APPENDIX G: HYDROLOGICAL STUDY

HYDROLOGY ASSESSMENT FOR THE PROPOSED COMMISSIEKRAAL PROJECT

Prepared For: SLR (Africa) (Pty) Ltd

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HYDROLOGY ASSESSMENT FOR THE PROPOSED COMMISSIEKRAAL PROJECT

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HYDROLOGY ASSESSMENT FOR THE PROPOSED COMMISSIEKRAAL PROJECT

1 INTRODUCTION

1.1 BACKGROUND

Highlands Hydrology (Pty) Ltd has been appointed by SLR (Africa) (Pty) Ltd to undertake a hydrological assessment for the proposed Commissiekraal project. Phase 1 of this project included a baseline assessment (completed in June 2013) while phase 2 of the project (completed in June 2014) included a site visit with brief comments on overall hydrology at the site, with major project risks identified. Phase 3 of the project (this report) includes a more detailed hydrology assessment which will is expected to form part of the Environmental Impact Assessment (EIA), associated Management Plan (EMP), as well as Integrated Water Use License Application (IWULA), to be submitted to the Department of Water and Sanitation (DWS). For the purposes of this study, only surface infrastructure associated with the proposed Commissiekraal project has been assessed.

1.2 SCOPE OF WORK

The scope of work for this study included the following:

- Baseline Assessment – baseline climatic data to be used in hydrological calculations. This included the sourcing of appropriate rainfall data, site specific rainfall depth/duration/frequency analysis as well as a regional and local hydrological assessment (Phase 1).
- Site Visit – This provided a better understanding of the dominant hydrological flow regimes at the site as well as help provide input for flood hydrology calculations (Phase 2 and Phase 3).
- Rainfall Runoff Assessment- High level assessment of rainfall-runoff response (Phase 3).
- Flood Assessment- modelling of floodlines for both the 1:50 and 1:100 year return periods for the two adjacent non-perennial streams near the site and the perennial Pandana River which lies to the north of the site. For the remaining non-perennial stream at the site, a 100m buffer was adopted (Phase 3).
- Conceptual Storm Water Management Plan- This was developed based on South African best practice guidance and conceptualized through mapping and indicative design drawings (Phase 3).
- A technical report detailing the achieved scope of work (Phase 3).

1.3 APPLICABLE GUIDANCE

The following guidance documents are applicable to this study when considering the aforementioned scope of work:

- Department of Water Affairs and Forestry, 1998. *National Water Act, Act 36 of 1998;*
- Department of Water Affairs and Forestry, 1999. *Government Notice 704 (Government Gazette 20118 of June 1999); and*
- Department of Water Affairs and Forestry, 2006, "*Best Practice Guideline No. G1: Storm Water Management*", DWAF, Pretoria, August 2006.

1.4 REGIONAL SETTING AND SITE LAYOUT

Figure 1.1 illustrates the regional setting of the proposed Commissiekraal project. The surface infrastructure (excluding haul roads) for the Commissiekraal project is limited to an area of approximately 0.15km^2 (hereafter referred to as 'the site').

Surface features and infrastructure on the site include:

- A box cut (in association with the mine shaft);
- Cable workshop and stone dust store;
- Diesel storage and refuel bay;
- Explosive and detonator stores;
- Offices;
- Oil trap;
- Pollution control dam;
- Roads and parking;
- Sewage treatment plant;
- Silt trap;
- Overburden and soil stockpile
- Product stockpile, crusher and Run of Mine (ROM) stockpile;
- Water Tanks;
- Workshop, wash bay and stores; and
- Additional infrastructure.

Figure 1.2 presents the site layout.

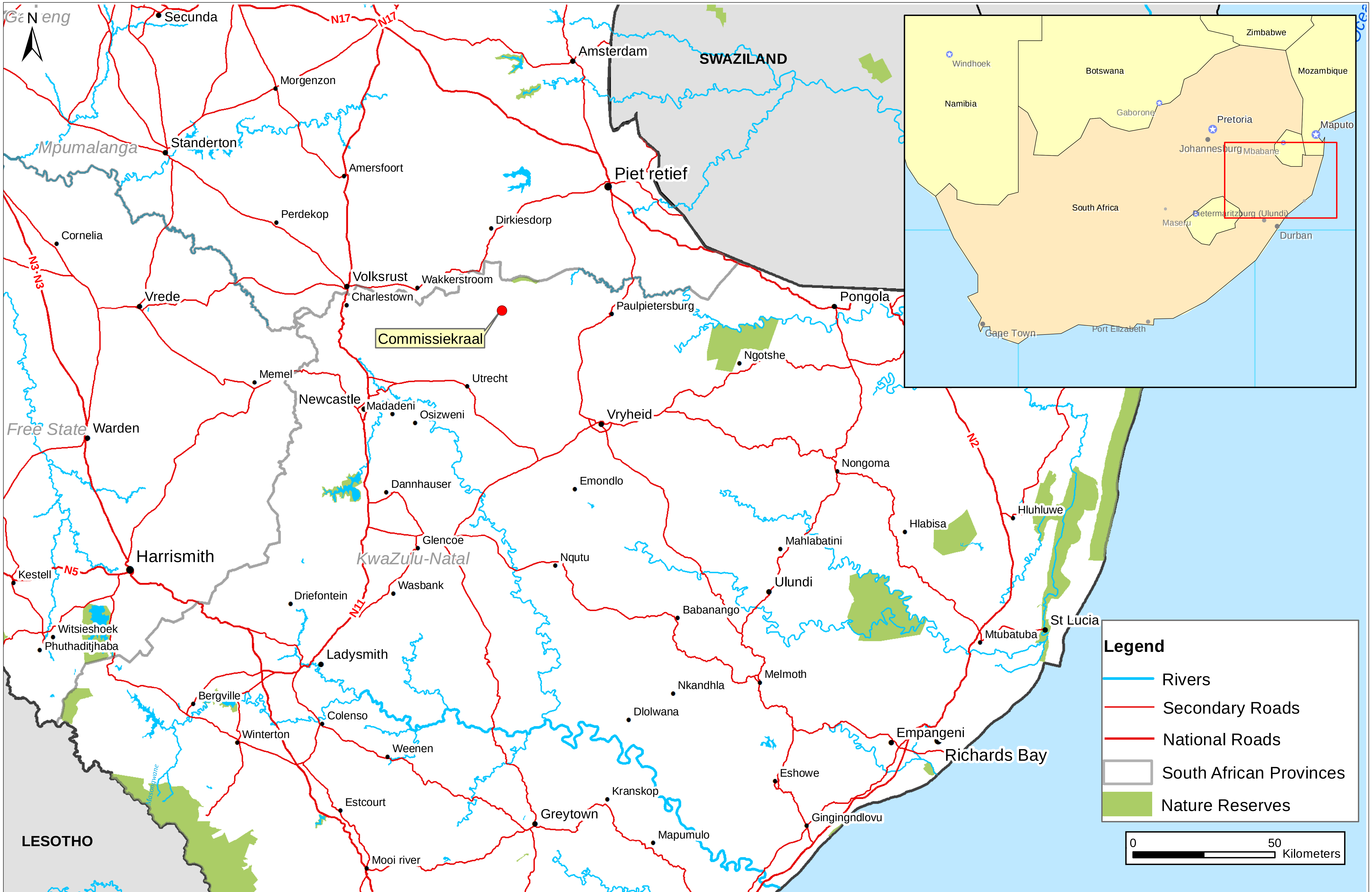
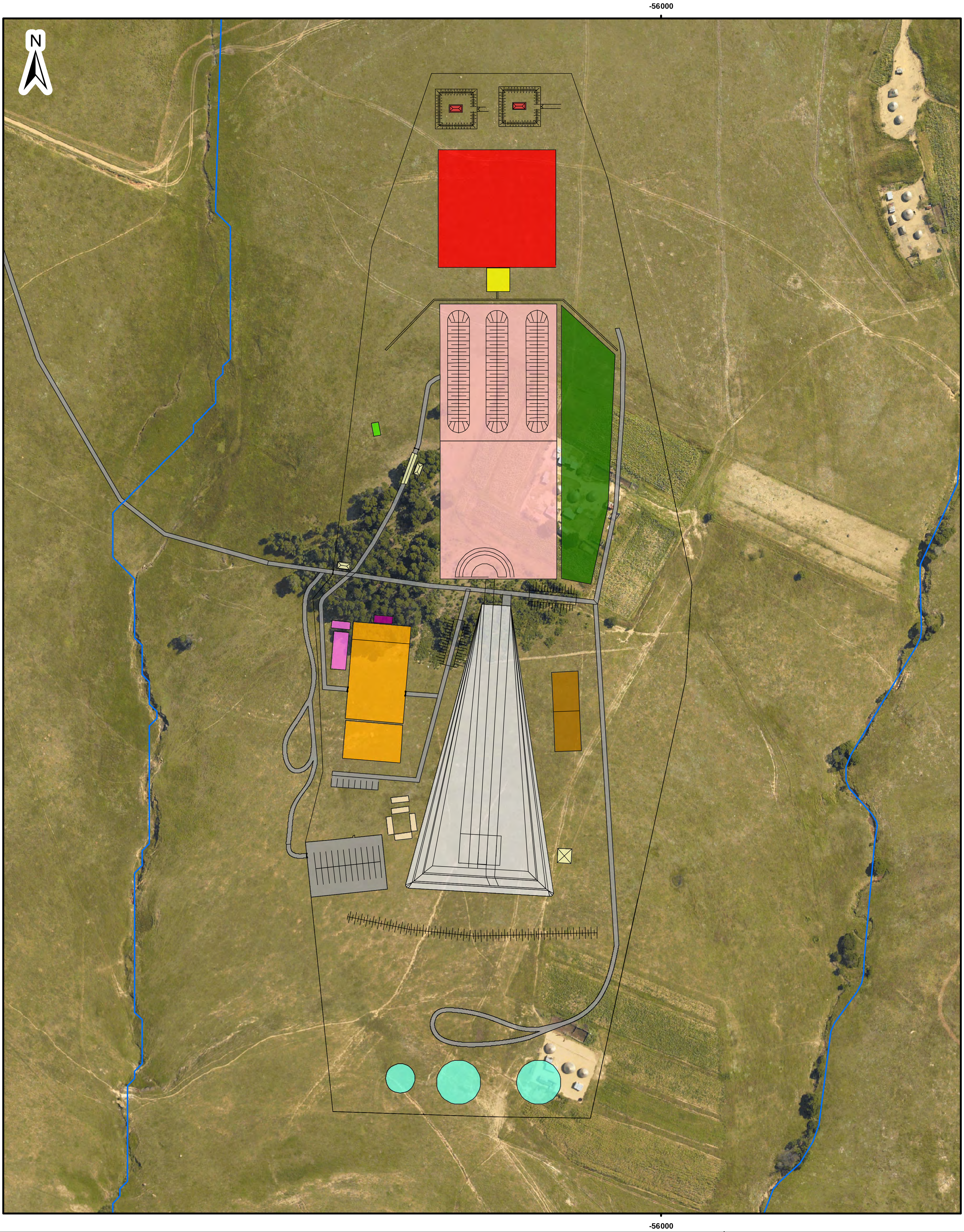


Figure 1-1 Regional Setting of the Commissiekraal Operation



Legend

Additional Infrastructure	Offices	Sewage Treatment Plant
Boxcut (Shaft)	Oil Trap	Silt Trap
Cable Workshop and Stone Dust Store	Overburden and Soil Stockpile	Stockpile, Crusher and R.O.M
Diesel Storage an Refuel Bay	Pollution Control Dam	Water Tanks
Explosives and Detonator Stores	Road and Parking	Workshop, Washbay and Stores

0 100 Meters

Scale: 1:2,500 @ A3

Projection: Transverse Mercator
Datum: WGOLPU94

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Figure 1.2

Site Layout

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2 BASELINE INFORMATION

Baseline information discussed in this section refers to includes information on the climate and the hydrology of the site.

