PWMA12/T30/00/5212/12

water & sanitation

Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

DIRECTORATE: OPTIONS ANALYSIS

FEASIBILITY STUDY FOR THE MZIMVUBU WATER PROJECT

FEASIBILITY DESIGN: NTABELANGA DAM

OCTOBER 2014

FEASIBILITY STUDY FOR THE MZIMVUBU WATER PROJECT

APPROVAL

Report title:	***Feasibility Design: Ntabelanga Dam***
Authors:	*A Pepperell, T Speirs, C Brand* *and various other team members*
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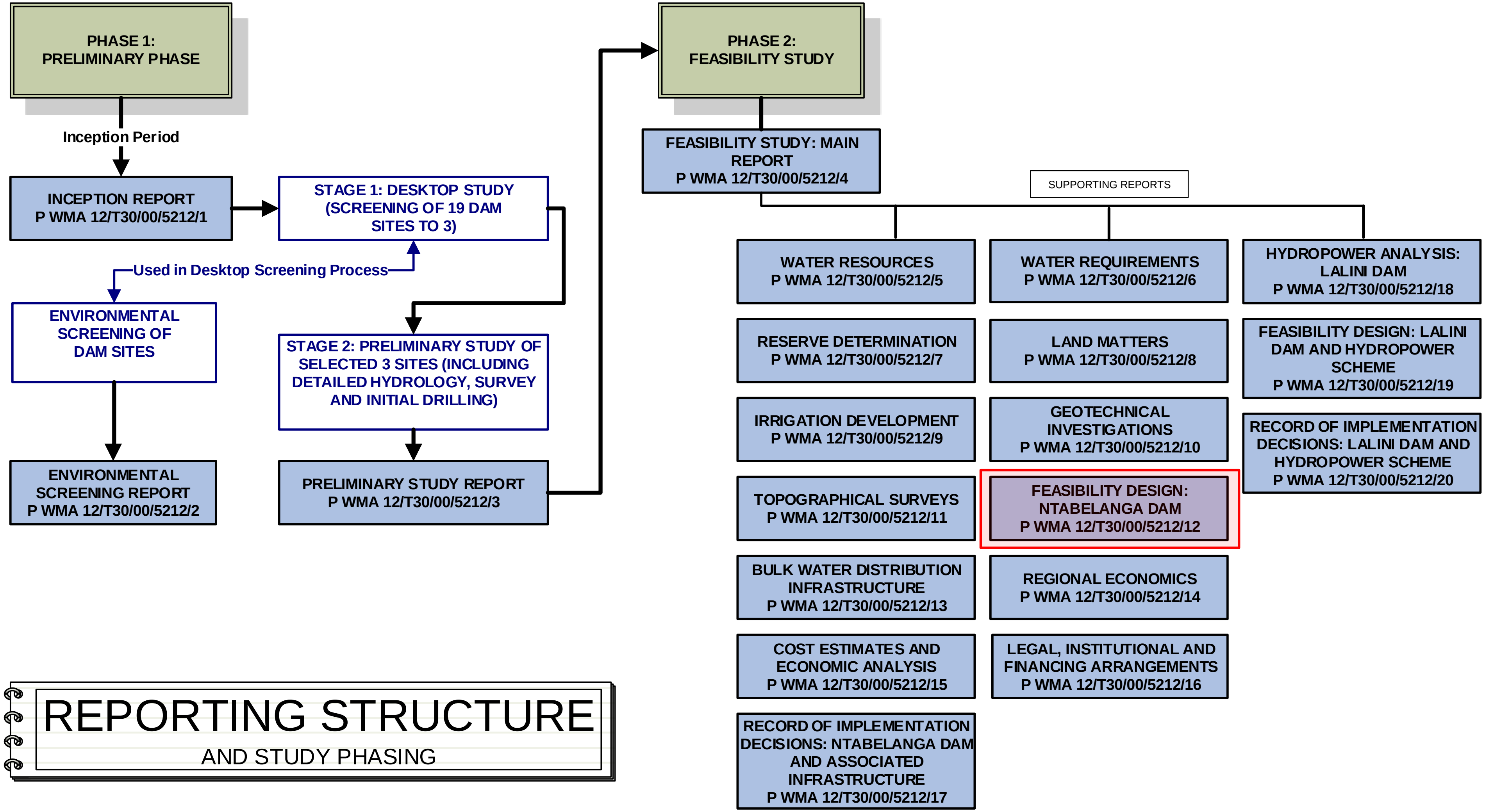
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REFERENCE

This report is to be referred to in bibliographies as:

Department of Water and Sanitation, South Africa (2014). ***Feasibility Study for the Mzimvubu Water Project: Feasibility Design: Ntabelanga Dam***

DWS Report No: P WMA 12/T30/00/5212/12

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Note on Departmental Name Change:

In 2014, the Department of Water Affairs changed its name to the Department of Water and Sanitation, which happened during the course of this study. In some cases this was after some of the study reports had been finalized. The reader should therefore kindly note that references to the Department of Water Affairs and the Department of Water and Sanitation herein should be considered to be one and the same.

Note on Spelling of Laleni:

The settlement named Laleni on maps issued by the Surveyor General is locally known as Lalini and both names therefore refer to the same settlement.

EXECUTIVE SUMMARY

INTRODUCTION

The Mzimvubu River catchment in the Eastern Cape Province of South Africa is within one of the poorest and least developed regions of the country. Development of the area to accelerate the social and economic upliftment of the people was therefore identified as one of the priority initiatives of the Eastern Cape Provincial Government.

Harnessing the water resources of the Mzimvubu River, the only major river in the country which is still largely unutilised, is considered by the Eastern Cape Provincial Government as offering one of the best opportunities in the Province to achieve such development. In 2007, a special-purpose vehicle (SPV) called ASGISA-Eastem Cape (Pty) Ltd (ASGISA-EC) was formed in terms of the Companies Act to initiate planning and to facilitate and drive the Mzimvubu River Water Resources Development.

The five pillars on which the Eastern Cape Provincial Government and ASGISA-EC proposed to model the Mzimvubu River Water Resources Development are:

- *Forestry;*
- *Irrigation;*
- *Hydropower;*
- *Water transfer; and*
- *Tourism.*

As a result of this the Department of Water and Sanitation (DWS) commissioned the Mzimvubu Water Project with the overarching aim of developing water resources schemes (dams) that can be multi-purpose reservoirs in order to provide benefits to the surrounding communities and to provide a stimulus for the regional economy, in terms of irrigation, forestry, domestic water supply and the potential for hydropower generation amongst others.

The study commenced in January 2012 and was completed by October 2014 in three stages as follows:

- *Inception;*
- *Phase 1 – Preliminary Study; and*
- *Phase 2 – Feasibility Study.*

The purpose of this study is not to repeat or restate the research and analyses undertaken on the several key previous studies described below, but to make use of that information previously collected, to update and add to this information, and to undertake more focussed and detailed investigations and feasibility level analyses on the dam site options that have then been identified as being the most promising and cost beneficial.

PURPOSE OF REPORT

This report describes the process taken to develop an optimum selection of dam location, dam type, spillway type, and the feasibility level design of the selected type of dam, at the Ntabelanga site that was selected in Phase 1, as described in the Preliminary Study Report No. P WMA 12/T30/00/5212/3.

It was confirmed and agreed that the sizing and modus operandi of the Ntabelanga Dam and its associated works would take into account its multi-purpose role, namely:

- i. *To supply potable water to an estimated current population of 502 822 people (rising to some 726 616 people in 2050), and other potable water consumers in the region;*
- ii. *To supply raw water for irrigation of some 2 868 ha of high potential agricultural land;*
- iii. *To generate hydropower locally at the dam wall to reduce the cost of energy consumption when pumping water;*
- iv. *To provide sufficient flow of water downstream of the Ntabelanga Dam to meet environmental water requirements for an ecological Class C; and*
- v. *To provide additional balancing storage volume and consistent downstream flow releases to enable a second dam at Lalini (just above the Tsitsa Falls) to generate significant hydropower for supply into the national grid.*

These multi-purpose usages and requirements for the Ntabelanga Dam are described in the Water Requirements Report No. P WMA 12/T30/00/5212/6, and the Irrigation Development Report No. P WMA 12/T30/00/5212/9.

DAM LOCATION

A review of the location of the Ntabelanga Dam wall, identified both in previous studies and in Phase 1 of this study, was undertaken both using topographical mapping as well as field reconnaissance.

The proposed Ntabelanga Dam is located approximately 55 km north of Mthatha on the Tsitsa River, at co-ordinates 31° 7' 1.40"S, 28°40' 20.45"E.

It was concluded that there were no better upstream dam wall locations available with regard to river valley shape (which affects dam wall length), geology/founding conditions, close proximity to construction materials, and the depth verses volume characteristics of the impoundment.

Both upstream and downstream of the primary dam site, the valley widens and flattens, and the next suitable dam site location downstream was actually one of the others previously eliminated in the Phase 1 screening process (Malepelepe).

Therefore the more detailed Ntabelanga Dam wall siting investigations focussed on the narrowest part of the Tsitsa River valley just before the valley widens.

Phase 1 drilling was undertaken on an alignment selected on site by the study team Engineering Geologist and which had the optimum geomorphology in this reach of the river. The two holes drilled in Phase 1 indicated very good foundation conditions on competent dolerite, with a competent dolerite right flank abutment, and a combination of dolerite overlain by sandstone (at higher elevation) on the left flank abutment.

Further site investigations (core drilling) and materials trial pitting and sampling were carried out on these alignments, and potential spillway locations, in Phase 2, and these are described in detail in Geotechnical Investigations Report No. P WMA 12/T30/00/5212/10.

