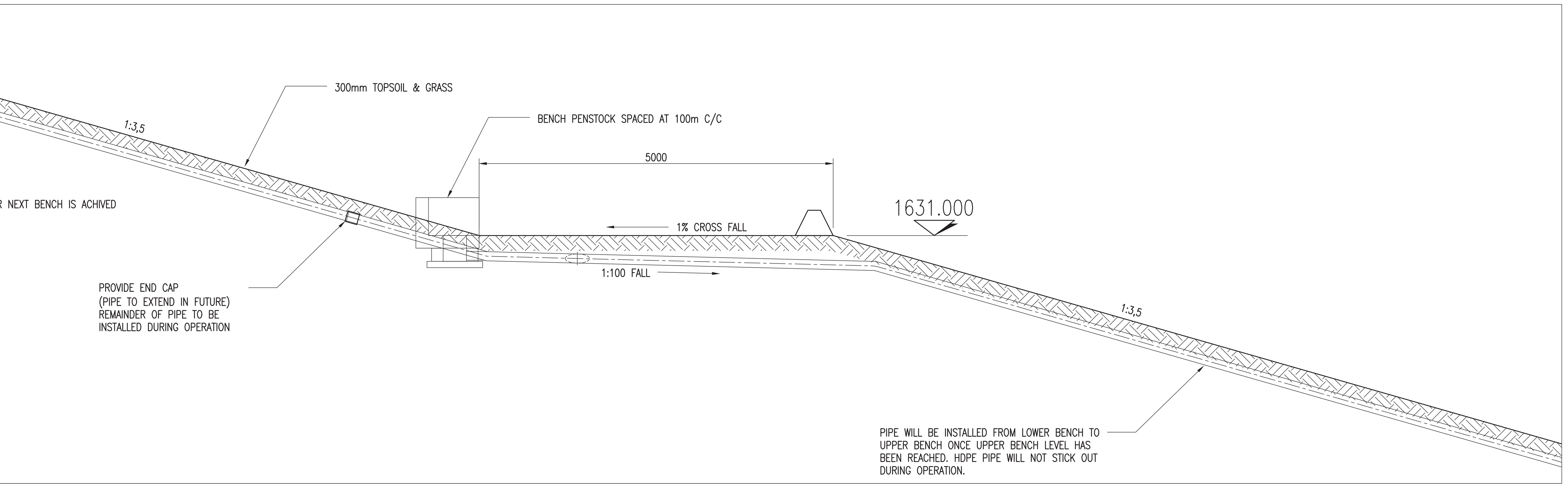


DETAIL 3
1:50

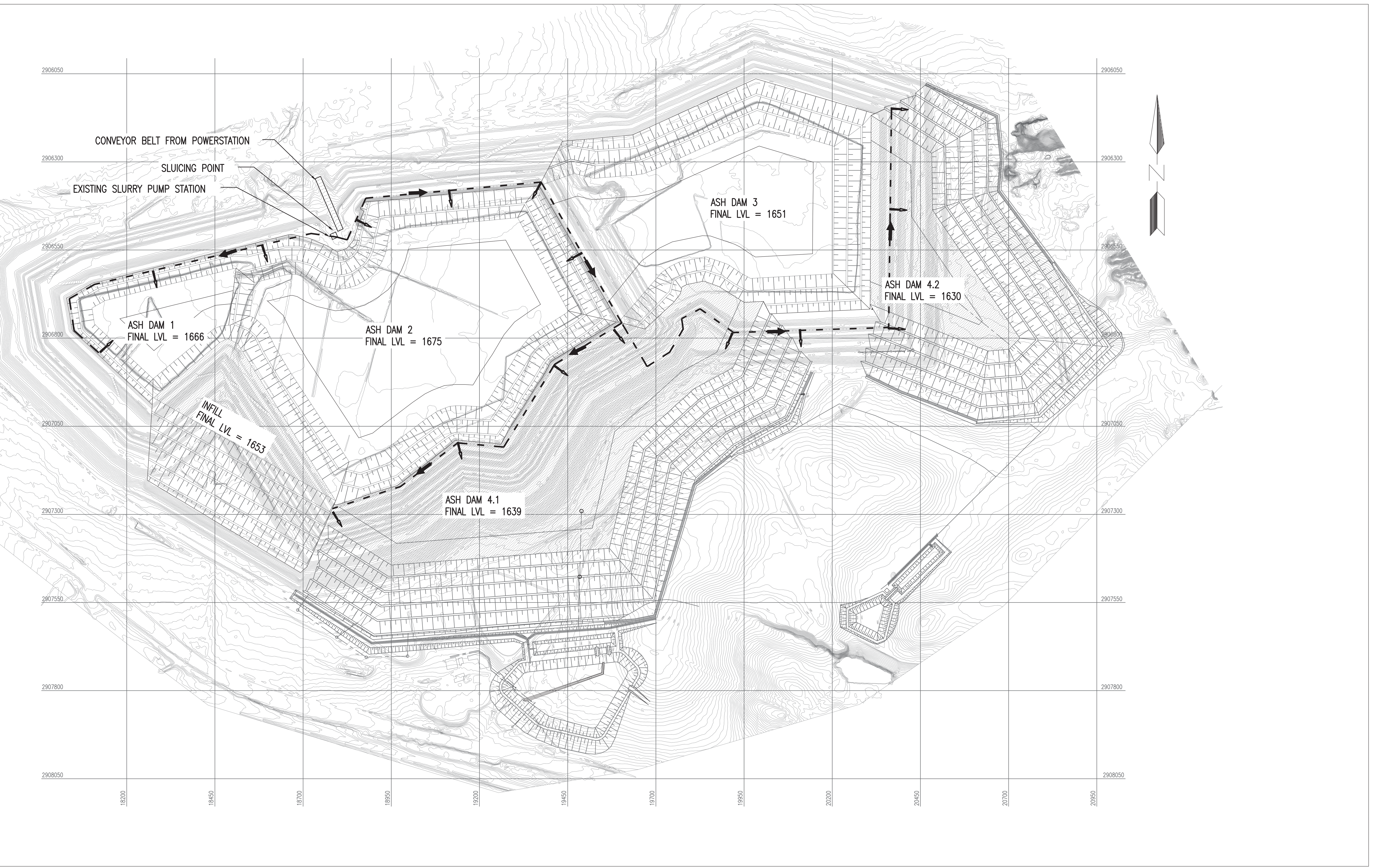
SECTION C–C



TYPICAL PLAN SHOWING BENCH PENSTOCK
1:50



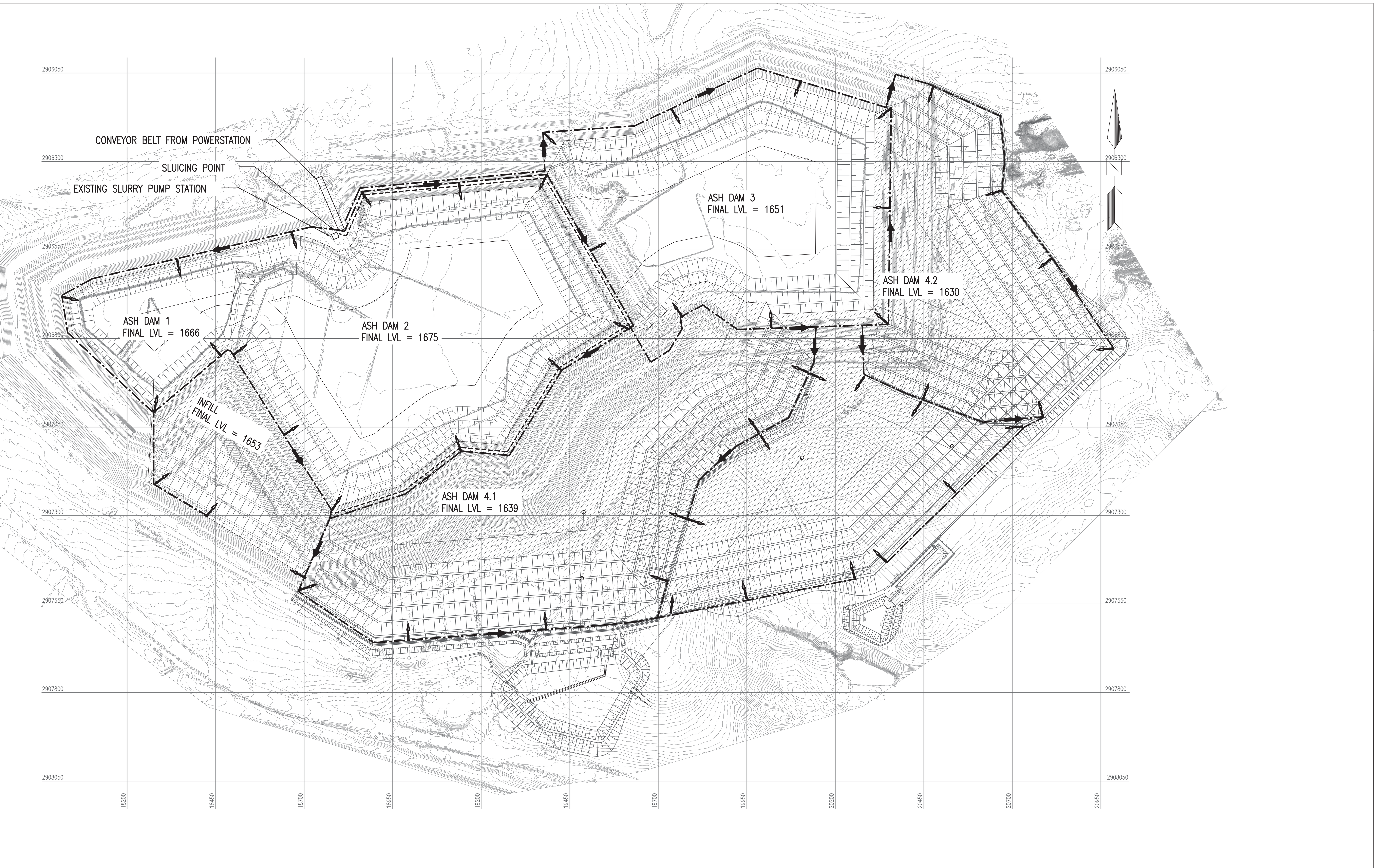
SECTION A–A
1:50



LAYOUT
1:5000

LEGEND :

BOTTOM BOILER ASH
Ø300 STEEL PIPE



LAYOUT
1:5000

LEGEND :

POSSIBLE TRAPEZOIDAL CANAL: PFA
1m BASE x 1,5m DEPTH

PULVERISED FUEL ASH
ø450 HDPE CLASS 10