2.1 CLIMATE

Site specific rainfall and evaporation information is available in the following sections, as these are important considerations in this project.

2.1.1 RAINFALL

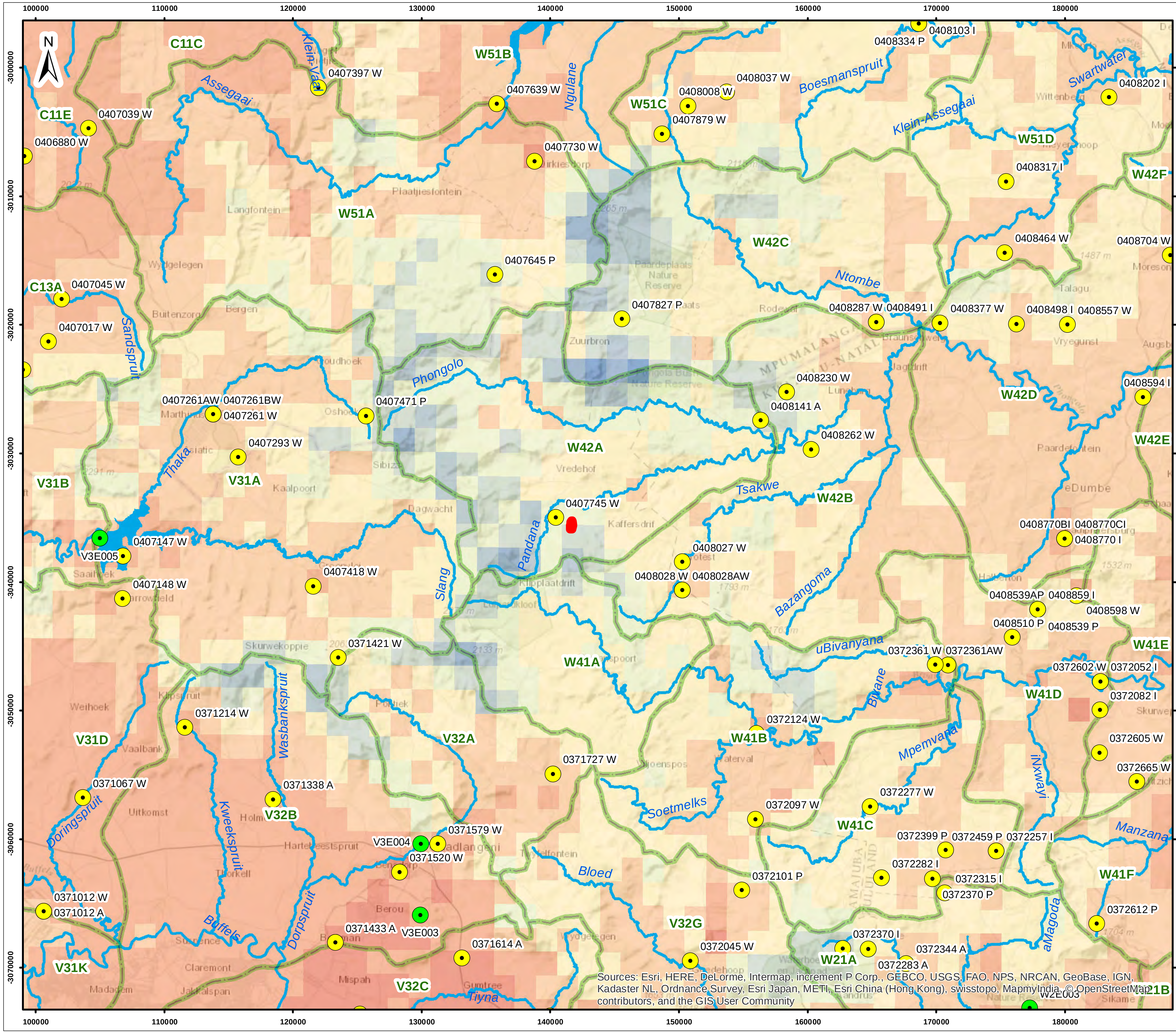
Various weather stations managed by both the South African Weather Services (SAWS) and the Department of Water and Sanitation (DWS) were considered in this project. These, together with their proximity to site are illustrated in Figure 2.1.

The closest rainfall station to the site is SAWS station 0407745 W (Elim) located approximately 1.2km to the west. According to TR102 (Design Rainfall Depths at Selected Stations in South Africa), there is a rainfall record length of 26 years. However, upon confirmation with SAWS, available data appears limited to a 10 year period between 1970 and 1980. Table 2.1 provides a summary of the monthly rainfall distribution at this station as per data received from the SAWS.

Figure 2.1 illustrates the change in Mean Annual Precipitation (MAP) in the greater area about the site as a result of topographic variability. The MAP for the SAWS Elim weather station is 1089mm as presented in Table 2.1. This compares well with the illustration of MAP as presented in
Figure 2.1, with interrogation of the underlying dataset indicating an MAP of 1042mm corresponding with the weather station location and an MAP of 1158mm corresponding with the site.

TABLE 2.1: MONTHLY RAINFALL DISTRIBUTION

Month	Rainfall (mm)
Jan	210
Feb	142
Mar	109
Apr	78
May	28
Jun	5
Jul	6
Aug	23
Sep	54
Oct	103
Nov	141
Dec	190
Total	**1089**



Legend

- Site

Raingauges

- DWAF
- SAWS
- Quaternary Catchments
- Rivers (500K Dataset)
- Dams (500K Dataset)

Mean Annual Precipitation (mm)

- > 1400
- 1301 - 1400
- 1201 - 1300
- 1101 - 1200
- 1001 - 1100
- 901 - 1000
- 801 - 900
- 701 - 800
- 601 - 700
- < 600

5 0 5 Kilometers

Scale: 1:289,000 @ A3

Projection: Transverse Mercator
Datum: Hartebeeshoek, LO29

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Figure 2.2

Weather Stations and MAP

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2.1.2 RETURN PERIOD RAINFALL DEPTHS

For the development of a storm water management plan and flood model, design rainfall was the most important rainfall variable to consider as it is the driver behind peak flows.

Design storm estimates for various return periods and storm durations were sourced from the Design Rainfall Estimation Software for South Africa, developed by the University of Natal in 2002 as part of a WRC project K5/1060 (Smithers & Schulze, 2002). This method uses a Regional L-Moment Algorithm (RLMA) in conjunction with a Scale Invariance approach to provide site specific estimates of design rainfall (depth, duration and frequency), based on surrounding station records. WRC Report No. K5/1060 provides more detail on the verification and validation of the method.

The design rainfall estimates (24-hour storm) using the above technique have been compared to that obtained in TR102 for the SAWS Elim rainfall station, which uses the MAP for the site (1089 mm) and a site location factor in order to determine the design rainfall estimates (Hydrological Research Unit, 1978).

TABLE 2.2: 24-HOUR STORM DEPTHS

Return Period	Rainfall Depth (24 hour)	
	RLMA (Smithers /Schulze)	TR102
2	93	74
5	122	103
10	143	125
20	163	148
50	191	183
100	213	212
200	235	244

In this project, the RLMA technique was selected due to it being based on localised observed data which are specific to the site location and are more conservative for the return periods of interest (50-year and 100-year).

It is important to note, that no allowances for climate change have been made. Climate change should be considered when using the design rainfall depths presented. A risk analysis using the expected life of a structure or process will indicate the relevance of considering climate change (i.e. as the expected life increases the influence of climate change increases).

2.1.3 EVAPORATION

Monthly evaporative estimates to be used in the sizing of containment facilities were sourced from the DWS gauge (V3E005) approximately 36km west of the site as illustrated in Figure 2-2. This station provides a record length of 25 years for the period 1988- 2013. The gauge records S-Pan evaporation, which generally exceeds evaporation from a natural water surface. Table 2.1 presents the monthly S-Pan evaporative values for the gauge V3E005.

TABLE 2.3: MONTHLY EVAPORATION DISTRIBUTION

Month	Mean Monthly Evaporation – S-Pan (mm)
Jan	163
Feb	143
Mar	135
Apr	108
May	96
Jun	82
Jul	89
Aug	118
Sep	158
Oct	160
Nov	165
Dec	174
Total	**1592**

2.1.4 AVERAGE CLIMATIC CONDITIONS

Using the outcome of the investigation into rainfall and evaporation for the site a plot of average monthly climate is presented in Figure 2.2. In addition to the rainfall and evaporation data presented above, this figure makes use of average maximum and minimum temperature sourced from WorldClim datasets (WorldClim, 2015).

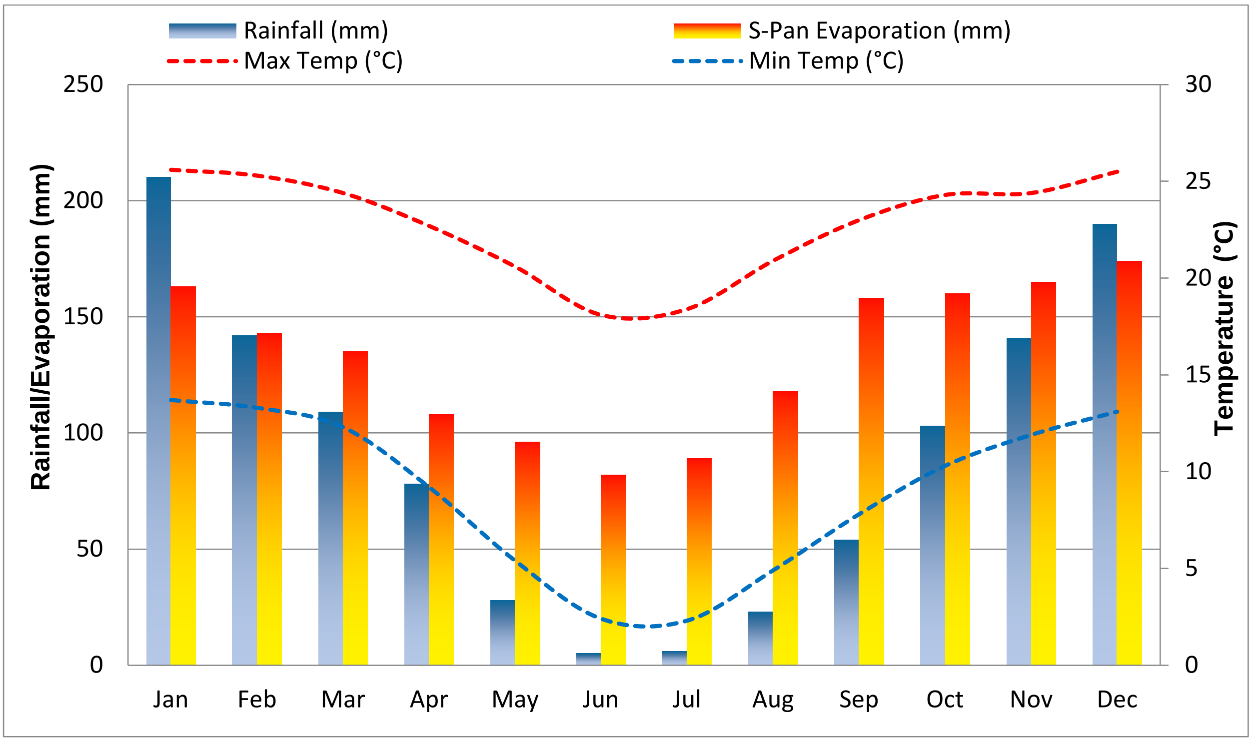


FIGURE 2.2: AVERAGE CLIMATIC CONDITIONS

2.2 TOPOGRAPHY AND LAND COVER

The site is positioned in the Pandana River Valley on the toe slope of a hill rising to small plateau. According to spot heights provided by the client, elevations on site approximate 1490m AMSL. Slopes on site are mild at under 20%, becoming steeper to the south as elevations increase. To the north of the site, slope is reduced in association with the bottom of the valley.