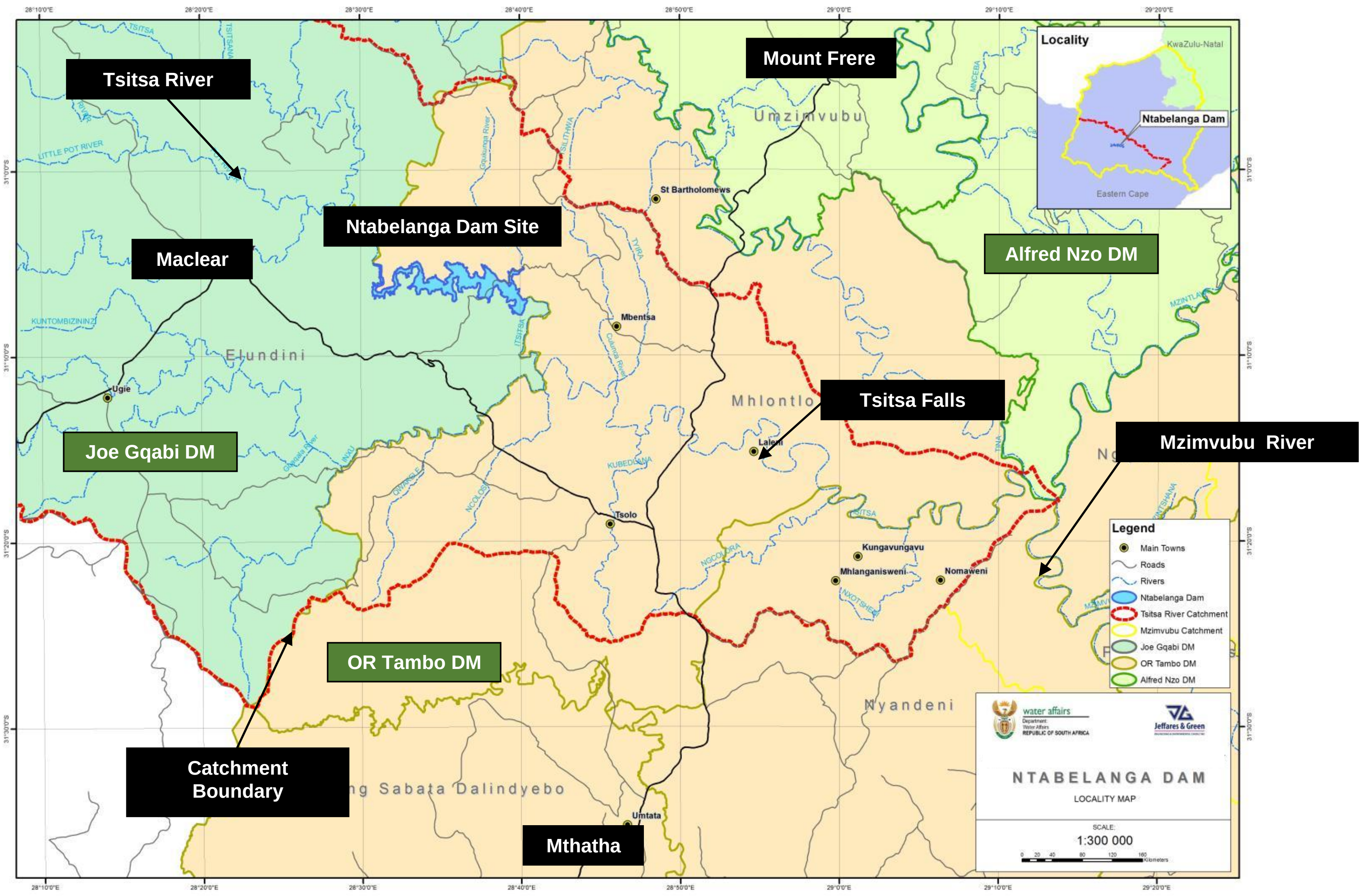


Figure 1: Location of Ntabelanga Dam

DAM TYPE ANALYSIS

It was deemed important to consider the range of possible dam type options before committing to the further core drilling to be undertaken in Phase 2. The selected dam type options also determined what other geotechnical investigations (including materials sourcing and geophysics) should be undertaken in parallel with the core drilling.

All previous studies and Phase 1 of this study had considered only earth embankment/clay core (earth fill) options. In this feasibility analysis, the study team considered several other options, as well as various spillway arrangements.

The following dam types were investigated in Phase 2:

- *Roller compacted concrete (RCC) dam;*
- *Concrete faced rock fill dam (CFRD);*
- *Earth core rock fill dam (ECRD);*
- *Earth fill embankment dam with earth core (EF); and*
- *Composite central concrete gravity spillway/embankment flank options (CCS).*

Further options regarding the spillway alternatives of left or right bank side channels, channels cut through the hill, or central spillway were also investigated.

Key factors used in determining the optimum dam type were as follows:

- *Availability of sufficient quantities and quality of construction materials in the vicinity of the dam wall;*
- *Constructability issues, especially relating to dealing with river flow during construction;*
- *The ability of DWS to design and construct the dam in-house;*
- *Spillway location and capacity requirements;*
- *Operational requirements and outlet works arrangements;*
- *Environmental impacts; and*
- *The cost of the works.*

In order to assess materials requirements, quantities were calculated for all of the above dam types, based upon typical design criteria (foundation excavation depths, embankment slopes, etc), which were undertaken for all of the above dam types and their spillway options.

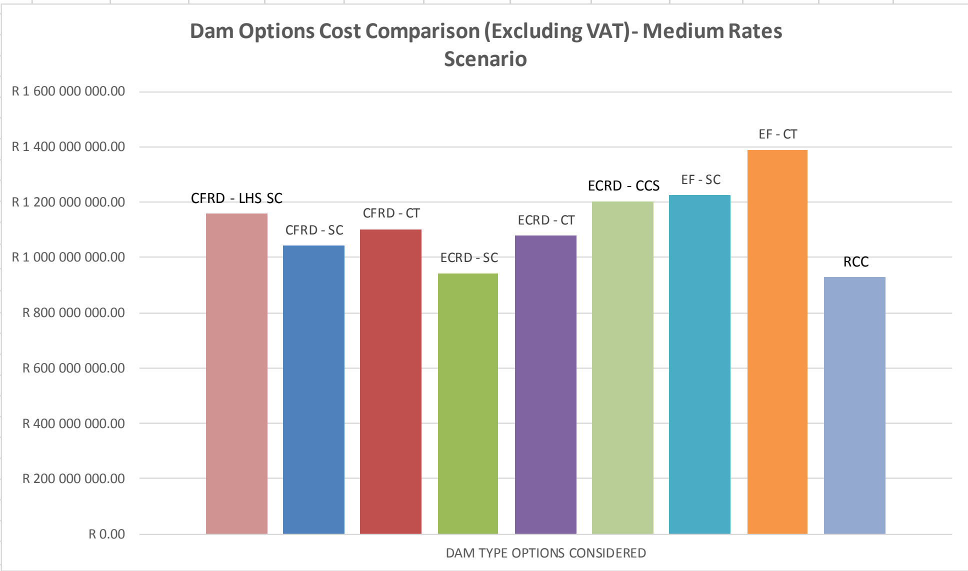
The results of these analyses produced a ranking of dam types as shown in Table 1.

Table 1: Capital Cost Comparison of Dam Type and Spillway Options

Option No.	Dam Wall Type	Spillway Type	Capital Cost (Rmillion)		
			Low	Medium	High
1	CFRD	Side Channel (SC)on Right Flank	932	1 043	1 153
2	CFRD	Cut-Through	989	1 103	1 218
3	CFRD	SC Left	1 036	1 158	1 279
4	**ECRD**	**SC Right**	**848**	**944**	**1 040**
5	ECRD	CT	977	1 079	1 181
6	Earth fill	SC Right	1 147	1 224	1 301
7	Earth fill	CT	1 305	1 390	1 474
8	**RCC**	**Central Ogee**	**769**	**929**	**1 089**
9	CCS	Composite Central Channel Spillway	1 009	1 203	1 397

(green)	Lowest
(yellow)	Second Lowest

The green highlighted cells show the lowest cost option. For the low and medium rate ranges of major quantity unit rates this is Option No. 8, an RCC dam, with Option No.4, the ECRD dam with a Side Channel Spillway cut through the Right-hand Flank, coming second lowest. Only for the highest rates does this ranking reverse. Figure 2 shows the comparative costs of all the options for the medium rates case, as well as main materials quantity information and how much excavated material needs to be disposed of to spoil.



Percentage of lowest cost option	125%	112%	119%	102%	116%	130%	132%	150%	100%
Cost Excluding VAT R'millions	**1 158**	**1 043**	**1 103**	**944**	**1 079**	**1 203**	**1 224**	**1 390**	**929**
Total rock excavation used in embankment	450 000	780 400	1 186 179	815 600	927 600	166 700	0	0	382 381
Total rock excavation to spoil	0	0	452 500	0	737 300	0	858 400	1 716 650	39 413
Total all materials to spoil	187 700	79 000	610 500	79 000	895 300	23 000	978 200	1 915 350	0

Figure 2: Dam Options Cost Comparison

*As can be seen for the "medium rates" scenario, which is considered to be a reasonable assumption given the nature of the dam site and proximity to construction materials, the **RCC** and **ECRD (with right hand side channel spillway)** options are ranked very closely, with all other options more than 10% higher in cost.*

It is therefore concluded that there is little to choose between these two options as far as costs are concerned, and other factors were therefore considered to inform the decision-making process.

OTHER DAM TYPE SELECTION CONSIDERATIONS

The following considerations were made:

- *Ability to build in stages if a smaller dam is built first and then raised;*
- *Speed of implementation to first water delivery;*
- *Ability of DWS Infrastructure Branch to undertake detailed design in-house;*
- *Ability of DWS construction unit division to undertake construction in-house;*
- *Simplified infrastructure layout and access;*
- *Low maintenance inputs;*
- *Less risk when dealing with floods during construction; and*
- *Environmental impacts.*

CONCLUSION ON DAM TYPE SELECTION

Taking the various decision-making factors into consideration, it is concluded that the preferred dam type is the RCC solution.

This would provide for a simplified operational layout, better aesthetics and less environmental impact than an ECRD dam with a side channel spillway, and would offer the better opportunity for implementation in a shorter time period.

The fact that the DWS Infrastructure Branch is considering the implementation of the project in-house to reduce the implementation time, and that they have more experience with RCC technology than rock-fill, would further justify the preference of RCC as the dam type to be implemented.

*Therefore the dam and ancillary works that will be further described in the following sections are based on the **RCC solution**.*

The draft Scope of Work for detailed design of the works allows for a further review of the dam type and this decision will therefore be re-evaluated in the detailed design stage in the light of more detailed analysis based on additional geotechnical information.

A general arrangement and elevations of the proposed RCC dam solution is given in Figures 3 and 4.

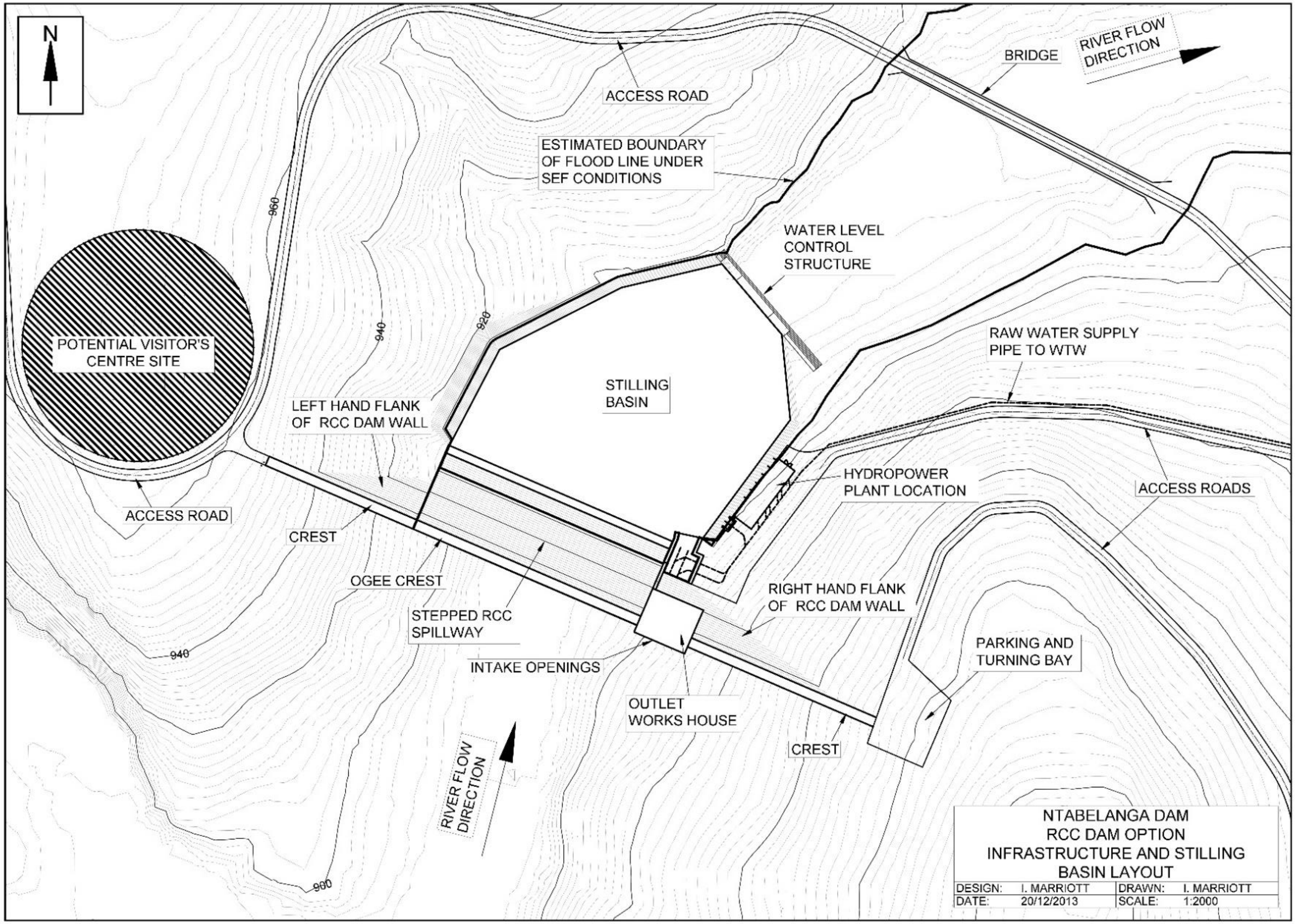


Figure 3: Proposed RCC Dam Layout Plan

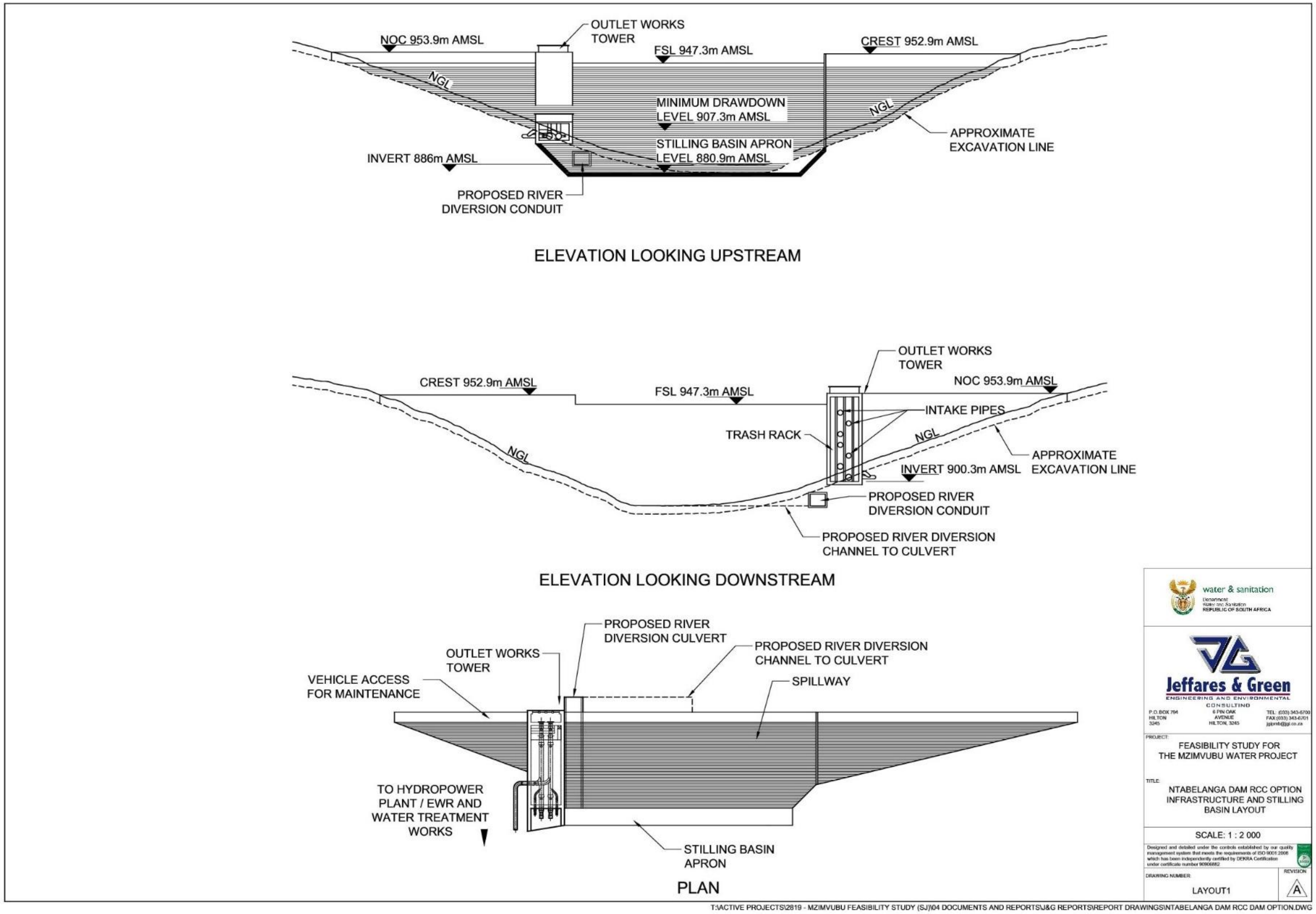


Figure 4: Proposed RCC Dam Elevations

DAM CHARACTERISTICS

The proposed Ntabelanga Dam has the following characteristics:

Full Supply Level (FSL):	*947.3 m.a.s.l.*
Non-Overspill Crest Level – right flank (NOCL):	*953.9 m.a.s.l.*
Minimum bed level in river at dam:	*886.7 m.a.s.l.*
Crest width:	*6 m*
Minimum operating level (MOL):	*918.00 m.a.s.l.*
Emergency drawdown minimum outlet level:	*907.00 m.a.s.l.*
Maximum dam wall height to NOC:	*66.1 m*
Wall crest length (incl. spillway):	*407 m*
Spillway crest length:	*150 m*
Gross stored volume at FSL:	*490 million m^3*
Mean Annual Runoff at dam:	*415 million m^3*
Storage below MOL (V_{50} sedimentation):	*37 million m^3*
Surface area of lake behind dam:	*31.5 km^2*
Backwater reach upstream of dam:	*15.5 km*

The dam wall height, impoundment volume, and downstream risk factors for the Ntabelanga Dam put this structure into a Category 3 dam under Gazetted Dam Safety Guidelines.

The flood criteria for design of this dam are as follows:

1 in 200 year return period Design Flood:	*2 500 m^3/sec*
Safety Evaluation Flood (SEF):	*5 530 m^3/sec*

The above dam capacity fully meets the potable and irrigation water requirements as well as providing regulated flow releases in the river below the dam to meet the EWR requirements, to generate an average of 1.6 MW of hydropower at the dam wall, and to assure sufficient river flow downstream for the Lalini Dam and Hydropower Scheme.

FEASIBILITY DESIGN

As described in other reports in this series, the dam will have the following purposes:

- *Potable water supply to a new water treatment works with a capacity of 102 000 m^3/day (and a bulk water distribution system supplying some 726 616 people in the year 2050);*
- *Raw water supply to 2 868 ha of high potential irrigable land, mostly in the Tsolo area;*
- *Generation of hydropower ranging seasonally from 0.75 MW to a peak of 5.0 MW;*
- *Maintaining Environmental Water Releases downstream of the dam; and*
- *Releasing water downstream to supplement flow to a potential hydropower scheme at the Lalini Dam site.*

The feasibility design section of this report describes the design process for the dam, its outlet works, pumping stations and conveyance systems supplying water to the infrastructure above, as well as the hydropower plant at the dam itself.

ASSOCIATED INFRASTRUCTURE

In addition to the dam and its outlet and conveyance works, the feasibility design also includes the layouts and requirements for the following associated infrastructure:

- *Water treatment works location;*
- *Raw water pump station to the irrigation systems;*
- *Staff Housing;*
- *Local road upgrades and realignments;*
- *Road bridge across the river downstream of the dam;*
- *Wastewater treatment plant;*
- *Temporary water supply;*
- *Main access roads to national roads;*
- *EWR release facility;*
- *Hydropower plant;*
- *Flow gauging stations;*
- *Power supplies;*
- *Other access roads to dam crest; and*
- *Potential location of a Visitor's Centre.*

An overall perspective of the dam and its associated infrastructure is given in Figure 5.

Budget provisions have also been allowed for a 10 year land care and catchment management programme which is being undertaken by the Eastern Cape Department of Environmental Affairs, as well as the potential funding of in-field equipment and development of the proposed irrigated agriculture farming units.

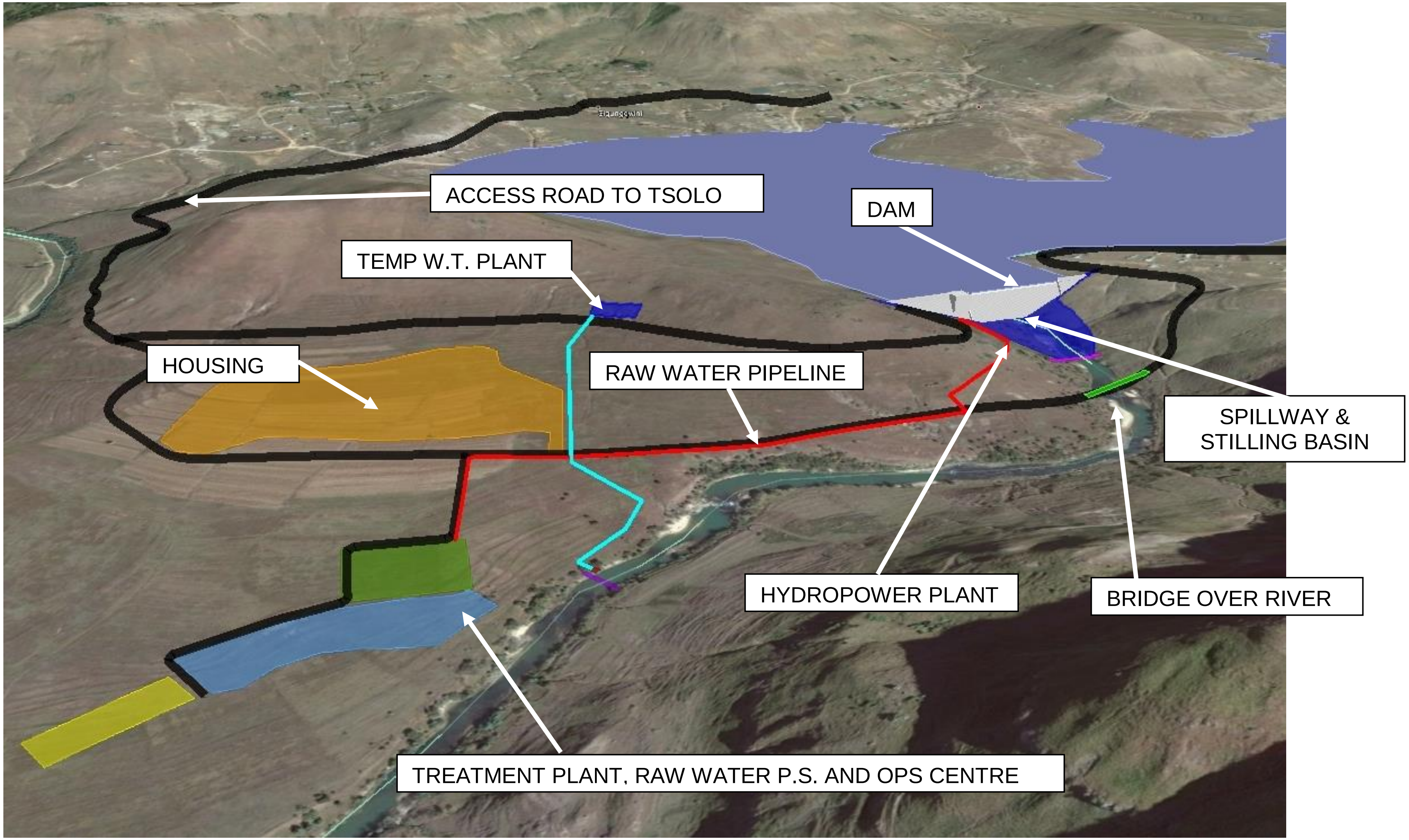


Figure 5: Aerial perspective of the Ntabelanga Dam and Associated Infrastructure

COST ESTIMATE

The cost estimate for the Ntabelanga Dam and its associated infrastructure, water supply and irrigation schemes, land care programme, and in-field development of irrigated farming units, is given in Table 2.