Land cover on the site is largely natural with disturbed areas in the vicinity of the site limited to small areas of subsistence agriculture. According to the SA Vegetation Atlas (SANBI, 2012) natural vegetation of the site is classified as Paulpietersburg Moist Grassland (towards the north) and Wakkerstroom Montate Grassland (towards the south).

Both the topography and land cover of the site are regarded as important considerations in the determination of runoff generated during design rainfall events.

2.3 GEOLOGY AND SOILS

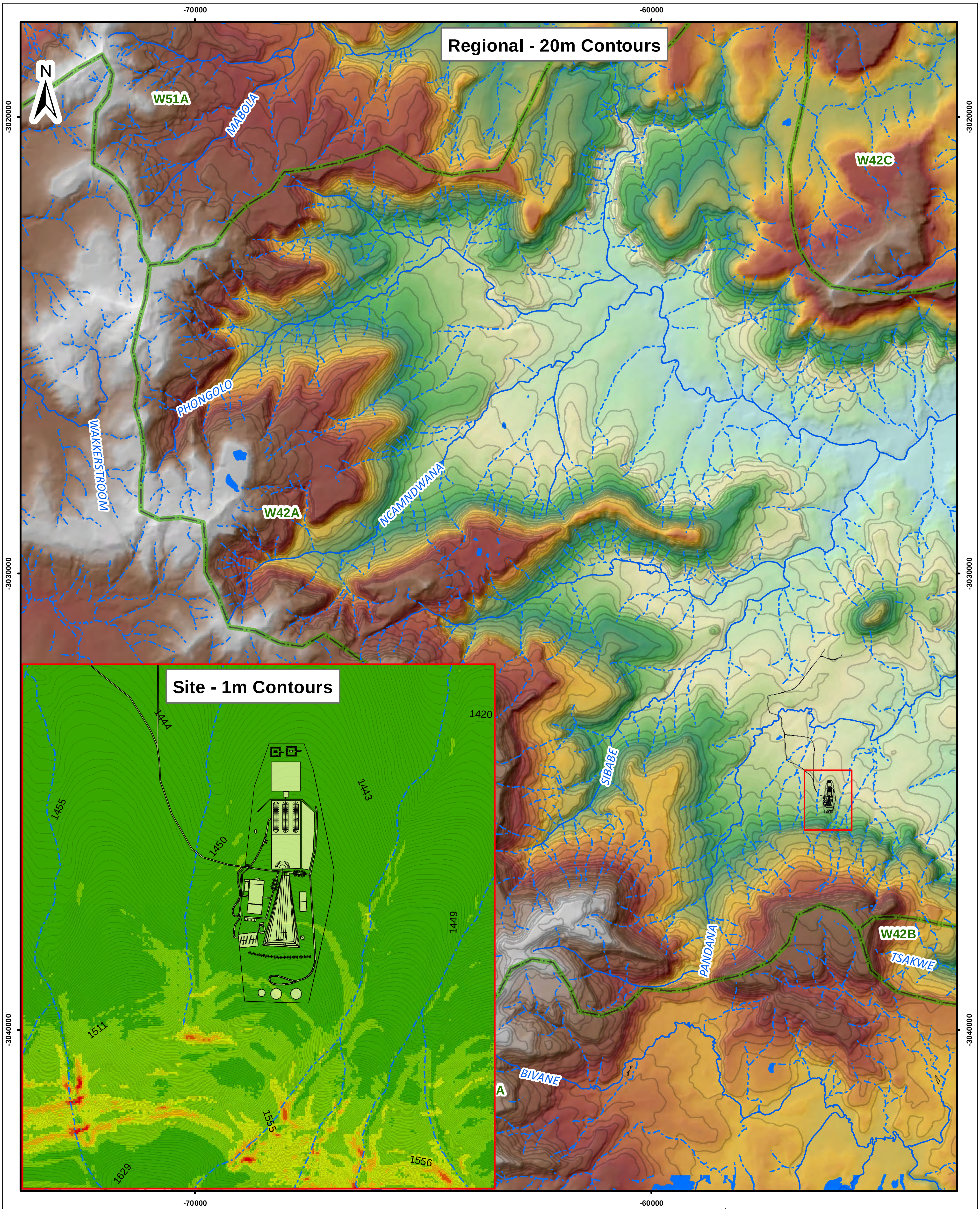
According to the WR2005 (WR2005, 2009) geology dataset, the site is part of the Vryheid formation and is predominantly underlain by a mix of arenite, shale and coal lithologies associated with the valley. Overlying these lithologies are soils defined as Sandy Loams. It is expected that soils in the valley will be deeper, while soils on the plateau will be less deep.

2.4 HYDROLOGY

In terms of surface drainage at (or near to) the site, the non-perennial and perennial stream network as per the 1:50,000 topographical map sheets were extracted and used in the generation of Figure 2.3 to give an indication of the nature of the river systems. According to this stream network, there are two non-perennial streams located close to the eastern and western boundary of the site, with the site infrastructure positioned on the watershed between these two non-perennials.

The site consequently drains in both an easterly and westerly direction, defined according to the position of the non-perennial's watershed. Runoff generated on site (unmanaged) will enter either of the non-perennial steams, flowing towards the north before joining the Pandana River which is a perennial river associated with quaternary catchment W42A. The Pandana River in turn joins the Pongola River which runs to the Indian Ocean.

Wetlands have not been considered in this study and the reader is referred to the relevant wetland specialist report.



Legend

Non-Perennial River (50K)
Perennial River (50K)
Dams (50K)

Slope (%)
- < 20
- 20 - 40
- 40 - 60
- 60 - 80
- 80 - 100
- >100

Elevation (m AMSL)
High : 2200
Low : 1300

0 2 Kilometers
Scale: 1:10,000 @ A3
Projection: Transverse Mercator
Datum: WGOLPU94

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Figure 2.3

Site Topograpy and Hydrology

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3 AVERAGE RUNOFF AND CATCHMENT WATER USE

3.1 MEAN ANNUAL AND MONTHLY RUNOFF

The Mean Annual Runoff (MAR) for the catchment associated with the site was estimated using both the mean annual WR2005 naturalised flow response for 1920 – 2004 and the mean monthly WR2012 naturalised flow response from 1920 to 2009 (WR2005, 2009). Naturalised flow is obtained by removing man-made influences such as dams, irrigation schemes and abstractions. In the case of the site, there is little development in the areas about the site and in upslope area from the site, with only a few plots of subsistence farming noted. Naturalised flow is consequently a suitable predictor of actual flow on site.

WR2005 is the standard dataset that has historically been used for MAR assessments, whereas WR2012 is the current dataset which while providing more detail for MAR, is still undergoing revision. Both WR2005 and W2012 are consequently necessary to considered.

In assessing the mean annual and monthly runoff of the site, the rainfall-runoff response of the site was assumed to be the same as the regional rainfall-runoff response as determined for the quaternary catchment W42A in which the site falls.

3.1.1 WR2005

Using the WR2005 quaternary catchments dataset, and an estimated 0.15km^2 for the site (which accounts for the full site boundary), it estimated that runoff generated from the site accounts for approximately 0.044 million m^3 of the quaternary catchments 116.3 million m^3 (equivalent to 0.038% of total quaternary runoff).

3.1.2 WR2012

The WR2012 mean monthly estimate of runoff for the site (using 0.15km^2 contributing area) for the period from 1980 to 2009 (30 years), is illustrated in Figure 3.1. 30 years is considered a period over which a climate 'normal' can be derived as described by the World Meteorological Agency (WMO, 2015). The mean annual runoff over this period is 0.036 million m^3 which is less than the WR2005 estimate (the WR2005 has a higher mean annual runoff estimate for quaternary W42A than the WR2012 estimate).

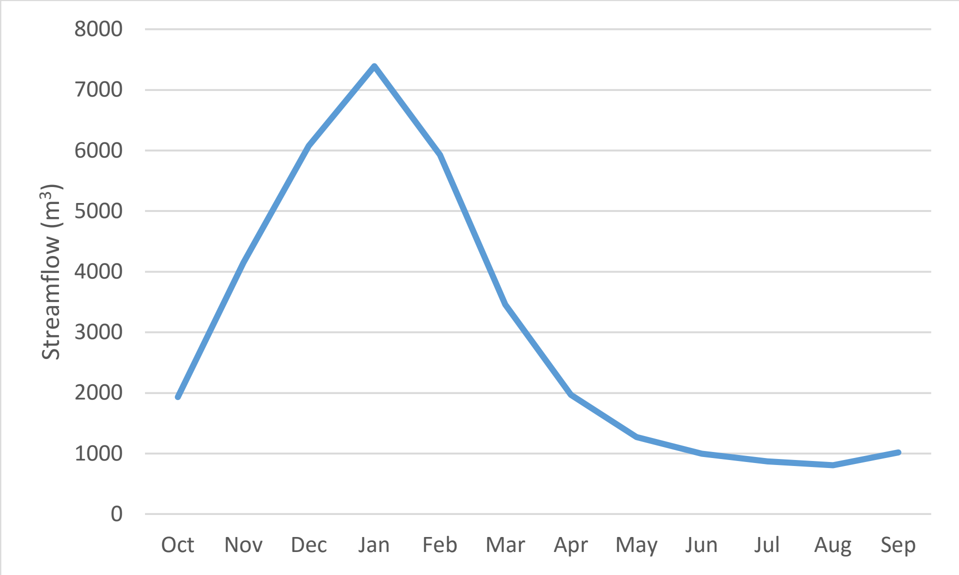


FIGURE 3.1: MEAN MONTHLY RUNOFF FOR THE SITE USING WR2012 (AVERAGE 1980 TO 2009)

3.2 COMPARISON OF WR2012 WITH MEASURED FLOW

A single measurement of stream flow was taken on the Pandana River at coordinates 27°24'44.0" S and 30°25'54.2" E as illustrated in Figure 3.2. The measurement was performed by Stephen van Staden of Scientific Aquatic Services (SAS) in September 2015. The details relating to the site visit and streamflow estimate are described in the SAS specialist report.

The stream flow was estimated at 29.5 l/s and was scaled to a monthly runoff volume of 79,012m^3 for September 2015. In comparing this value to the average WR2012 estimate (1980 – 2009), a contributing area of 24km^2 was used to scale the runoff generating area upstream of the measurement point with the total area of the quaternary catchment, resulting in a WR2012 (1980 – 2009) runoff volume estimate of 163,184m^3.

A plot of previous WR2012 estimates for September is provided in Figure 3.3. This indicates the variability of WR2012 monthly runoff (1990 – 2009) for September compared to the September 2015 site estimate. This plot shows that in some years, the September 2015 flow estimate better approximates the WR2012 estimates. On average, the WR2012 data over estimates the site estimate by 100%.

A longer period of recorded data for the Pandana River will be required for conclusions to be drawn regarding the rainfall/runoff response at the site. This will also assist in better understanding groundwater/surface water interactions and the effect this has on low-flows and associated link to modelled WR2012 flow estimates. It is therefore recommended that a weir with associated streamflow gauge and rain gauge be installed on the Pandana River. For the interim period until a weir is installed it is recommended that at a minimum, monthly estimates of streamflow are taken using the methodology described in the SAS specialist report.

FIGURE 3.2: :LOCATION OF THE SEPTEMBER 2015 STREAM FLOW ESTIMATE

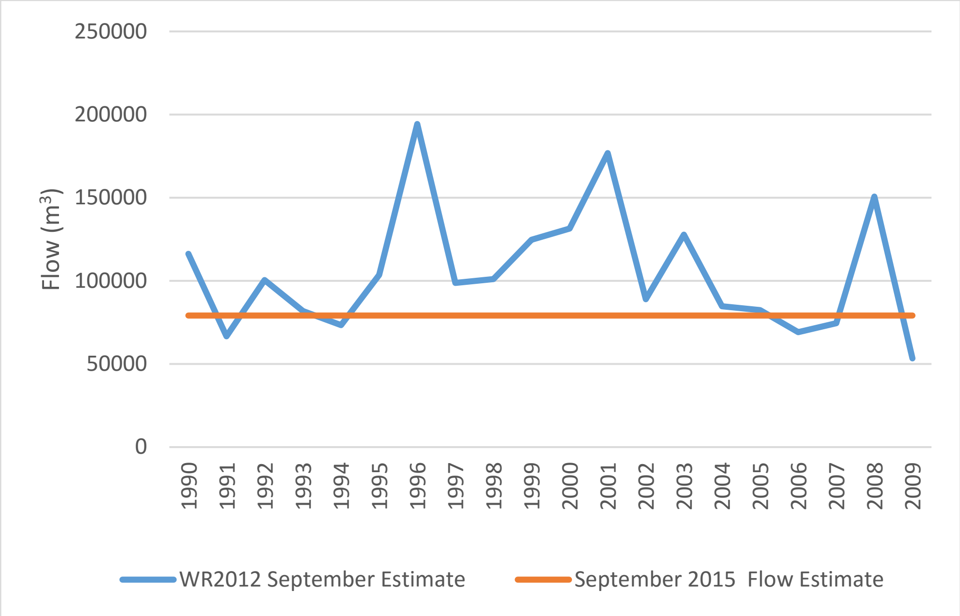


FIGURE 3.3: WR2012 ESTIMATE (1990 TO 2009) VS SEPTEMBER 2015 ESTIMATE

3.3 CATCHMENT WATER USE

In terms of water resource utilisation in the greater water management area, Section 3 (Water Resource Profile) of the document titled Zululand District Municipality Water Services Development Plan (DC26) (2013) has been reviewed (Zululand District Municipality, 2013). This document gives an indication of water resources use and availability for the main rivers namely the Mfolozi, Mkuze and Pongola. The proposed project located in quaternary catchment W42A flows into the Pongola river system. According to the document "*It is evident that apart from the Pongola catchments, water from these sub-areas is currently over-utilised and a deficit is created. However, according to Basson and Rossouw*[1]*, this deficit is a result of the provision made for future implementation of the Reserve". The document further refers to a "surplus "of water in the Pongola River with the Pongola catchment currently "under-utilised*". More detail on water resources and associated utilisation can be found in Table 3.1 or within the detailed document (Zululand District Municipality, 2013).

It is recommended that the DWS, as part of their custodianship of water resources in South Africa, further investigate the deficit/surplus of water resources within the affected catchments. This investigation should consider current legal/illegal abstractions as well as the reserve (basic human needs as well as ecological considerations) so that a more informed decision can be made.

[1] (Basson & Rossouw, 2003)

TABLE 3.1: WATER BALANCE - SUMMARY OF THE WATER AVAILABLE AND REQUIRED WITHIN ZULULAND DISTRICT MUNICIPALITY FOR THE YEAR 2000 (MILLION M3 (Kℓ) PER ANNUM).

			Mfolozi	Mkuze	Pongola	Total
Available water	Natural resource	surface water	36	15	616	667
		groundwater	5	12	8	25
	Usable return flow	Irrigation	5	6	21	32
		Urban	4	0	0	4
		Mining & bulk	1	0	0	1
	Total local yield*		**51**	**33**	**645**	**729**
	Transfers in		0	30	0	30
		Total available	**51**	**63**	**645**	**759**
Water requirements	Consumer groups	Irrigation	51	61	213	325
		Urban**	12	1	1	14
		Rural**	11	10	6	27
		Mining & bulk industrial***	4	0	1	5
		Afforestation****	2	6	34	42
	Total local requirements		**80**	**78**	**255**	**413**
	Transfers out		18	0	30	**48**
		Total used	**98**	**78**	**285**	**461**
Balance			**-47**	**-15**	**360**	**298**

Source: (Basson & Rossouw, 2003)

4 FLOOD MODELLING

4.1 REQUIREMENT FOR FLOOD MODELLING

The aim of the flood modelling undertaken as part of this study is to fulfil the requirements of the National Water Act (Act 36 of 1998) and more particularly, Government Notice 704 (Government Gazette 20118 of June 1999) (hereafter referred to as GN 704). The final mine plan will need to consider the specific provisions of GN704 for both the surface water infrastructure (considered in this study) and the underground mine workings (not considered in this study). The principle condition of GN 704 applicable to this project with regards to flooding is summarised as follows:

- *Condition 4* which defines the area in which mine workings or associated structures may be located with reference to a watercourse and associated flooding. The 50 year flood-line and 100 year flood line are used for defining suitable locations for mine workings (prospecting, underground mining or excavations) and associated structures respectively. Where the flood-line is less than 100 metres away from the watercourse, then a minimum watercourse buffer distance of 100 metres is required for both mine workings and associated structures.

The site is located on the watershed between two non-perennial streams. The proximity of the site to these two streams necessitated flood modelling in order to define the 50-year and 100-year flood-lines. The two non-perennial streams and a portion of the Pandana River accounted for the streams for which flood modelling was undertaken.

4.2 MODEL APPROACH

The primary topographic dataset available for flood modelling was an XYZ point file (5m_DEM_LO31.xyz) with a continuous and uniform 5m grid spacing. This dataset was subsequently interpolated into a Digital Elevation Model (DEM) with a 5m cell size. The availability of a continuous DEM for the site allowed for the adoption of a 2-D flood modelling approach. Unlike a 1-D approach (using cross-sections) which samples the DEM at set cross-section locations, a 2-D model approach e uses a continuous model 'mesh'. The advantage of a 2-D model is consequently its ability to account for more variation in the topographic data through the improved use of all available topographic data.

The outcome of the hydrologic and hydraulic model as it pertains to the flood modelling for the site is presented in APPENDIX A: Flood Model Setup.

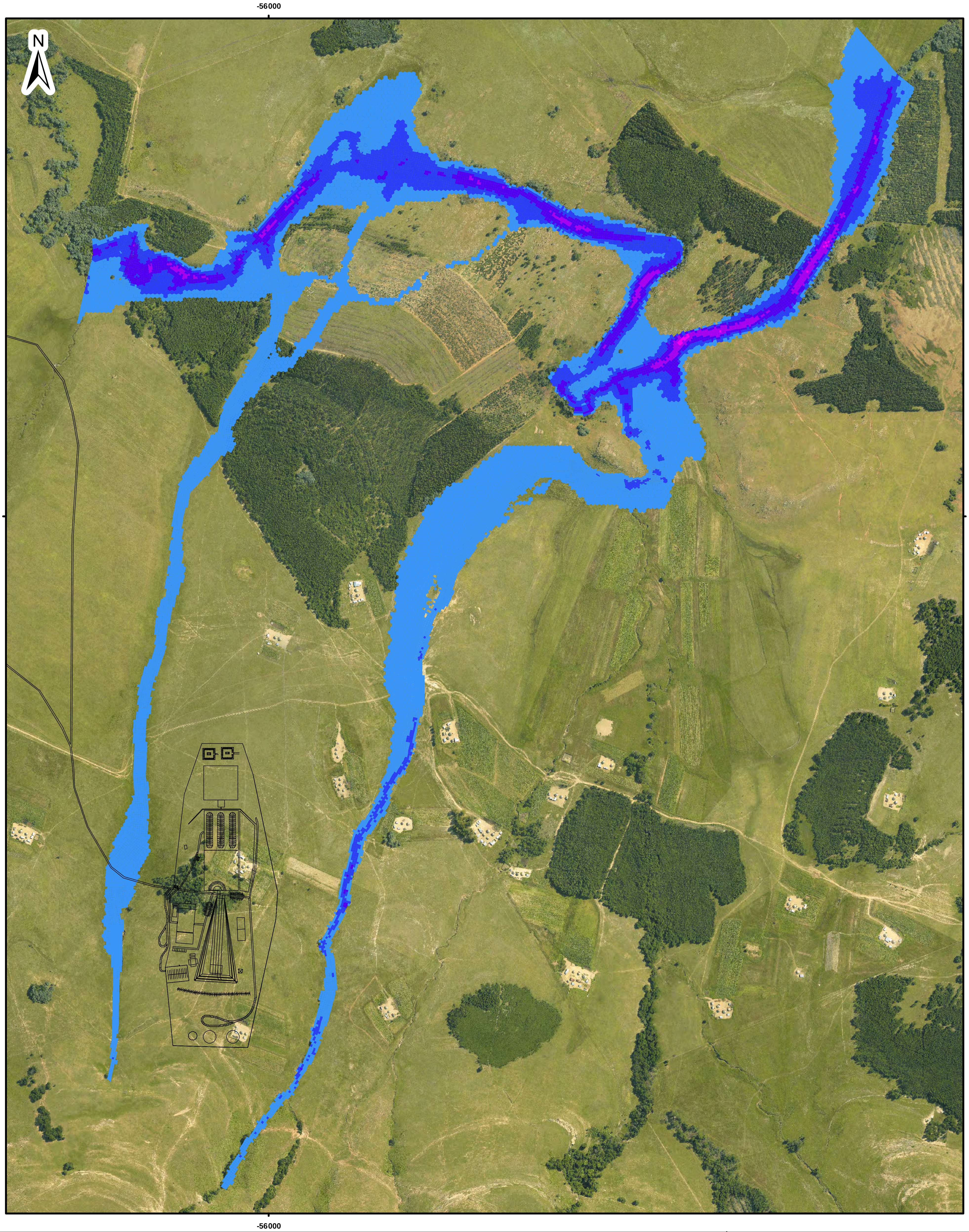
4.3 FLOOD MODELLING RESULTS

The overall results of the flood modelling results are presented in Figure 4.1 while illustrate the maximum depth anticipated for the 1:100 year event. As expected, the depth of flooding is greater in the Pandana River reaching a maximum of 5m. The two tributaries adjacent the site exhibit less flooding with depths under 1m associated with the west tributary where flood waters are able to spread out more over the floodplain. The east tributary exhibits deeper flooding associated with a more constrained floodplain.