This does not include any of the Lalini Dam and hydropower scheme infrastructure which is dealt with in a separate Report No. P WMA 12/T30/00/5212/19. This dam is, however, sized to provide adequate flow releases downstream when operating conjunctively with the Lalini Hydropower scheme component.

Table 2: Capital Cost Estimates

COMPONENT	R'million
Ntabelanga dam and associated works	1 075
Ntabelanga dam hydropower works	88
Ntabelanga land compensation/mitigation costs	18
Ntabelanga power transmission	29
Sub-Total Ntabelanga Dam and Associated Works	1 209
Engineering and EMP Costs (12%)	145
Sub-Total Ntabelanga Dam and Associated Works incl Eng & EMP	1 354
Escalation in Each Year @ 5.5% p.a.	265
Sub-Total Ntabelanga Dam and Associated Works incl Eng, EMP & ESC	1 619
VAT (14%)	227
Add in R22 million per year for catchment management (no esc)	220
Allowance for other offset activities (50% of R100 million)	50
Total Ntabelanga Dam and Associated Works (incl Esc + VAT)	**2 116**
COMPONENT	**R'million**
Ntabelanga water treatment works	643
Ntabelanga primary & secondary bulk treated water distribution system	1 234
Ntabelanga tertiary bulk treated water distribution system (DM's)	1 425
Ntabelanga bulk irrigation water supply system	497
Sub-Total Ntabelanga WTW and Bulk Water Systems	3 799
Engineering and EMP Costs (12%)	456
Sub-Total Ntabelanga WTW and Bulk Water Systems incl Eng & EMP	4 255
Escalation in Each Year @ 5.5% p.a.	1 067
Sub-Total Ntabelanga WTW and Bulk Water Systems incl Eng, EMP & ESC	5 322
VAT (14%)	745
Total Ntabelanga WTW and Bulk Water Systems (incl Esc + VAT)	**6 068**

…. (cont.)

Table 2: Capital Cost Estimates (cont.)

COMPONENT	R'million
In-farm irrigation investment costs	***105***
Engineering and EMP Costs(12%)	*13*
Sub-Total in-farm irrigation investment costs incl Eng & EMP	*118*
Escalation in Each Year @ 5.5% p.a.	*40*
Sub-Total in-farm irrigation investment costs incl Eng, EMP & ESC	*158*
VAT (14%)	*22*
Total in-farm irrigation investment costs (incl Esc + VAT)	***180***
GRAND TOTAL NTABELANGA (R'MILLION INCL ESC AND VAT)	***8 364***

More detailed costing breakdowns and cash flow projections for each individual project component are given in Report No. P WMA 12/T30/00/5212/15. It should be noted that there are several risks involved in the accuracy of the above cost estimate:

- *Estimating at feasibility level at best has a confidence level of ± 10%*
- *Escalation rates could increase or decrease, especially given the volatile nature of the economy at the moment*
- *Rand foreign exchange rates are also volatile and this will affect the cost of all imported materials, services and equipment.*
- *The timing of the various components implementation may change which, if later, would increase the escalation cost.*
- *The amount of non-grant finance is unknown, and if significant will increase costs, depending on the terms of such loans, interest rates and foreign exchange rates.*

One example of the impact of the above risks is that every month's delay in fully implementing a R8.4 billion project increases escalation cost by R38.5 million (at 5.5% p.a.)

ESTIMATED OPERATION AND MAINTENANCE COSTS

Operation and maintenance costs will to some extent depend upon the institutional arrangements set up to operate the scheme, and the structures and management costs of the one or more entities involved. Economies of scale can be lost if the management and operation of the works is split between several different organisations.

An estimate has been made of the likely management, maintenance and operational costs of these works based upon current costs and salary scales.

Maintenance costs per annum are based upon the percentages of capital cost recommended in DWS's Water Supply Planning and Design Guidelines. Operational staffing costs have been sourced from those currently applied to similar works operated by Amatola Water.

Energy costs (pumping, etc) are based upon an average tariff per kWh using ESKOM's Ruraflex tariff, and assuming that pumping would be restricted to non-peak hours (i.e. up to 19 hours pumping per day). This is the current tariff used for pumping by Amatola Water in this region.

Table 3 summarizes these annual operating and maintenance costs, but these should be treated with caution pending decisions being made on the eventual institutional arrangements.

Table 3: Annual Management, Operation, and Maintenance Cost Estimate (2014 Price Levels)

COMPONENT	ANNUAL MAINTENANCE COSTS R'MILLION	ANNUAL OPS STAFFING COSTS R'MILLION	POWER COSTS/ANNUM R'MILLION		TREATMENT COSTS/ANNUM R'MILLION
			ON COMMISSIONING	BY 2050	
NTABELANGA DAM + MINI HYDRO + ASSOCIATED INFRASTRUCTURE	*8*	*4.2*	*3*	*3*	
NTABELANGA WTW AND POTABLE BULK WATER SYSTEM (PRIMARY ONLY)	*20.1*	*12.3*	*36*	*48.9*	*7.7*
NTABELANGA POTABLE BULK WATER SYSTEM (SECONDARY)	*9*	*4.1*	*2.5*	*3*	
NTABELANGA POTABLE BULK WATER SYSTEM (TERTIARY)	*12*	*11.6*	*1.5*	*2*	
NTABELANGA IRRIGATION SYSTEM (DELIVERY TO EDGE OF FIELDS)	*5.3*	*2.5*	*18.6*	*18.6*	
LALINI DAM AND HYDROPOWER SCHEME	*29.9*	*6.8*	*3*	*3*	
TOTALS: R'MILLION/ANNUM	***84.3***	***41.5***	***64.6***	***78.5***	***7.7***

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LIST OF ACRONYMS AND ABBREVIATIONS

ASGISA-EC	Accelerated and Shared Growth Initiative for South Africa – Eastern Cape
CAPEX	Capital Expenditure
CFRD	Concrete-faced rock fill dam
CMA	Catchment Management Agency
CTC	Cost to Company
CV	Coefficient of Variability
DAFF	Department of Agriculture, Forestry and Fisheries
DBSA	Development Bank of Southern Africa
DEA	Department of Environment Affairs
dia.	Diameter of a pipe
DM	District Municipality
DME	Department of Minerals and Energy
DoE	Department of Energy
DRDAR	Department of Rural Development and Agrarian Reform
DRDLR	Department of Rural Development and Land Reform
DWA	Department of Water Affairs
DWS	Department of Water and Sanitation
EA	Environmental Authorisation
EAP	Environmental Assessment Practitioner
EC	Eastern Cape
ECRD	Earth core rock fill dam
EF	Earth fill (dam)
EIA	Environmental Impact Assessment
EMP	Environmental Management Plan
EPWP	Expanded Public Works Programme
ESIA	Environmental and Social Impact Assessment
EWR	Environmental Water Requirements
FSL	Full Supply Level
GERCC	Grout enriched RCC
GN	Government Notices
GW	Gigawatt
GWh/a	Gigawatt hour per annum
IAPs	Invasive Alien Plants
IB	Irrigation Board
IFC	International Finance Corporation
IPP	Independent Power Producer
IRR	Internal Rate of Return
IVRCC	Internally vibrated RCC
ISO	International Standards Organisation
kW	Kilowatt
LM	Local Municipality
ℓ/s	Litres per second
ℓ/c/d	Litres per capita per day

MAP	Mean Annual Precipitation
MAR	Mean Annual Runoff
MEC	Member of the Executive Council
MIG	Municipal Infrastructure Grant
million m^3	Million cubic metres
MW	Megawatt
NEMA	National Environmental Management Act
NERSA	National Energy Regulator of South Africa
NHRA	National Heritage Resources Act
NOCL	Non-overspill crest level
NWA	National Water Act
NWPR	National Water Policy Review
NWRMS	National Water Resources Management Strategy
O&M	Operations and Maintenance
OPEX	Operational Expenditure
PICC	Presidential Infrastructure Co-Ordinating Committee
PPA	Power Purchase Agreement
PPP	Public Private Partnership
PSC	Project Steering Committee
PSP	Professional Services Provider
RBIG	Regional Bulk Infrastructure Grant
RCC	Roller-compacted concrete
REIPPPP	Renewable Energy Independent Power Producer Procurement Programme
RWI	Regional Water Institution
RWU	Regional Water Utilities
SAWS	South African Weather Service
SEZ	Special Economic Zone
SIP	Strategic Integrated Project
SMC	Study Management Committee
SPV	Special Purpose Vehicle
TCTA	Trans Caledon Tunnel Authority
ToR	Terms of Reference
UOS	Use of System
URV	Unit Reference Value
WEF	Water Energy Food
WRYM	Water Resources Yield Model
WSA	Water Services Authority
WSP	Water Services Provider
WTE	Water Trade Entity
WUA	Water User Association

LIST OF UNITS

Description	Standard unit
Elevation	m a.s.l.
Height	m
Distance	m, km
Dimension	mm, m
Area	m^2 , ha or km^2
Volume (storage)	m^3
Yield, Mean Annual Runoff	m^3/a
Rotational speed	rpm
Head of Water	m
Pressure	Pa
Diameter	mm or m
Temperature	ºC

Description	Standard unit
Velocity, speed	m/s, km/hr
Discharge	m^3/s
Mass	kg, tonne
Force, weight	N
Gradient (V:H)	%
Slope (H:V) or (V:H)	1:5 (H:V) or 5:1 (V:H)
Volt	V
Power	W
Energy used	kWh
Acceleration	m/s^2
Density	kg/m^3
Frequency	Hz

1. BACKGROUND AND INTRODUCTION

The Mzimvubu River catchment in the Eastern Cape Province of South Africa is situated in one of the poorest and least developed regions of the country. Development of the area to accelerate the social and economic upliftment of the people was therefore identified as one of the priority initiatives of the Eastern Cape Provincial Government.

Harnessing the water resources of the Mzimvubu River, the only major river in the country which is still largely unutilised, is considered by the Eastern Cape Provincial Government as offering one of the best opportunities in the Province to achieve such development. In 2007, a special-purpose vehicle (SPV) called ASGISA-Eastem Cape (Pty) Ltd (ASGISA-EC) was formed in terms of the Companies Act to initiate planning and to facilitate and drive the Mzimvubu River Water Resources Development.

The five pillars on which the Eastern Cape Provincial Government and ASGISA-EC proposed to model the Mzimvubu River Water Resources Development are:

- Forestry;
- Irrigation;
- Hydropower;
- Water transfer; and
- Tourism.

The Department of Water and Sanitation (DWS) commissioned the Mzimvubu Water Project with the overarching aim of developing water resources schemes (dams) that can be multi-purpose reservoirs in order to provide benefits to the surrounding communities and to provide a stimulus for the regional economy, in terms of irrigation, forestry, domestic water supply and the potential for hydropower generation amongst others.