Flooding near the site is illustrated in Figure 4.2 which presents both the 1:50 and 1:100 year flood extents. The difference between the 1:50 and 1:100 year events is limited by the topography of the floodplains which rise quickly enough to constrain the spread of flood waters. Flooding buffers pertaining to the river centreline are also included since GN704 refers to a minimum watercourse buffer of 100m. Two flooding buffers have been used which were derived from two approaches; one a product of the 5m DEM analysis and the other a product of the digitizing according to the aerial photo of the site. Some differences are noted between the two buffers (DEM vs. aerial photo). To be fully compliant with GN704 will require a measurement of the horizontal distance from the river bank of 100m and the exclusion of any mine workings or associated structures within this zone.

In assessing the layout of infrastructure with regards to the illustrated 1:50 and 1:100 year flood extents, the only sensitive infrastructure is the access road which is positioned over the floodplain to the west of the site (Figure 4.1 and Figure 4.2). In addition to this crossing, other stream crossings are noted in association with the access road to site. Flood extents for these additional crossings have not been calculated but in principal, all linear infrastructure (such as pipelines or roads) associated with the site and which cross over streams should not interfere with the existing conveyance of river water (either during normal flow periods or during a flood). Culverts and bridges should consequently be sufficiently sized to provide capacity to convey the 1:100 year flood event over the expected life of the structure to minimise impacts and ensure that the natural flow regime can be maintained as far as possible.

It is important to note, that no allowances for climate change have not been made. A risk analysis using the expected life of a structure will indicate the relevance of considering climate change (i.e. as the expected life increases the influence of climate change increases).



Legend

100-Year Flooding

Flood Depth (m)

- < 1
- 1 - 2
- 2 - 3
- 3 - 4
- 4 - 5

0 250 Meters

Scale: 1:8,700 @ A3

Projection: Transverse Mercator
Datum: WGOLPU94

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Figure 4.1

Overall Flooding

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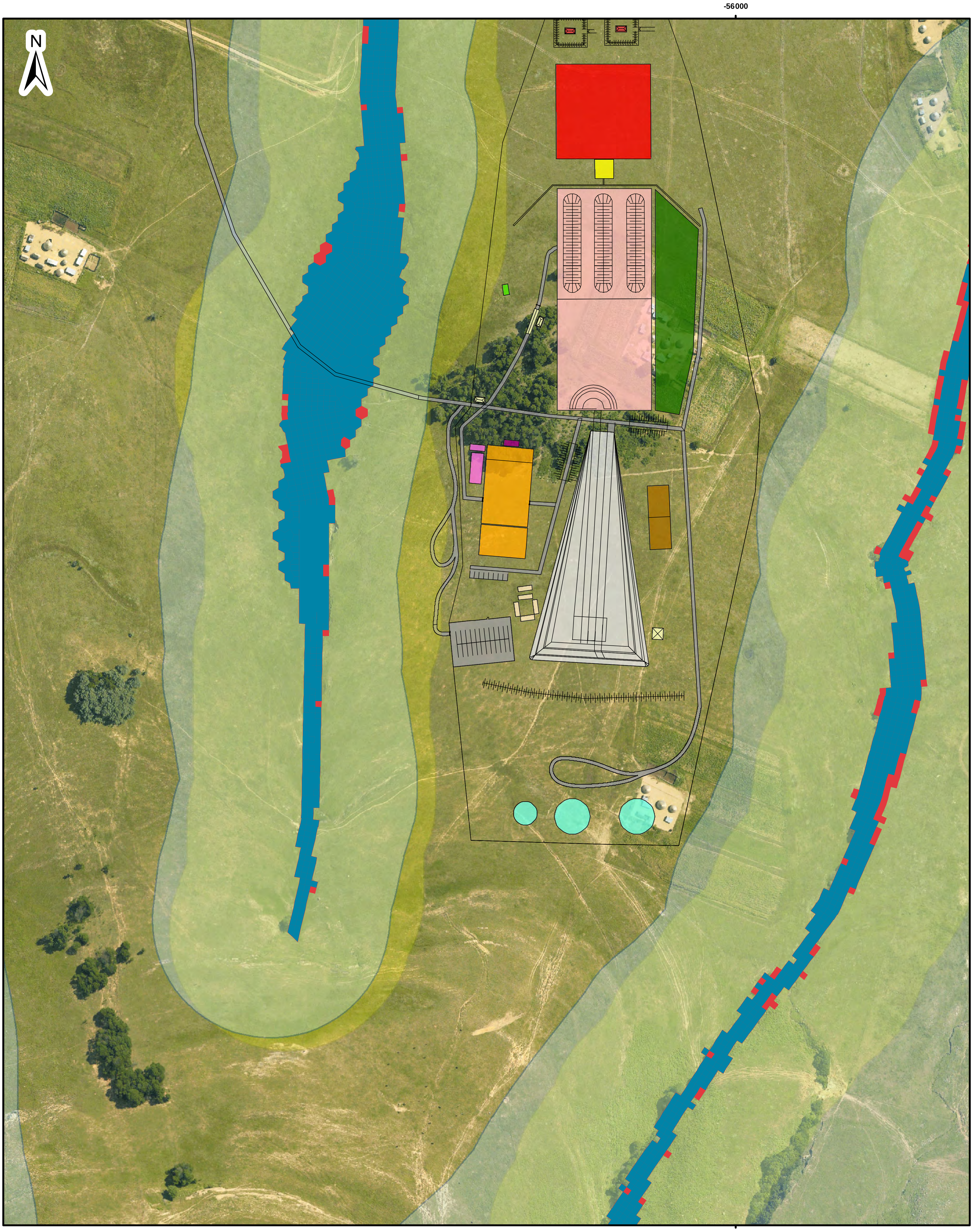

Legend

DEM Centreline (100m Buffer)	
100m River Buffer (50K Mapping)	
50-Year Flood Extent	
100-Year Flood Extent	
Additional Infrastructure	
Boxcut (Shaft)	Road and Parking
Cable Workshop and Stone Dust Store	Sewage Treatment Plant
Diesel Storage and Refuel Bay	Silt Trap
Diesel and Refuel Bay	Stockpile, Crusher and R.O.M
Explosives and Detonator Stores	Water Tanks
Offices	Workshop, Washbay and Stores
Oil Trap	
Pollution Control Dam	

0 100 Meters

Scale: 1:3,200 @ A3

Projection: Transverse Mercator
Datum: WGOLPU94

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Figure 4.2

Site Flooding and River Buffers

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5 CONCEPTUAL STORM WATER MANAGEMENT PLAN

The aim of this storm water management plan (SWMP) is to fulfil the requirements presented in Government Notice 704 (Government Gazette 20118 of June 1999) which deals with the separation of clean and dirty water. The conceptual storm water management plan will form a necessary part of the Integrated Water Use License Application (IWULA), to be submitted to the Department of Water and Sanitation (DWS). This storm water management plan also complies with the principles presented in the DWS Best Practice Guideline G1 for Storm water Management.

5.1 DWAF GOVERNMENT NOTICE 704

The Department of Water Affairs and Forestry (now the Department of Water and Sanitation), established GN 704 to provide regulations on the use of water for mining and related activities aimed at the protection of water resources. There are important definitions in the regulation which require understanding.

5.1.1 IMPORTANT DEFINITIONS IN GN 704

- **Clean water system:** This includes any dam, other form of impoundment, canal, works, pipeline and any other structure or facility constructed for the retention or conveyance of unpolluted water.

- **Dirty water system:** This includes any dam, other form of impoundment, canal, works, pipeline, residue deposit and any other structure or facility constructed for the retention or conveyance of water containing waste.

- **Dirty area:** This refers to any area at a mine or activity which causes, has caused or is likely to cause pollution of a water resource (i.e. polluted water)

5.1.2 APPLICABLE CONDITIONS IN GN 704

The principle conditions of GN 704 applicable to the development of a SWMP for the site are summarised as follows:

- *Condition 5* which indicates that no residue or substance that causes or is likely to cause pollution of a water resource may be used in the construction of any dams, impoundments or embankments or any other infrastructure.
- *Condition 6* which describes the capacity requirements of clean and dirty water systems. Clean and dirty water systems must be kept separate and must be designed, constructed, maintained and operated such that these systems do not spill into each other more than once in 50 years.

- *Condition 7* which describes the measures that must be taken to protect water resources. All dirty water or substances which cause or are likely to cause pollution of a water resource either through natural flow or by seepage are to be mitigated.

5.2 CLEAN AND DIRTY WATER CATCHMENTS

In Figure 5.1, clean and dirty areas have been delineated for the surface works. These areas were delineated using the 5m DEM of the site. In addition to the dirty and clean areas Figure 5.1 also indicates the position of the self-contained (dirty) box-cut as well as site roads which require appropriate road side management to contain coal spillages as a result of site operation and haulage.

The clean area south of the site (Clean - A) is comprised of:

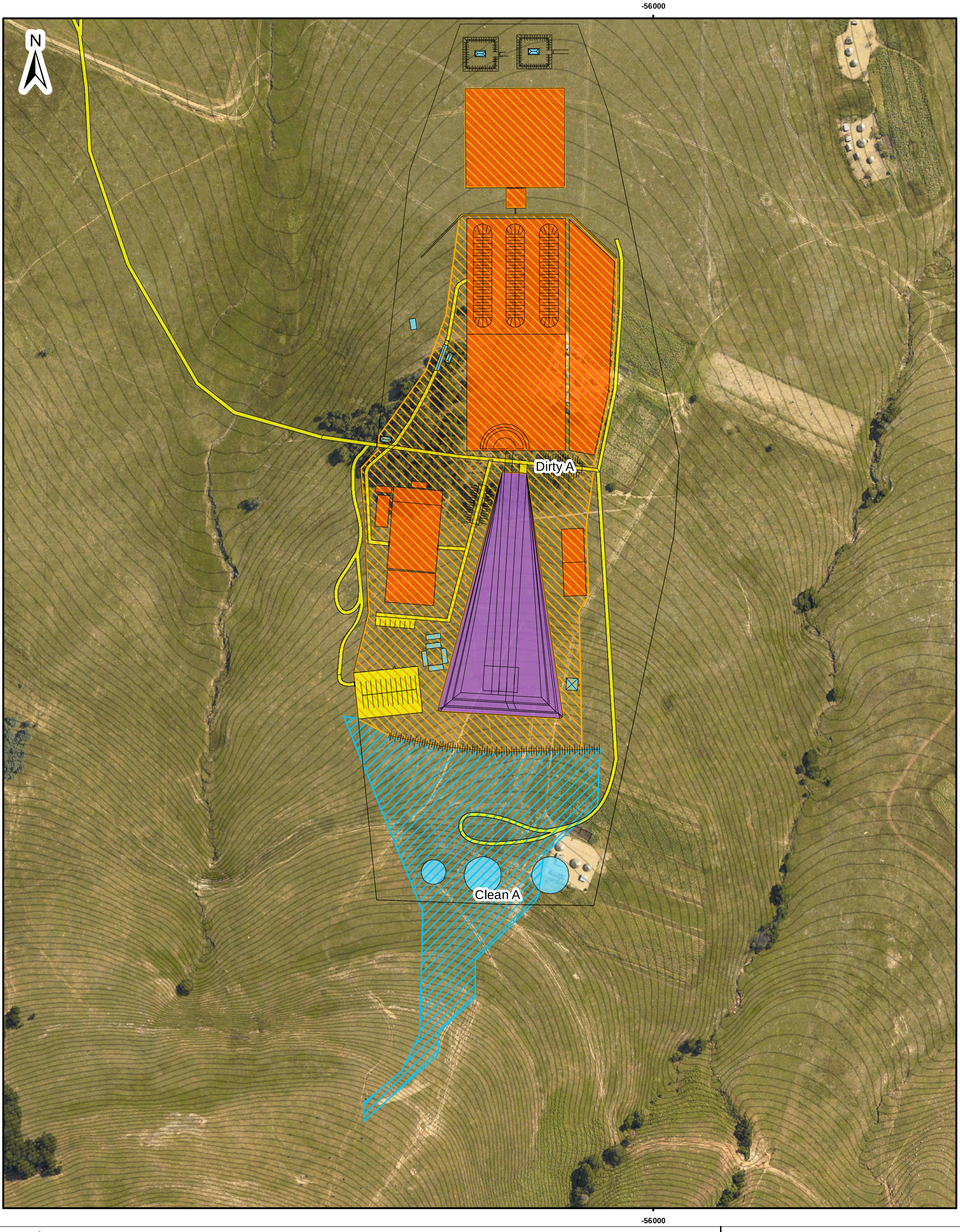
- Water tanks;
- Grassland; and
- Turning area.