1.1 Study Locality

The Mzimvubu River Catchment is situated in the Eastern Cape (EC) Province of South Africa which consists of six District Municipalities (DM) and two Metropolitan Municipalities (Buffalo City and Nelson Mandela Bay). These include Cacadu DM in the west across to the Alfred Nzo DM in the east with the two Metropolitan Areas being located around the two major centres of the province, East London and Port Elizabeth, both of which border the Indian Ocean.

The Mzimvubu River Catchment is situated within three of the DM’s namely the Joe Gqabi DM in the north-west, the OR Tambo DM in the South and the Alfred Nzo DM in the east and north east. A locality map of the whole catchment area and its position in relation to the DM’s in the area is provided in Figure 1-1 overleaf.

The study commenced in January 2012 and was completed in October 2014 in three stages as follows:

- Inception;
- Phase 1 – Preliminary Study; and
- Phase 2 – Feasibility Study.

The purpose of the study is not to repeat or restate the research and analyses undertaken on the several key previous studies described below, but to make use of that information previously collected, to update and add to this information, and to undertake more focussed and detailed investigations and feasibility level analyses for the dam site options identified as being the most promising and cost beneficial.

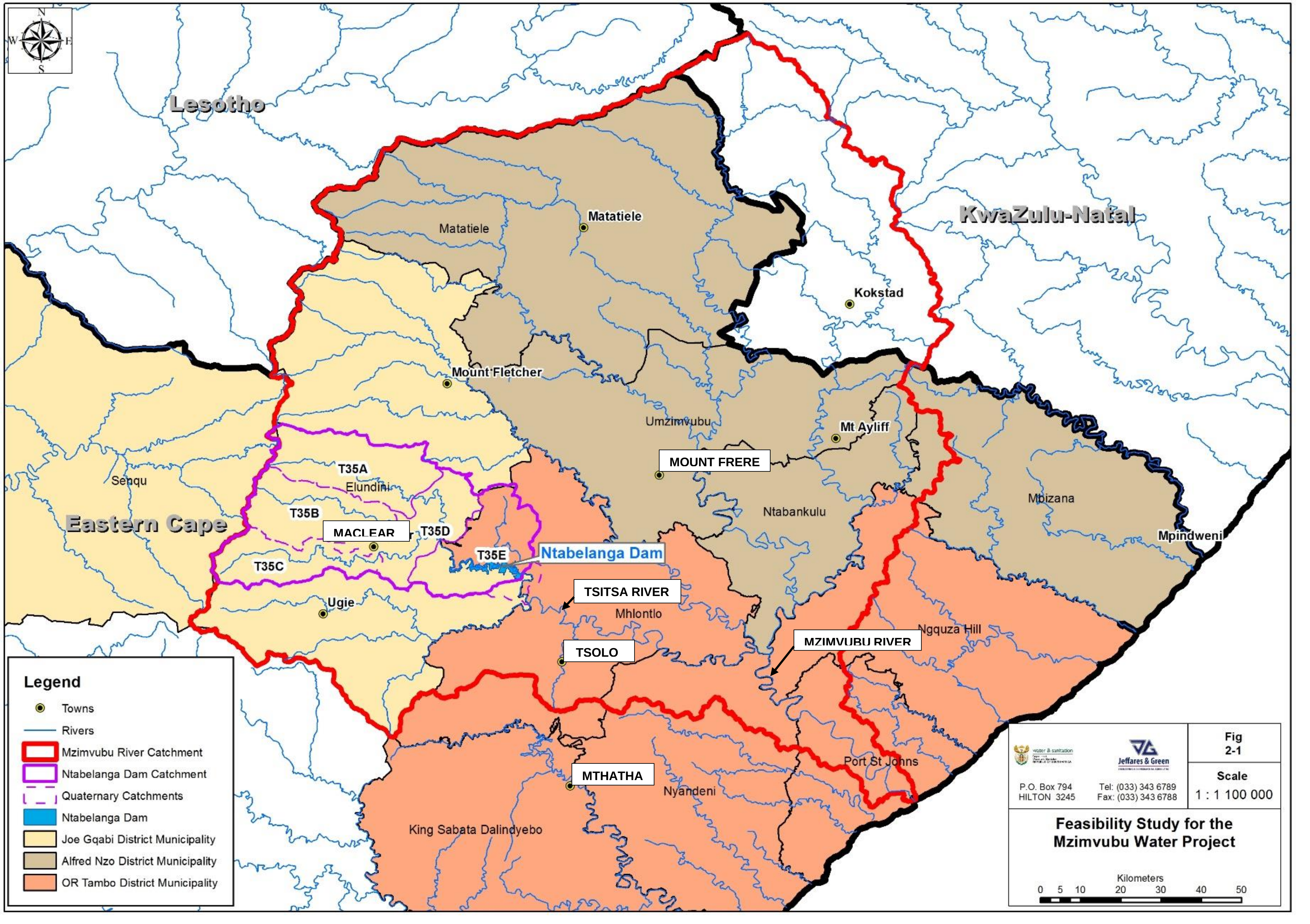


Figure 1-1: Locality Map of the Mzimvubu Catchment

1.1.1 *Inception Phase*

The aim of the inception phase was to finalise the Terms of Reference (TOR) as well as to include, *inter alia*, the following:

- A detailed review of all the data and information sources available for the assignment;
- A revised study methodology and scope of work;
- A detailed review of the proposed project schedule, work plan and work breakdown structure indicating major milestones;
- Provision of an updated organogram and human resources schedule; and
- Provision of an updated project budget and monthly cash flow projections.

The inception phase has been completed and culminated in the production of an inception report (DWS Report Number P WMA 12/T30/00/5212/1) which also constitutes the final TOR for the study.

1.1.2 *Preliminary Study Phase*

The preliminary report describes the activities undertaken during the preliminary study phase, summarizes the findings and conclusions, and provides recommendations for the way forward and scope of work to be undertaken during the feasibility study phase.

The Preliminary Study Phase was divided into two stages:

- Desktop Study; and
- Preliminary Study.

The aim of the desktop study was, through a process of desktop review, analyses of existing reports and data, and screening, to determine the three best development options from the pre-identified 19 development options (from the previous investigation). This process is described in Section 2 of this report.

The aim of the preliminary study was to gather more information with regard to the three selected development options as well as to involve the Eastern Cape Provincial Government and key stakeholders in the process of selecting the single best development option to be taken forward into Phase 2 of the study.

The main activities undertaken during of the second stage of Phase 1 were as follows:

- Stakeholder involvement;
- Environmental screening;
- Water requirements (including domestic water supply, irrigation and hydropower);
- Hydrological investigations;
- Geotechnical investigations;
- Topographical survey investigations, and
- Selection process.

1.1.3 Phase 2 – Feasibility Study

The preliminary study recommended a preferred dam site and scheme development to be taken forward to Feasibility Study level.

The key activities undertaken during the Feasibility Study were as follows:

- Detailed hydrology (over and above that undertaken during the Preliminary Study);
- Reserve determination;
- Water requirements investigation (including agricultural and domestic water supply investigations);
- Topographical survey (over and above that undertaken during the Preliminary Study);
- Geotechnical investigation (more detailed investigations than during the Preliminary Study);
- Dam design;
- Land matters;
- Public participation;
- Regional economics; and
- Legal, institutional and financial arrangements.

An Environmental Impact Assessment was undertaken in a separate study that ran in parallel to this one;

1.1.4 Additional Detailed Investigations for Lalini Dam and Hydropower Scheme

Further detailed investigations were undertaken for a second dam on the Tsitsa at Lalini (just above the Tsitsa Falls) which would be operated conjunctively with the Ntabelanga Dam to generate significant hydropower for supply into the national grid.

The Feasibility Design of the Lalini Dam and hydropower scheme is described in Report No. P WMA 12/T30/00/5212/19.

1.2 Purpose of this Report

This report describes the process taken to develop an optimum selection of dam location, dam type, and spillway type, and the feasibility level design of the selected type of dam, at the Ntabelanga site that was selected in Phase 1, as described in the Preliminary Study Report No. P WMA 12/T30/00/5212/3.

It was confirmed and agreed that the sizing and modus operandi of the Ntabelanga Dam and its associated works would take into account its multi-purpose role, namely:

- To supply potable water to an estimated current population of 502 822 people (rising to some 726 616 people in 2050), and other potable water consumers in the region;
- To supply raw water for irrigation of some 2 868 ha of high potential agricultural land;
- To generate hydropower locally at the dam wall to reduce the cost of energy consumption when pumping water;
- To provide sufficient flow of water downstream of the Ntabelanga Dam to meet environmental water requirements for an ecological Class C; and
- To provide additional balancing storage volume and consistent downstream flow releases to enable a second dam at Lalini (just above the Tsitsa Falls) to generate significant hydropower for supply into the national grid.

2. NTABELANGA DAM FINAL SITING AND SIZING

2.1 Location

A review of the location of the Ntabelanga Dam wall identified in previous studies, and confirmed by Phase 1 of this study, was undertaken using both topographical mapping as well as field reconnaissance.

As shown in Figure 1-1, the proposed Ntabelanga Dam is located approximately 55 km north of Mthatha on the Tsitsa River, at co-ordinates 31° 7'1.40"S, 28°40'20.45"E.

The 1 971 km^2 of catchment area contributing to the Ntabelanga Dam in the tertiary catchment T35 is somewhat developed, with approximately 10% of the catchment area under commercial forestry.

Table 2-1: Catchment Area: Ntabelanga Dam Site

Quaternary Catchment	Catchment Area (km^2)
T35A	476.5
T35B	396.8
T35C	307.0
T35D	348.9
T35E to Ntabelanga Dam Wall	441.9
TOTAL	**1 971.1**

Note: The total area of quaternary catchment T35E is 493.5 km^2, of which 51.6 km^2 lies below the dam wall.

It was concluded that the Ntabelanga site provided a very favourable river valley shape (which affects dam wall length), geology/founding conditions, close proximity to construction materials, and the depth verses volume characteristics of the impoundment.

Both upstream and downstream of the primary dam site, the valley widens and flattens, and the next suitable dam site location downstream was actually one of the others previously eliminated in the Phase 1 screening process, called the Malepelepe dam site.

Therefore the more detailed Ntabelanga Dam wall siting investigations for the Feasibility Study have focussed on the narrowest part of the Tsitsa River valley just before the valley widens.

Phase 1 drilling was undertaken on an alignment with the optimum geomorphology selected on site by the study team's Engineering Geologist. The two core holes drilled in Phase 1 indicated very good foundation conditions on competent dolerite, with a competent dolerite right abutment, and a combination of dolerite overlain by sandstone (at higher elevation) on the left abutment.

The general locations of the two alternative embankment wall alignments which were considered are indicated in Figure 2-2.

This also shows the locations of additional drilling and other geotechnical investigations that were undertaken in Phase 2 of the study.

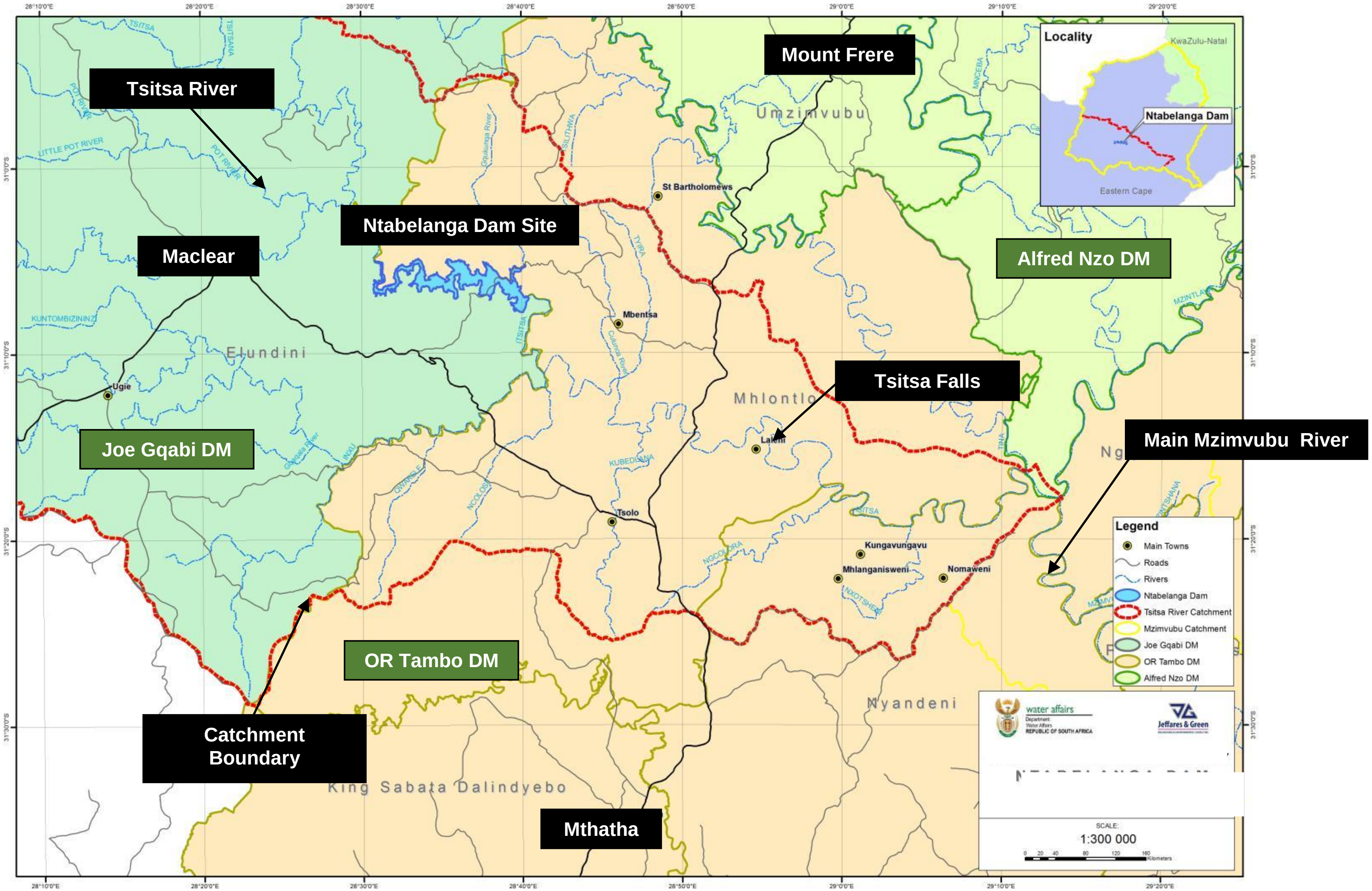
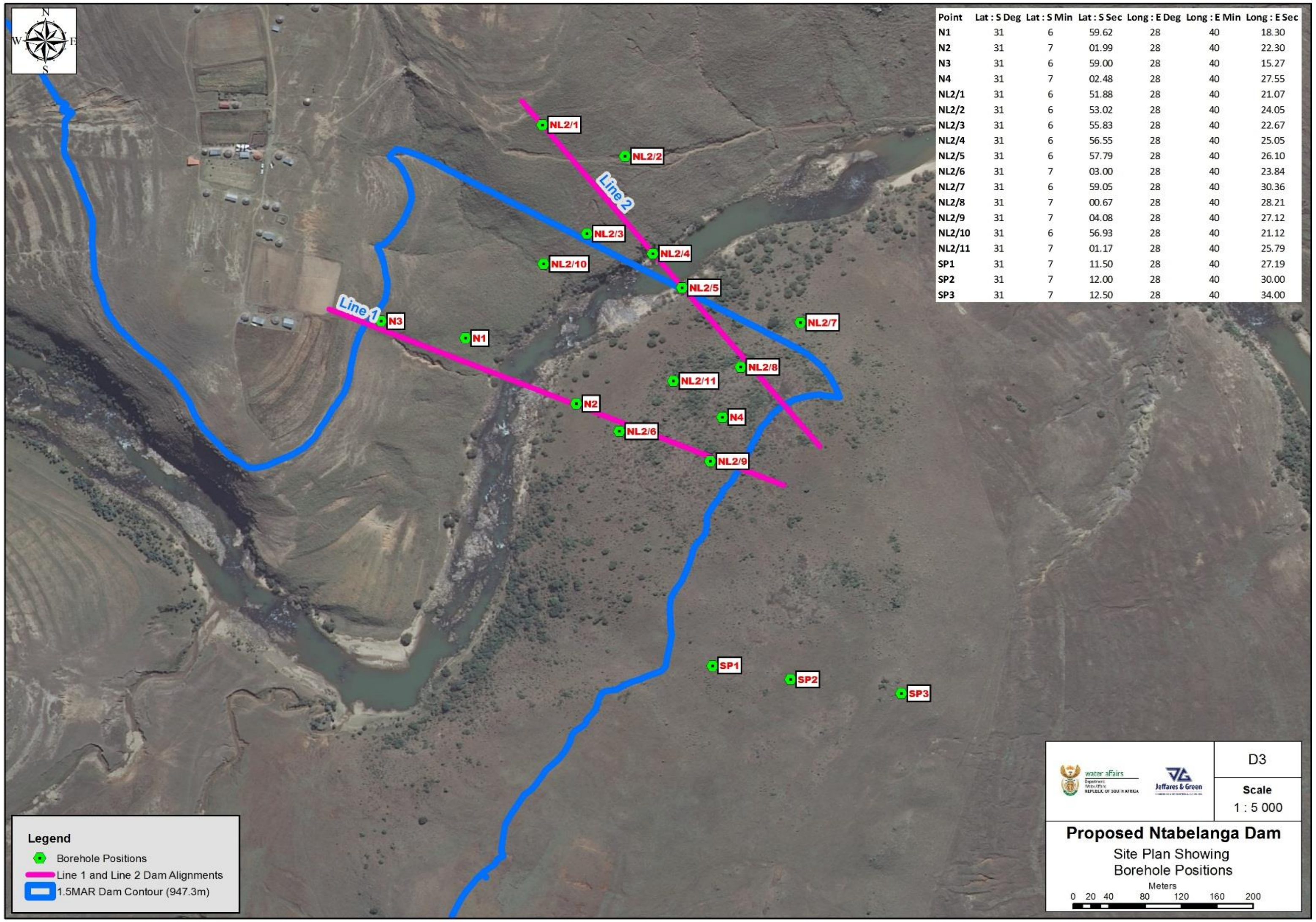


Figure 2-1: Location of Ntabelanga Dam



Point	Lat : S Deg	Lat : S Min	Lat : S Sec	Long : E Deg	Long : E Min	Long : E Sec
N1	31	6	59.62	28	40	18.30
N2	31	7	01.99	28	40	22.30
N3	31	6	59.00	28	40	15.27
N4	31	7	02.48	28	40	27.55
NL2/1	31	6	51.88	28	40	21.07
NL2/2	31	6	53.02	28	40	24.05
NL2/3	31	6	55.83	28	40	22.67
NL2/4	31	6	56.55	28	40	25.05
NL2/5	31	6	57.79	28	40	26.10
NL2/6	31	7	03.00	28	40	23.84
NL2/7	31	6	59.05	28	40	30.36
NL2/8	31	7	00.67	28	40	28.21
NL2/9	31	7	04.08	28	40	27.12
NL2/10	31	6	56.93	28	40	21.12
NL2/11	31	7	01.17	28	40	25.79
SP1	31	7	11.50	28	40	27.19
SP2	31	7	12.00	28	40	30.00
SP3	31	7	12.50	28	40	34.00

Figure 2-2: Alternative Dam Wall Alignments

At Line 2, some 200 m downstream of Line 1, the centreline would coincide with the “nose” of the right abutment hill whence the valley immediately widens into a floodplain. This would allow a shorter side channel spillway discharge chute, would provide slightly easier access and more working space for construction, and would mean that the infrastructure immediately downstream (possibly hydropower house, pumping station, administration buildings, water treatment works) would be located closer to the dam wall but away from any potential backwater flooding effects below the dam.

In Phase 2, further site investigations (core drilling) and materials trial pitting and sampling were carried out on these two alignments and potential spillway locations in Phase 2, as described in detail in Geotechnical Investigations Report No. P WMA 12/T30/00/5212/10.

Line 1 was eventually selected by the study team Approved Professional Person as the preferred alignment, after further consideration of the results of the geotechnical investigations, and other factors such as avoiding lineaments on the valley flanks.

Table 2-2 shows the co-ordinates of the borehole sites.

Table 2-2: Co-ordinates of Site Investigation Boreholes

	Southings			Eastings		
Point	**Deg**	**Min**	**Sec**	**Deg**	**Min**	**Sec**
N1	31	6	59.62	28	40	18.30
N2	31	7	1.99	28	40	22.30
N3	31	6	59.00	28	40	15.27
N4	31	7	2.48	28	40	27.55
NL2/1	31	6	51.88	28	40	21.07
NL2/2	31	6	53.02	28	40	24.05
NL2/3	31	6	55.83	28	40	22.67
NL2/4	31	6	56.55	28	40	25.05
NL2/5	31	6	57.79	28	40	26.10
NL2/6	31	7	3.00	28	40	23.84
NL2/7	31	6	59.05	28	40	30.36
NL2/8	31	7	0.67	28	40	28.21
NL2/9	31	7	4.08	28	40	27.12
NL2/10	31	6	56.93	28	40	21.12
NL2/11	31	7	1.17	28	40	25.79
SP1	31	7	11.50	28	40	27.19
SP2	31	7	12.00	28	40	30.00
SP3	31	7	12.50	28	40	34.00

Note: “N” prefix are boreholes for dam and “S” prefix are for flank and spillway geology.

2.2 Dam Size

The dam wall height and its impoundment volume have been based on the assumption that Ntabelanga Dam is to be used conjunctively with a potential second dam sited downstream at Lalini, which itself would be used to produce hydropower.

In Phase 1, the optimal capacity for the Ntabelanga Dam was determined to be some 490 million m^3 of water. DWS requested the study team to investigate whether the conjunctive use scheme could economically produce more power by building a larger Ntabelanga Dam, and for this reason larger dam capacities of up to 650 million m^3 have been considered.

Yield modelling and economic analyses of the 490 million m^3 to 650 million m^3 Ntabelanga Dam options undertaken for this Feasibility Study showed that the return on the significantly increased capital cost investment of the larger dam sizes over the optimal dam, in terms of both incremental yield and additional power produced, would not prove to be worthwhile, and the larger Ntabelanga Dam would also not generate any more job creation or economic development opportunities in the region than the optimally sized dam(see Report nos. P WMA 12/T30/00/5212/5 and P WMA 12/T30/00/5212/15).

In addition, a 650 million m^3 dam would have a 7 m higher Full Supply Level (FSL) than the “optimal dam”, and would inundate 47 km^2 of land compared with 39 km^2 for the 490 million m^3 dam (an increase of about 21% in land lost, most of which could be arable land and including a number of households).