The dirty in the north of the site (Dirty - A) has been considered as a continuous dirty water generating area comprised of:

- Cable workshop and stone dust store;
- Diesel storage ;
- Refuel bay;
- Oil trap;
- Pollution control dam;
- Roads and parking;
- Silt trap;
- Overburden and soil stockpile
- Product stockpile, crusher and Run of Mine (ROM) stockpile; and
- Workshop, wash bay and stores.

As per the guidance of GN704, these dirty water generating areas need to be managed appropriately. Furthermore, the storage/handling of fuel, lubricants and chemicals will require special attention due to their hazardous nature. These areas are required to be managed on impermeable floors with appropriate bunding and sumps.

Details on the methodology used to derive storm flows and volumes for the clean and dirty areas is provided in Appendix B – Storm Water Management Calculations. The resulting design hydrographs used in the development of the SWMP are presented in Figure 5.2.



Legend

- Contours (1m)
- Clean Area
- Dirty Area
- Clean
- Dirty (Contained)
- Dirty (Management Required)
- Dirty (Road Side Managment Required)
- Dirty (Self-Contained)

0 100 Meters

Scale: 1:3,000 @ A3

Projection: Transverse Mercator
Datum: WGOLPU94

Tholie Logistics

Figure 5.1

Dirty and Clean Areas

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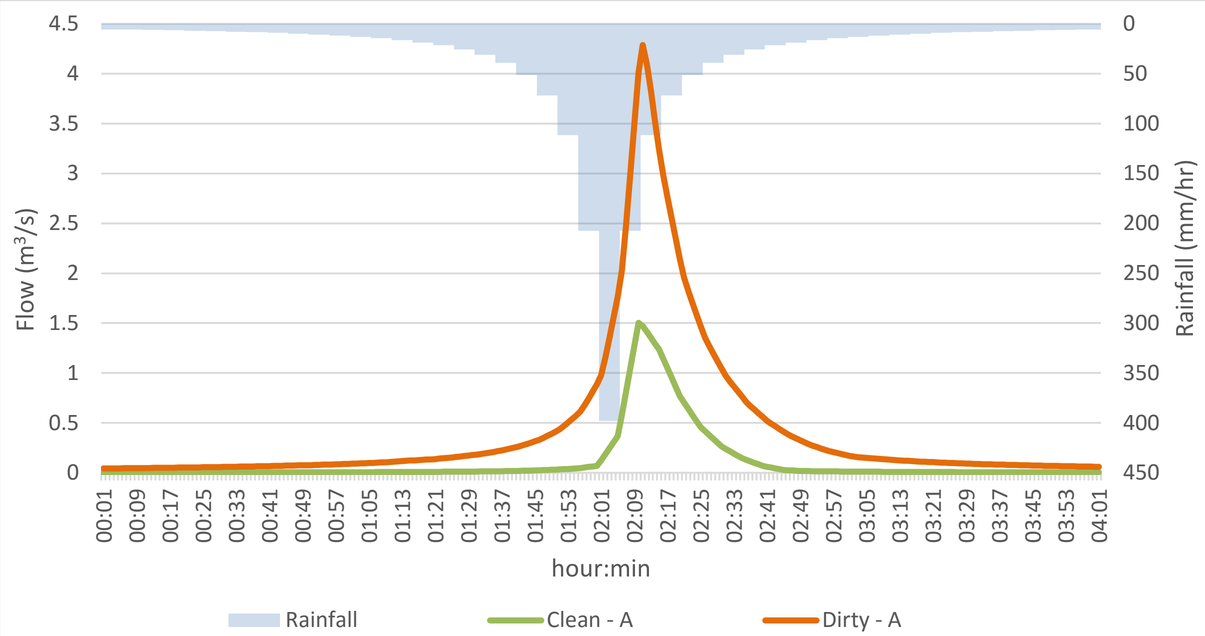


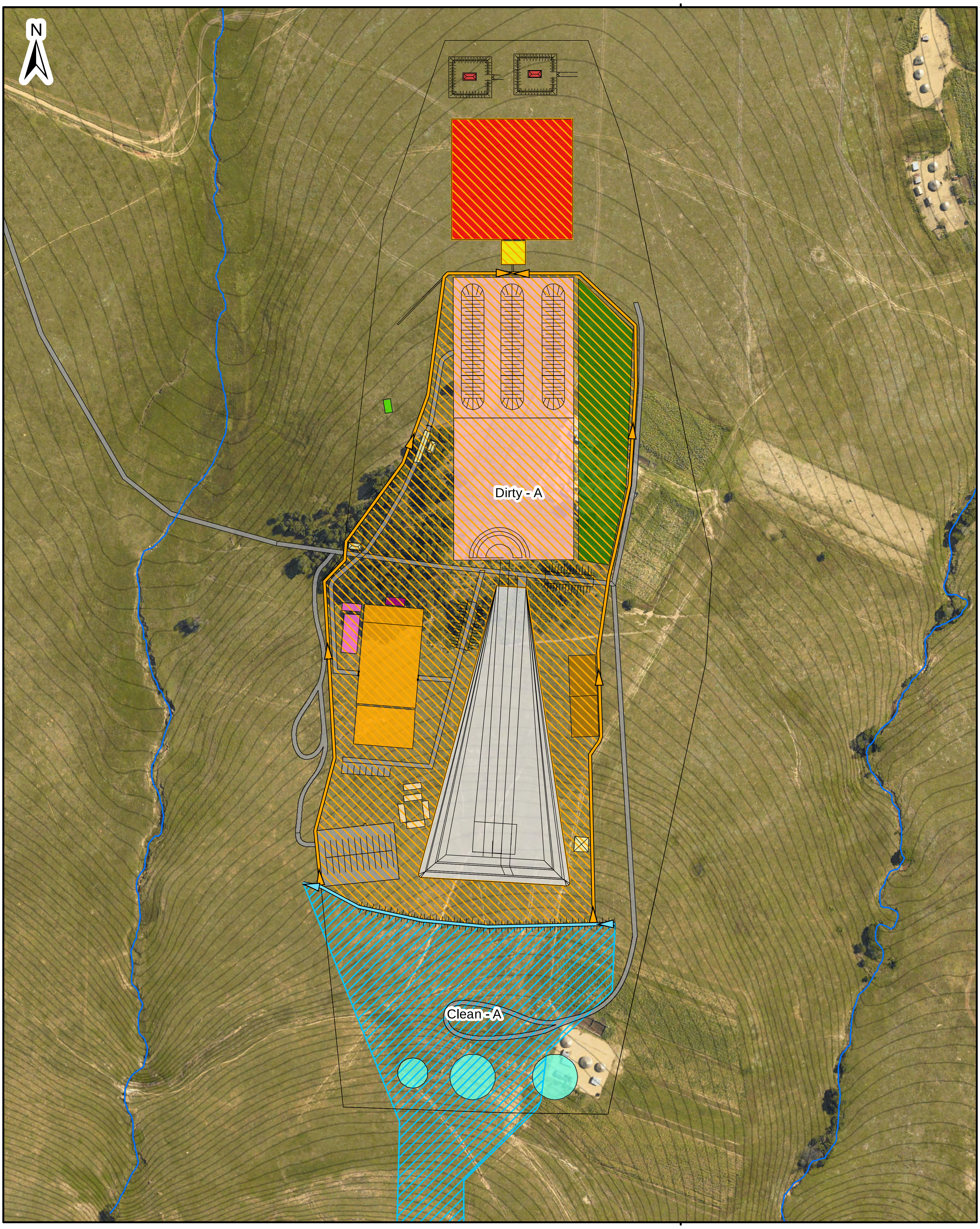
FIGURE 5.2: 1:50 YEAR DESIGN HYDROGRAPHS

5.1 Storm Water Management Infrastructure

Storm water management infrastructure has been conceptually designed in this report as per the requirements of GN 704 and presented in Figure 5.3. The dirty water containment facility has been sized according to the layout of the site provided by the client.

5.1.1 Clean Water Berm

According to the site layout provided by the client, the clean water area (Clean – A) to the south is diverted through the use of a diversion berm, and excludes a channel component. Since the peak runoff is estimated to be 1.5m^3/s, a diversion berm (if constructed appropriately), will likely be sufficient. In order to assess this assumption, a trapezoidal channel was modelled with a side slopes of 1 vertical: 5 horizontal and a base 5m wide, in order to mimic the shallow flow anticipated alongside the berm. A Manning's 'n' roughness coefficient of 0.03 (maintained grass) was used. The channel was sized to enable conveyance of the 1:50 year flood. Maximum flow depth was noted to be approximately 0.14m while velocities approximated a maximum of 2m/s. A maximum velocity of 2m/s is high enough to potentially cause soil erosion. The South African National Roads Agency Limited (SANRAL) drainage manual (SANRAL, 2006) provides guidance on maximum permissible velocities for grass covers and should be consulted during detailed design.



Legend

Contours (1m)	Boxcut (Shaft)	Pollution Control Dam
River	Cable Workshop and Stone Dust Store	Road and Parking
Clean Diversion	Diesel Storage an Refuel Bay	Sewage Treatment Plant
Dirty Diversion	Explosives and Detonator Stores	Silt Trap
Clean Area	Offices	Stockpile, Crusher and R.O.M
Dirty Area	Oil Trap	Water Tanks
Additional Infrastructure	Overburden and Soil Stockpile	Workshop, Washbay and Stores

0 100 Meters

Scale: 1:2,500 @ A3

Projection: Transverse Mercator
Datum: WGOLPU94

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Figure 5.3

Conceptual Stormwater Management Plan

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5.1.2 DIRTY WATER BERMS/CHANNELS

Dirty water containment systems have been designed to ensure dirty water generated on the site is contained. These systems will also consist of a berm and channel component routing to a containment facility. Leach tests are required to be undertaken to determine the potential for pollutants to enter the environment through seepage, and thereby the requirement for lining of the dirty water channels. A conservative approach has nevertheless been assumed whereby all dirty channels will be concrete lined.

The berm and channel component have been designed to accommodate the 1:50 year flood and serve two main purposes:

- Diverting upstream clean water which would otherwise flow into the identified dirty areas.
- Contain dirty water in the identified dirty areas and direct towards the appropriate dirty water containment facility.

Figure 3.5 represents a typical dirty water containment berm and channel. The berm component will be constructed from the material excavated from the channel and supplemented by topsoil stockpiling if required. The side slopes for all berms and channels will be kept constant at 1 vertical: 2 horizontal. The channel component has been sized using PCSWMM to meet the requirement of accommodating the 1:50 year flood. A Manning's 'n of 0.015 was used in the calculations, associated with a concrete lined channel.