From the above, and from observations during a recent field visit, the 650 million m^3 sized dam would have a significantly higher impact on the communities within the inundated area, including the availability of suitable areas for their resettlement, making the provision of alternative access roads much more difficult to solve, and the significant cost implications.

Following the undertaking of an extension of the Phase 1 topographical survey covering extended areas around the Ntabelanga Dam site and impoundment areas, it was noted that increasing the full supply level above that required for the 490 million m^3 dam solution would also require a saddle dam in addition to the main dam.

Therefore, whilst the geotechnical investigations and additional surveys were planned to cover the eventuality of a 650 million m^3 dam solution, it was considered unlikely that a dam larger than 490 million m^3 would be an optimum solution.

With regard to dam type comparisons, it was therefore agreed that these be based on the 490 million m^3 option, which has been described as the “maximum” dam option.

This capacity dam has the following parameters:

Full Supply Level	947.3 m.a.s.l.
Non Overspill Crest Level	953.9 m.a.s.l.
Minimum bed level in river at dam	885.0 m.a.s.l.
Maximum dam wall height to NOC	67.7 m
Wall crest length	440 m

In all cases a dam wall crest width of 6 m was used for comparison purposes, but this should be revisited in the detailed design stage.

3. DAM TYPE ANALYSES

3.1 Dam Options Investigated

It was deemed important to consider the range of possible dam type options before committing to the further core drilling to be undertaken in Phase 2. The selected dam type options also determined what other geotechnical investigations (including materials sourcing and geophysics) should be undertaken in parallel with the core drilling.

All previous studies and Phase 1 of this study had considered only earth embankment/clay core (earth fill) options. In this feasibility analysis, the study team considered several other options, as well as various spillway arrangements.

The following dam types were investigated in Phase 2

- Roller compacted concrete (RCC) dam;
- Concrete faced rock fill dam (CFRD);
- Earth core rock fill dam (ECRD);
- Earth fill embankment dam with earth core (EF); and
- Composite central concrete channel spillway/embankment flank options (CCS).

Figure 3-1 overleaf shows the cross-section profile of the valley, together with the FSL and NOCL of the 490 million m^3 Ntabelanga Dam. The figure has equal vertical and horizontal scales.

Key factors used in determining the optimum dam type were as follows:

- Availability of sufficient quantities and quality of construction materials in the vicinity of the dam wall;
- Constructability issues, especially relating to dealing with river flow during construction;
- The ability of DWS to design and construct the dam in-house;
- Spillway location and capacity requirements;
- Operational requirements and outlet works arrangements;
- Environmental impacts; and
- The cost of the works.

In order to assess materials requirements, quantities were calculated for all of the above dam types, based upon typical design criteria (foundation excavation depths, embankment slopes, etc.), which were undertaken for all of the above dam types and their spillway options.

As discussed below, the geology of the area features competent dolerite founding conditions and on the dam flanks.

Given this fact, and the length of the overflow crest that would be required, various side channel spillway options would offer the most favourable spillway configuration and would provide an abundant source of good construction material, and for this reason, the initial focus was to investigate such spillway options in some detail.

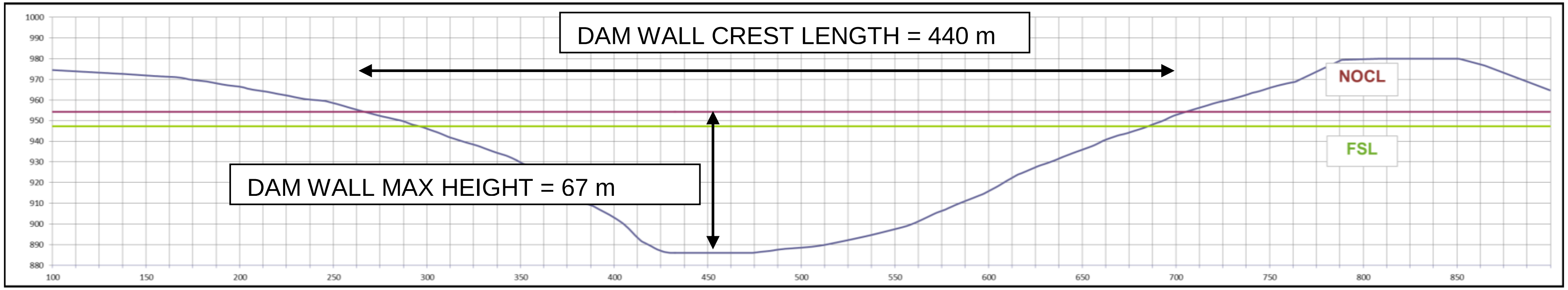


Figure 3-1: Cross-section of valley (looking downstream) at dam wall centreline

3.2 Spillway Options

3.2.1 Spillway Capacity Requirements

The Design Flood has been determined as described in the report included herein as Appendix A. From that analysis it was determined that the 1:200 year return period Design Flood would be of the order of 2 500 m^3/sec, and the Safety Evaluation Flood (SEF) to be 5 530 m^3/sec (both unrouted values at this feasibility stage). The freeboard requirements for the SEF would be the controlling case, and for the purposes of this spillway and dam options comparison, a spillway crest length of 200 m, with a freeboard of 5.5 m was used.

To illustrate the implication of the quantum of these flood figures, and in order to pass this SEF with acceptable overflow depth and flow velocity, a conventional ogee spillway built along the dam wall centreline would need to have a crest length of between 150 m and 200 m. As can be seen from the above cross-section, such a “conventional” spillway would constitute up to 50% of the crest length of the dam, and the spillway structure would span the highest section of the dam even if the spillway is offset as far to the flank as possible, with consequential very high costs. As concrete works are by far the highest cost component of any composite dam, such an arrangement could result in an uneconomic structure for either the earth fill or rock fill embankment options.

In such cases, the typical solution is to build a side channel spillway and discharge chute - either built in reinforced concrete and crossing the end section of the embankment on the flank of the dam, or aligned further outside this line and cut through the hill as a separate rock-lined channel.

Such arrangements can be applied to both rock fill and earth fill embankment dam options. However, the hydraulics of such side channel spillways are quite complex, and can only be properly optimised if laboratory modelling is undertaken, which would only be undertaken at the detailed design stage and not during this feasibility study.

3.2.2 *Spillway Design Approach*

The following three types of uncontrolled spillways were investigated for the Ntabelanga Dam:

- Straight spillway;
- Side channel spillway; and
- Off channel ("cut-through") spillway.

The spillways are compared for a full supply level (FSL) at 947.3 m.a.s.l.

a) *Spillway discharge parameters*

The control structure for all three spillway options will be in the form of an ogee spillway.

The discharge for an ogee spillway is given by the following relationship:

$$Q = C_D \sqrt{2g} L H_D^{1.5}$$

Where:

Q = discharge in m^3/s
C_D = discharge coefficient at the design head (H_D) as illustrated in Figure 3-2 where P is the approach depth to the spillway.
L = crest length in metres
H_D = total energy head on the crest in metres at design flow.
g = gravitational acceleration. (9.81m/s^2)
h_o = approach velocity head component of total energy head

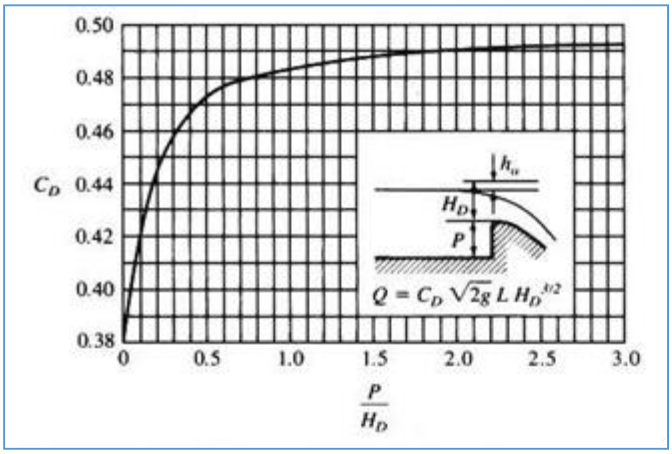


Figure 3-2: Ogee Discharge Co-efficient

As suggested in Figure 3-2, the discharge coefficient (C_D) reaches a maximum of 0.492 when the spillway approach depth (P) is equal to or greater than some 2.5 times the total energy head (H_D).

Figure 3-3 illustrates the effect on the discharge coefficient under flow conditions other than the design flow. In order to size the ogee section of the 200 m long spillway, a flow depth of 5.5 m was selected representing SEF conditions.

With the spillway height (P) at 32.5 metres and with a design flow depth (H) of 5.5 metres for the SEF the total approach depth will be 38 metres the approach velocity will be small and the velocity head contribution to the total energy head on the spillway may be ignored for now and total energy head (H_D) will then be represented by the flow depth.

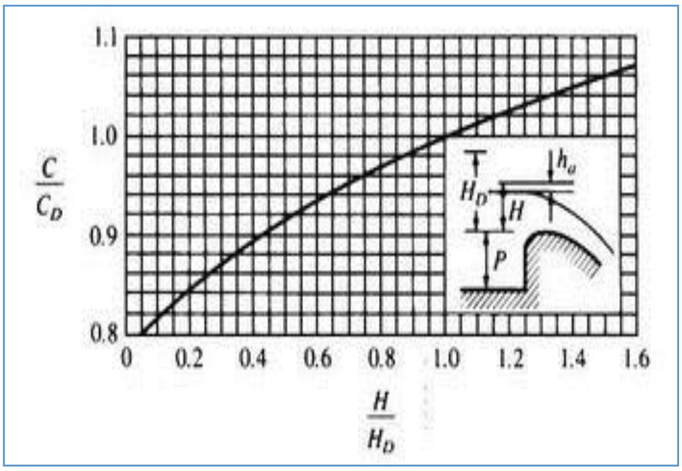


Figure 3-3: Change of Co-efficient Under Deeper Flow Conditions

The ratio of spillway height (P) to the energy head (H_D) will then be 5.9 which represents a discharge coefficient (C_D) of 0.492 (Figure 3-2).

Table 3-1 shows the depth and flow capacity for ogee spillways of various lengths, using the discharge coefficient of 0.492 x $(2g)^{0.5}$ = 2.179.

For a side channel spillway, a preliminary crest length of 200 m was again selected, based upon a reasonable unit discharge value, and taking into account the volume of rock required from this channel excavation that could be used in the embankment works of a rock fill dam.

Applying this adjusted discharge coefficient to the Design Flood based on the relationship between the design head flow and flows other than the RDF presented in Figure 3-3, returns a 3.2 m flow depth over the spillway.

For the SEF, the depth of flow would increase to 5.5 m and if the dam embankment NOC is at this level then this represents a freeboard of 2.3 metres during an RDF event.

This freeboard is considered to be adequate to allow for wind run-up, surges, seiches, etc, but this requirement would need to be revisited again during the detailed design stage.

Adding a 1.2 m high wave wall along the upstream crest of the dam embankment would increase the allowable depth of flow over the spillway crest, which would have the effect of reducing the spillway crest length to 150 m and increasing the freeboard under SEF conditions to 6.6 m. For an in-channel spillway solution (i.e. for a roller compacted concrete (RCC) dam) this would reduce the spillway chute to a narrower width downstream, making the transitional flow back to the river channel via a stilling pond easier to achieve.

It must also be noted that an RCC dam would be more resilient to wave action over-splash and moderate overtopping than a rock fill or earth fill embankment dam, during an SEF event.