In Figure 5.4:

- a = Channel Depth
- b = Channel base breadth

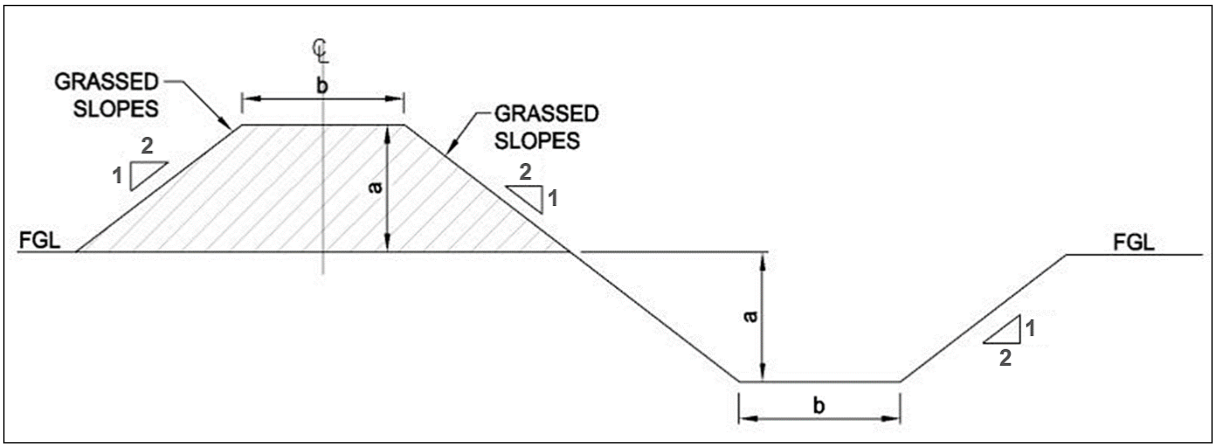

FIGURE 5.4: TYPICAL BERM AND CHANNEL FOR DIRTY STORMWATER DIVERSION SYSTEM

Table 5.1 presents the dimensions of the dirty diversions associated with the site. In the case of Dirty – A, there are two primary diversions. Since details on site levelling and subsequent drainage setup is not yet available, dirty water has been routed to a single diversion in order to retain a conservative approach. The final site drainage may result in a reduction in the volume/rate of dirty water requiring diversions, and consequently the dimensions outlined in Table 5.1 may be less.

TABLE 5.1: BERM AND CHANNEL DIMENSIONS FOR DIRTY STORMWATER

Catchment	a (m)	b (m)	Average longitudinal Slope (%)
Dirty - A	0.5	1.0	5.8

The average longitudinal slope used in the calculation of the channel dimensions is likely to differ once the site has been levelled. The channel dimensions should consequently be reviewed during detailed design.

5.1.3 DIRTY WATER CONTAINMENT – CAPACITY REQUIREMENTS

Condition 6 of GN 704 states that clean and dirty water systems must be kept separate and must be designed, constructed, maintained and operated such that these clean and dirty water systems do not spill into each other as a result of storm events below and including the 1 in 50 year event. A minimum freeboard of 0.8 m above full supply level must also be maintained as per the requirements of GN 704.

In this project, the capacity of the dirty water containment facility was calculated based on the summation of the 1:50 year design rainfall (24 hour) event for the catchment area **and** the highest monthly rainfall (January) falling over the catchment, **less** the corresponding monthly evaporation (January) taking place over the surface area of the proposed containment facility. PCSWMM was used to model the containment of water, with the volume of runoff associated with monthly rainfall calculated using the Rational Method and set as an initial depth in PCSWMM.

It should be noted that it is assumed that the containment facility will always contain sufficient storage to accommodate the 1 in 50 year rainfall event. It is therefore anticipated that a management plan will accompany the construction of the containment facilities, such that water which accumulates over the year will either be reused as part of mine processes, or treated (if necessary) and discharged.

Containment has been sized according to the layout provided, with an assumed constant surface area of 6000m^2 being used. The necessary depth of the containment facility to contain both the 1:50 year design rainfall and the wettest month was inferred, with a final depth of 2.0m (excluding freeboard allowance). Table 5.2 presents the volume requirements for the dirty water containment facility.

TABLE 5.2: DIRTY WATER CONTAINMENT FACILITY VOLUME REQUIREMENTS FOR 1:50 YEAR FLOOD EVENT

Catchment	Minimum Volume (m^3)	Recommended Volume (m^3)
Dirty - A	7,600	12,000

The ‘minimum volume’ as presented in Table 5.2 is based purely on a single 1:50 year storm event while the ‘recommended volume’ includes the influence of the wettest month’s rainfall. The aforementioned do not account for any seepage losses, additions of process water, dewatering, spillages, wash water or the like. The storm water dams will therefore need to be operated empty to comply with GN 704.

In this project, the “recommended volumes” as presented in Table 5.2 were calculated based on the summation of the 1:50 year design rainfall (24 hour) event for the catchment area **and** the highest monthly rainfall falling over the catchment, **less** the corresponding monthly evaporation taking place over the surface area of the proposed containment facility. Runoff coefficients used were determined according to the return period of interest, such that maximum monthly rainfall event was associated with a smaller runoff coefficient than the 1 in 50 year design rainfall event. The “recommended volumes” as presented in Table 5.2 should be evaluated and revised (if necessary) as part of the detailed design phase of the project to include additional process water requirements.

5.2 HAUL ROADS

Figure 5.1 illustrates the roads present on site. These roads are assumed to be dirty areas which will require mitigation of runoff in some manner. For those roads located within *Dirty – A*, specific road side management of dirty water is unnecessary since the dirty water diversions and containment facility associated with *Dirty – A*, will be sufficient. The remaining roads (outside of *Dirty – A*) are access roads used for either haulage or other purposes. In the case of haul roads, these will be required to be appropriately constructed with sufficient erosion control measures and silt traps to contain sediment and reduce impact on receiving water resources. Any dirty water mitigation measures associated with haul roads should take cognisance of the coal fines that are likely to build up over time. A suitable road side drainage management plan will consequently be necessary to ensure employed mitigation continues to operate effectively.

6 CONCLUSIONS AND RECOMMENDATIONS

Appropriate baseline information including rainfall data, depth-duration-frequency design rainfall estimates, evaporation data as well as both regional and local hydrological characteristics have been considered for the proposed Commissiekraal project.

The Rainfall/Runoff response for the site was investigated using both WR2005 and WR2012 modelled data as well as a point streamflow estimate done during September 2015. The scaled down monthly quaternary catchment data (WR2012) for September seemed to overestimate (on average) the measured flow in the Pandana River. In order to gain a better understanding of the rainfall/runoff response at the site and associated groundwater/surface water interactions, a longer and more continuous streamflow data will be required. This will also assist in providing a baseline so that potential impacts resulting from the proposed mining operation on receiving water resources can be assessed over time. It is therefore recommended that a weir with associated streamflow gauge and rain gauge be installed on the Pandana River. For the interim period until a weir is installed it is recommended that at a minimum, monthly estimates of streamflow are taken using the methodology described in the SAS specialist report. Available water resources of the greater Pongola River catchment have been assessed at a high level. It is recommended that the DWS, as part of their custodianship of water resources in South Africa, further investigate the deficit/surplus of water resources within the affected catchments. This investigation should consider current legal/illegal abstractions as well as the reserve (basic human needs as well as ecological considerations) so that a more informed decision can be made.

The overall results of the flood modelling results illustrate the maximum depth anticipated for the 1:50 and 1:100 year events. The difference between the 1:50 and 1:100 year events is limited by the topography of the floodplains which rise quickly enough to constrain the spread of flood waters. As expected, the depth of flooding is greater in the Pandana River reaching a maximum of 5m. The two tributaries adjacent the site exhibit less flooding with depths under 1m associated with the west tributary where flood waters are able to spread out more over the floodplain. The east tributary exhibits deeper flooding associated with a more constrained floodplain.
In assessing the layout of infrastructure with regards to the illustrated 1:50 and 1:100 year flood extents, the only infrastructure located within the boundaries of the flood extents is access road which is positioned over the floodplain to the west of the site. Additional stream crossings and associated bridge and culvert designs have not been considered in this assessment but in principle, these crossing need to be sufficiently sized to provide capacity to convey the 1:100 year flood event over the expected life of the structure to minimise impacts and ensure that the natural flow regime can be maintained as far as possible.

The conceptual storm water management plan has been developed based on the requirements of GN 704. This was done by identifying clean and dirty areas and managing them accordingly. Dirty water producing areas have been isolated by diverting upstream clean water around them via clean water diversion berms and dirty water produced in dirty areas has been routed to dirty containment facilities. Infrastructure has been designed based on the contributing catchment areas and catchment characteristics, and has been sized to contain the 1:50 year flood event. It is recommended that discussions are held with the DWS regarding the lining requirements for storm water management infrastructure, to ensure that the flood hydrology calculations can be revised accordingly during detailed design and prior to construction of infrastructure. The “recommended volumes” of the proposed dirty storm water dams should be investigated further during the detail design phase to accommodate operational storage volumes, without compromising the ability of the dams to contain the “minimum volumes” as per GN 704 compliance. It is recommended that priority is given to the reuse of dirty water within the process water circuit. This will also ensure that the containment facilities are operated without compromising capacity and therefore, ability to contain the “minimum volumes” as per GN 704 compliance. In the case of haul roads, these will be required to be appropriately constructed with sufficient erosion control measures and silt traps to contain sediment and reduce impact on receiving water resources.

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(Project Manager/Author)	(Project Author)	(Project Reviewer)

DISCLAIMER

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APPENDIX A: FLOOD MODEL SETUP

A.1 MODEL INPUTS

PCSWMM is a model package that makes use of the United States Environmental Protection Agency (USEPA) Storm Water Management Model (SWMM), which is a computer program that computes dynamic rainfall-runoff from developed urban and undeveloped or rural areas (Rossman, 2008).

The SWMM model suits application to this project since it is able to account for:

- Time-varying rainfall;
- Rainfall interception in depression storage;
- Infiltration of rainfall into unsaturated soil layers;
- Routing of overland flow;
- Dynamic wave flow routing of flood waters; and
- Model surface flooding;

The suitability of PCSWMM as a full 2D flood model was assessed in comparison to the UK's Environment Agency Study (Neelz & Pender, 2013) - *Benchmarking the latest generation of 2D hydraulic modelling packages* (in which PCSWMM was not included). The conclusion of this assessment was that PCSWMM model results were comparable to those produced by other software vendors (James, et al., 2014).

A.2 DESIGN HYDROGRAPHS

A.2.1 DESIGN STORM

The SCS Type 3 design storm for South Africa was used to define the rainfall distribution according to the RLMA (Smithers /Schulze) 24 hour design rainfall depth for the 1 in 50 and 1 in 100 year events (see Table 2.2).

A.2.2 MODEL SETUP

The Department of Water and Sanitation (DWS) 25m DEM data for the area was used to develop a catchment model of the Pandana River up to the intended downstream boundary of the hydraulic model. A 100ha (1km^2) contributing area was used to define the minimum catchment areas from which to derive design flows.

In accordance with the WR2005 (WR2005, 2009) soils dataset, soils for the area were set as *sandy loams* with subsequent hydraulic parameters being derived from supporting literature[2]. Land cover parameters were estimated according to the dominant land type (i.e. grassland). Infiltration losses were estimated through the use of the Green-Ampt infiltration model.

The resulting hydrological model development is illustrated in Figure A.1

A.2.3 PEAK FLOWS AND HYDROGRAPHS

Since time varying rainfall was used, it was possible to develop both peak flows and hydrographs (consequently enabling an unsteady (dynamic) hydraulic model to be developed).

Two hydrographs representing the 1 in 100 year design events for the upstream and downstream boundaries on the Pandana River are illustrated in Figure A.2. Additional hydrographs were derived for the subcatchments and junctions as presented in Figure A.1. Table A.1 presents peak flows for the key junctions and subcatchments used to define inflows in the hydraulic model.

It is important to note, that no allowances for climate change have not been made. A risk analysis using the expected life of a structure or process will indicate the relevance of considering climate change (i.e. as the expected life increases the influence of climate change increases).

[2] http://www.dynsystem.com/NetSTORM/docs/GreenAmptParameters.html

FIGURE A.1: HYDROLOGIC AND HYDRAULIC MODEL SETUP

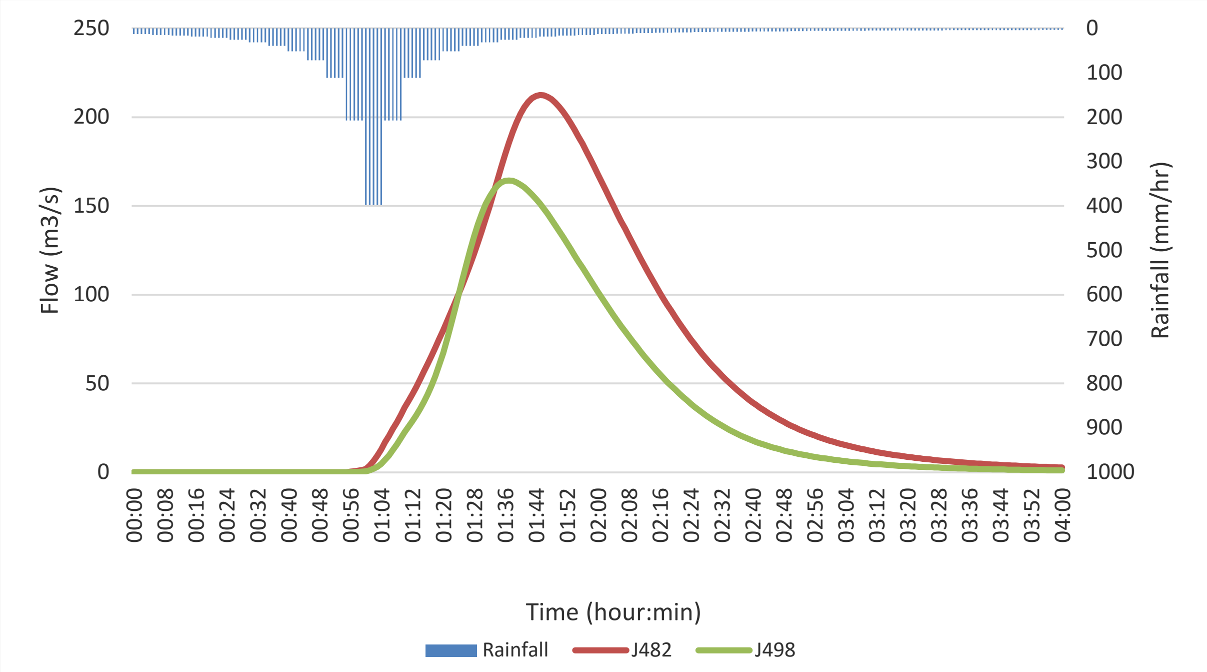

FIGURE A.2: 100-YEAR DESIGN HYDROGRAPHS FOR THE PANDANA RIVER

TABLE A.1: DESIGN PEAK FLOWS FOR KEY JUNCTIONS AND SUBCATCHMENTS

Model Label	Peak Flow (m^3/s)	
	50-Year Event	100-Year Event
J482	212.4	345.6
J498	164.3	258.4
J251	14.4	21.5
S173	7.0	10.6
S221	11.8	17.3
S179	6.8	10.2
S178	12.0	18.0
S152	6.4	9.4
S163	4.9	7.2
S156	7.8	11.5

A.2.4 Comparison to the Regional Maximum Flood

As a check on the PCSWMM peak flow estimates, the Regional Maximum Flood (RMF) (as outlined in the SANRAL Drainage Manual (SANRAL, 2006) was applied to the downstream boundary junction J482, using the Kovacs region 'K6'. The results of this comparison are presented in Table A.2.

TABLE A.2: RMF VERSUS PCSWMM COMPARISON FOR J482

Model Label	Peak Flow (m^3/s)	
	50-Year Event	100-Year Event
J482 (PCSWMM)	212.4	345.6
J482 (RMF)	309.0	393.0

In comparing the RMF results to those obtained from PCSWMM, it is apparent that the RMF estimates are higher for both the 50-year and 100-year events. Given that the RMF estimates represent the upper limits for their respective regions, it is expected that they will exceed the PCSWMM estimates. Nevertheless, the closer fit of the 100-year event results adds credibility to use of the PCSWMM peak flows estimates in study (since the 100-year event is the maximum event under consideration).

A.3 HYDRAULIC MODELLING

A.3.1 TOPOGRAPHIC DATA

An XYZ point dataset was provided as the primary topographic dataset for the study area and formed the foundation of the hydraulic model. The origin of the XYZ data (whether LIDAR, photogrammetry etc.) is unknown and details regarding its horizontal and vertical accuracy are absent. The XYZ data first required interpolation into a digital elevation model (DEM). Upon interrogating the XYZ data, it was noted that a uniform 5m grid spacing was used. The XYZ point dataset was consequently interpolated to a DEM using an inverse-distance weighting approach and a 5m cell size. The extent of the resulting DEM is presented in Figure A.1.

While a 5m DEM is fairly detailed, it will be limited in its ability to define features to an accuracy of less than 5m (horizontal). Consequently, features such as narrow bridges or river channels will be over-simplified and in some instance altogether absent from the DEM.

A.3.2 MODEL BOUNDARY

In developing a 2D PCSWMM model for the 3 streams of interest, it was necessary to define a channel boundary and a floodplain boundary. The purpose of these two boundaries is to enable the model to represent the channel in greater detail, and also enable the application of specific Manning's roughness values to the channel and floodplain. PCSWMM uses a 'wire mesh' to represent variations in topography, with intersections in the mesh representing points at which elevation is samples. In the instance of the channel boundary, a 5m mesh size was used which corresponded with the maximum level of detail from the DEM. In areas outside of the channel, the topography is less varied and a 10m mesh size was used. The extent of the aforementioned channel and floodplain boundaries are presented in Figure A.1.

A.3.3 ROUGHNESS COEFFICIENTS

Based on observations of the channel and floodplain characteristics from aerial topography a Manning's 'n' value of 0.04 was assigned to the floodplain and to the two non-perennials east and west of the site. A Manning's 'n' value of 0.04 corresponds with the grassland characterising the aforementioned area. For the Pandana River, a Manning's 'n' value of 0.06 was used in order to account for the presence of trees within the floodplain.

A.3.4 INITIAL FLOWS

The upstream boundaries of the model had initial flows added to improve model stability and account for a degree of streamflow prior to wet the channel before the main flood surge. Generalised flow values were used with $1m^3/s$ set as the intial flow of the two non-perennials, and $5m^3/s$ set as the initial flow of the Pandana River.

A.3.5 DOWNSTREAM BOUNDARY

A downstream boundary was included on the Pandana River in order to simulate the loss of water from the hydraulic model (since only a portion of the Pandana River was modelled). The downstream boundary outfalls were set to 'normal' which meant that the underlying elevation were sampled. In some instances the sampled elevations resulted in a negative slope, corresponding to a gain in elevation. Since a loss in elevation is expected along the downstream boundary, all negative slopes were assumed to be an inaccurate product of the DEM resolution and accuracy, and were manually altered to a negative slope.

A.4 MODEL RUN

Dynamic flow routing was set for the model run along with a variable time step (with a minimum of 0.2 seconds). The resulting routing continuity error was 0% which is the optimum result.

A.4.1 KEY ASSUMPTION IN THE HYDRAULIC MODEL

A number of assumptions have been made in undertaking the hydraulic modelling. These assumptions are in the context of the study and are considered appropriate in view of the level of detail required and the existing site conditions. The key assumptions include:

- That the topographic data provided was of a sufficient accuracy and coverage to enable hydraulic modelling at a suitable level of detail. The use of a 5m DEM results in a loss in detail of smaller features (under 10m) with features under 5m possibly not being included at all in the model. This is particularly relevant to narrow river sections where the loss of horizontal detail results in a generalisation of the river channel such that the full conveyance present in those sections cannot be accounted for. Loss of spatial detail in association with river channels will usually result in a more conservative flooding results since conveyance of the river channel is likely not captured.

- Hydraulic structures such as bridges and weirs were not modelled as part of this study. This limitation in the model is based on the understanding that only minor structures are likely to be present. The size of the peak flows occurring would consequently inundate any minor hydraulic structure present.
- The Manning's 'n' values used is considered suitable for use in both the 50 year and 100 year return periods modelled, as well as in representing both the channel and floodplain, for the reasons described in Section A.3.3.
- Unsteady state (dynamic) hydraulic modelling was undertaken, which accounts for the variation in flow caused by the surge in flood waters. This approach enables the influence of storage to be modelled such as the occurrence of wide floodplains which will cause a flattening out of a flood hydrograph.

APPENDIX B – STORM WATER MANAGEMENT CALCULATIONS

B.1 MODEL CHOICE

PCSWMM is a model package that makes use of the USEPA Storm Water Management Model (SWMM), which is a computer program that computes dynamic rainfall-runoff from developed urban and undeveloped or rural areas (Rossman, 2008).

The SWMM model suits application to this project since it is able to account for:

- Time-varying rainfall;
- Rainfall interception in depression storage;
- Infiltration of rainfall into unsaturated soil layers;
- Evaporation of standing surface water;
- Routing of overland flow; and
- Capture and retention of rainfall/runoff.

The development of SWMP's using SWMM has been undertaken for many thousands of studies through the world (Rossman, 2008).

B.2 MODEL SETUP

B.2.1 DESIGN STORM

The SCS Type 3 design storm for South Africa was used to define the rainfall distribution according to the RLMA (Smithers /Schulze) 24 hour design rainfall depth for the 1 in 50 and 1 in 100 year events (see Table 2.2).

B.2.2 MODEL PARAMETERISATION

The 5m DEM data for the site was used to separate dirty and clean areas (draining by gravity).
Land cover parameters were estimated according to the surface infrastructure layout.
In accordance with the WR2005 (WR2005, 2009) soils dataset, soils for the area were set as *sandy loams* with subsequent hydraulic parameters being derived from supporting literature[3]. Infiltration losses were estimated through the use of the Green-Ampt infiltration model.

[3] http://www.dynsystem.com/NetSTORM/docs/GreenAmptParameters.html

B.2 MODEL RUN

Kinematic wave routing was set for the model run along with a time step of 5 seconds. The resulting routing continuity error was 0% which is the optimum result, while the runoff continuity error was -0.4% which is close to optimum. The resulting peak flows and characteristics for the dirty and clean areas is presented in Table B.1.

TABLE B.1: CLEAN AND DIRTY AREA CHARACTERISTICS FOR THE 1:50 YEAR EVENT

Catchment	Area (ha)	Slope (%)	Rainfall (mm)	Infiltration (mm)	Runoff Coefficient	Peak Flow (m^3/s)
Clean Water A	2.55	13.1	191	125	0.35	1.5
Dirty Water A	5.81	10.0	191	81	0.58	4.3