For the purposes of feasibility level studies, for a side channel option, (embankment dams) the spillway ogee crest wall would consist of a mass gravity concrete section 200 metres long. For an RCC central spillway option, a spillway crest length of 200 metres has also been considered when comparing dam types. In both cases, spillway crest length and chute geometry would need to be optimised at detailed design stage following laboratory hydraulic modelling and possibly CFD modelling. For example, a labyrinth weir might be an economic solution, but benefits normally reduce under very high flow depth conditions. This can also be investigated during the detailed design modelling process.

b) Spillway side channel and chute design criteria

A side channel spillway would discharge into a channel and chute, and given the geology and topography of this site, there are several options possible, which were investigated in some detail.

Side channel options were sized using conventional open channel hydraulics formulae. Models were also checked using the US Army Corps of Engineers HEC-RAS channel flow modelling software. Several iterations were run to optimize channel dimensions.

The base width of the side channel was sized from 20 m at the upstream end to 50 m at the downstream end. The channel depth required varies from 12.5 m at the upstream end to 16.5 m at the downstream end. The side slopes were 1V:0.5H. The maximum water level at the upstream end of the side channel was limited to 3 m above the FSL to prevent submergence of the ogee crest during the SEF.

Table 3-1: Flow depth and Capacity Table for Ogee Crest Spillway

Coefficient (C)		2.179			$Q = C \times L \times H_o^{1.5}$				
	Note: $C = C_o \times (2 \times 9.81)^{0.5}$		Capacity (m³/s) at Crest Length (m)						
H_0 (m)	Q/m (m³/s/m)	V (m/s)	100	125	150	175	200	225	250
0	0.00	0.00	0	0	0	0	0	0	0
0.2	0.19	0.97	19	24	29	34	39	44	49
0.4	0.55	1.38	55	69	83	96	110	124	138
0.6	1.01	1.69	101	127	152	177	203	228	253
0.8	1.56	1.95	156	195	234	273	312	351	390
1	2.18	2.18	218	272	327	381	436	490	545
1.2	2.86	2.39	286	358	430	501	573	644	716
1.4	3.61	2.58	361	451	541	632	722	812	902
1.6	4.41	2.76	441	551	661	772	882	992	1102
1.8	5.26	2.92	526	658	789	921	1052	1184	1316
2	6.16	3.08	616	770	924	1079	1233	1387	1541
2.2	7.11	3.23	711	889	1067	1244	1422	1600	1778
2.4	8.10	3.38	810	1013	1215	1418	1620	1823	2025
2.6	9.14	3.51	914	1142	1370	1599	1827	2055	2284
2.8	10.21	3.65	1021	1276	1531	1787	2042	2297	2552
3	11.32	3.77	1132	1415	1698	1981	2264	2548	2831
3.2	12.47	3.90	1247	1559	1871	2183	2495	2806	3118
3.4	13.66	4.02	1366	1708	2049	2391	2732	3074	3415
3.6	14.88	4.13	1488	1860	2233	2605	2977	3349	3721
3.8	16.14	4.25	1614	2018	2421	2825	3228	3632	4035
4	17.43	4.36	1743	2179	2615	3051	3486	3922	4358
4.2	18.76	4.47	1876	2344	2813	3282	3751	4220	4689
4.4	20.11	4.57	2011	2514	3017	3519	4022	4525	5028
4.6	21.50	4.67	2150	2687	3225	3762	4300	4837	5374
4.8	22.91	4.77	2291	2864	3437	4010	4583	5156	5729
5	24.36	4.87	2436	3045	3654	4263	4872	5481	6090
5.2	25.84	4.97	2584	3230	3876	4522	5168	5814	6460
5.4	27.34	5.06	2734	3418	4101	4785	5469	6152	6836
5.6	28.88	5.16	2888	3610	4331	5053	5775	6497	7219
5.8	30.44	5.25	3044	3805	4566	5326	6087	6848	7609
6	32.02	5.34	3202	4003	4804	5604	6405	7206	8006
6.2	33.64	5.43	3364	4205	5046	5887	6728	7569	8410
6.4	35.28	5.51	3528	4410	5292	6174	7056	7938	8820
6.6	36.95	5.60	3695	4618	5542	6466	7389	8313	9237
6.8	38.64	5.68	3864	4830	5796	6762	7728	8694	9660
7	40.36	5.77	4036	5044	6053	7062	8071	9080	10089
7.2	42.10	5.85	4210	5262	6315	7367	8419	9472	10524
7.4	43.86	5.93	4386	5483	6580	7676	8773	9869	10966
7.6	45.65	6.01	4565	5707	6848	7989	9131	10272	11413
7.8	47.47	6.09	4747	5933	7120	8307	9494	10680	11867
8	49.31	6.16	4931	6163	7396	8628	9861	11094	12326
8.2	51.17	6.24	5117	6396	7675	8954	10233	11512	12791
8.4	53.05	6.32	5305	6631	7957	9284	10610	11936	13262
8.6	54.95	6.39	5495	6869	8243	9617	10991	12365	13739
8.8	56.88	6.46	5688	7110	8532	9954	11377	12799	14221
9	58.83	6.54	5883	7354	8825	10296	11767	13237	14708

SEF=	5530	m³/sec	RDF=	2500

A side channel spillway typically ends in a deflector or “flip” bucket and plunge pool arrangement, which is a cost effective energy dissipating structure. However, on account of the depth of the discharge channel and the high tail water levels downstream of the dam, the deflector bucket would drown during high flood peaks, and protection measures would be required to prevent erosion during small floods that do not spring clear. It was considered therefore that this would have to be sited at a high elevation above the bed, but would possibly not be the most cost effective energy dissipating structure.

An alternative energy dissipating structure considered was a stilling basin. The invert of the stilling basin would have to be at least 5 m below the existing river bed level to be effective during low flows. The side walls need to be at least above the tail water level that would occur when the downstream flow rate reaches 3 500 m^3/s, which would require a structure 15 m deep.

Whilst these side channel, chute and stilling basin solutions involved very significant hard rock excavation, it was noted that such excavated material was likely to be suitable for use in a rock-fill dam, RCC dam (stilling basin material only), and for concrete aggregate to meet all other structural concrete requirements. This was taken into consideration in the cost estimation process. As is later described herein, the stilling basin is also useful for dissipation of the energy of discharge from the proposed hydropower plant to be located just downstream of the dam wall.

As described above, all of the spillway, channel and chute options, feasibility study level hydraulic analyses were undertaken using both channel flow equations and HEC-RAS modelling. At detailed design stage, the selected solution should be optimised using physical laboratory scale modelling if possible. In summary, for the cases of earth-fill and rock-fill dam types, three optional spillway alignments were considered, as follows:

i. Spillway Option 1 (the “side-channel (right flank)” (SC-R) option) comprises a spillway channel cut into the upper right flank and orientated perpendicular to the dam axis, as indicated on Figure 3-4.

ii. Spillway Option 2 (the “Cut-Through” (CT) Option) proposes an excavation through the hill upstream of the dam as indicated on Figure 3-5.

iii. Spillway Option 3 (the “side-channel (left flank)” (SC-L) option) comprises a spillway channel cut into the upper left flank and orientated perpendicular to the dam axis, as indicated on Figure 3-6.

Given that i, ii and iii above require significant excavation, the approach taken was to select a dam configuration that would incorporate as much of the excavated material as possible into the works, and thus minimize the amount of material required to be imported from distance, or disposed of to spoil. In each case the figures show the depth of flow profile through the spillway and chute sections, indicating where subcritical and supercritical flow occurs.

The downstream water level and flood line is also shown under SEF conditions, and this has been considered when undertaking the analyses of both stilling basin, and to avoid downstream ancillary works being affects by floods. This flood line was also determined using the HEC-RAS modelling software, which is described in more detail in the Water Resources Report No. P WMA 12/T30/00/5212/5.

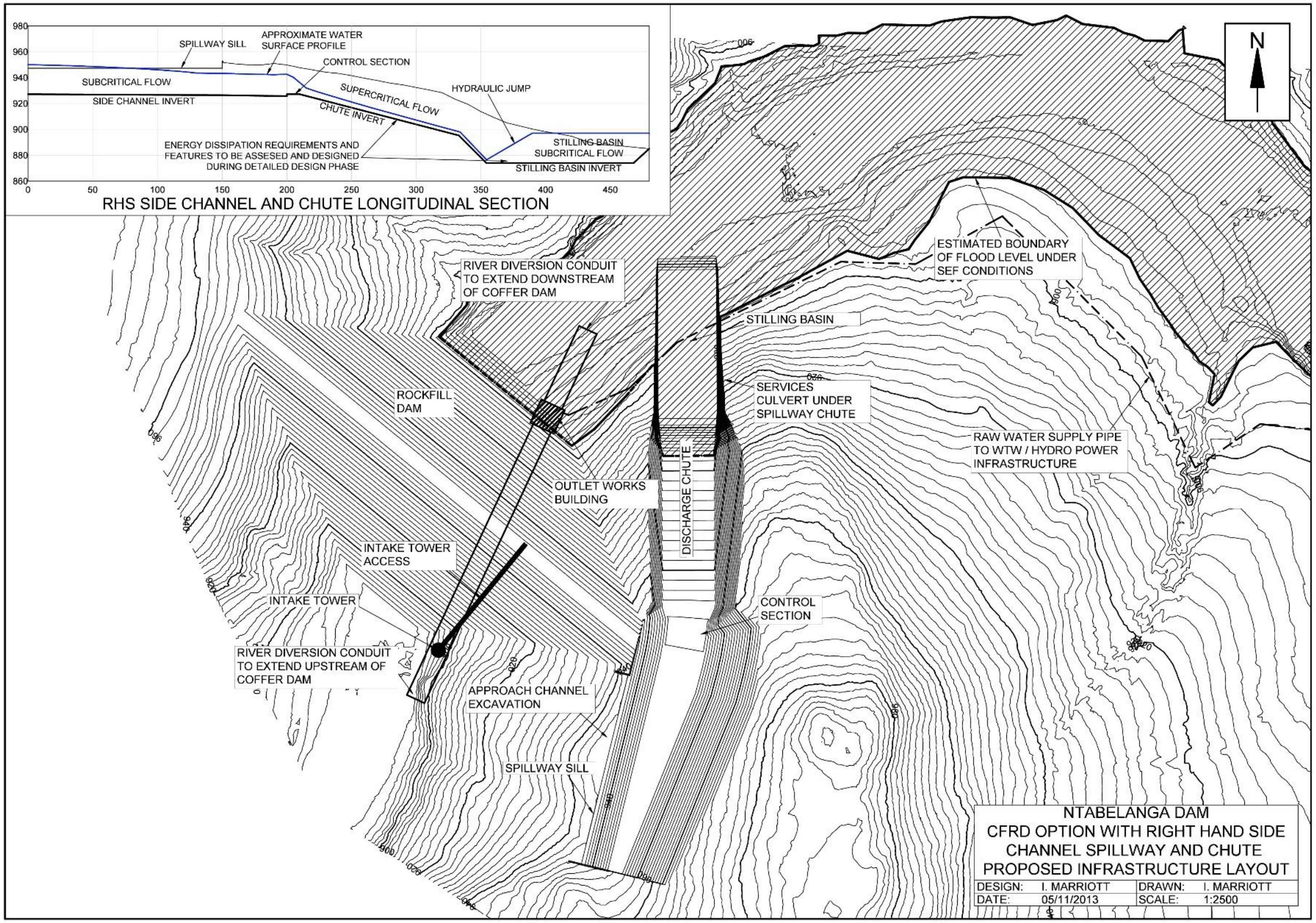


Figure 3-4: Side Channel Spillway Option Arrangement on Right Flank (Option 1)

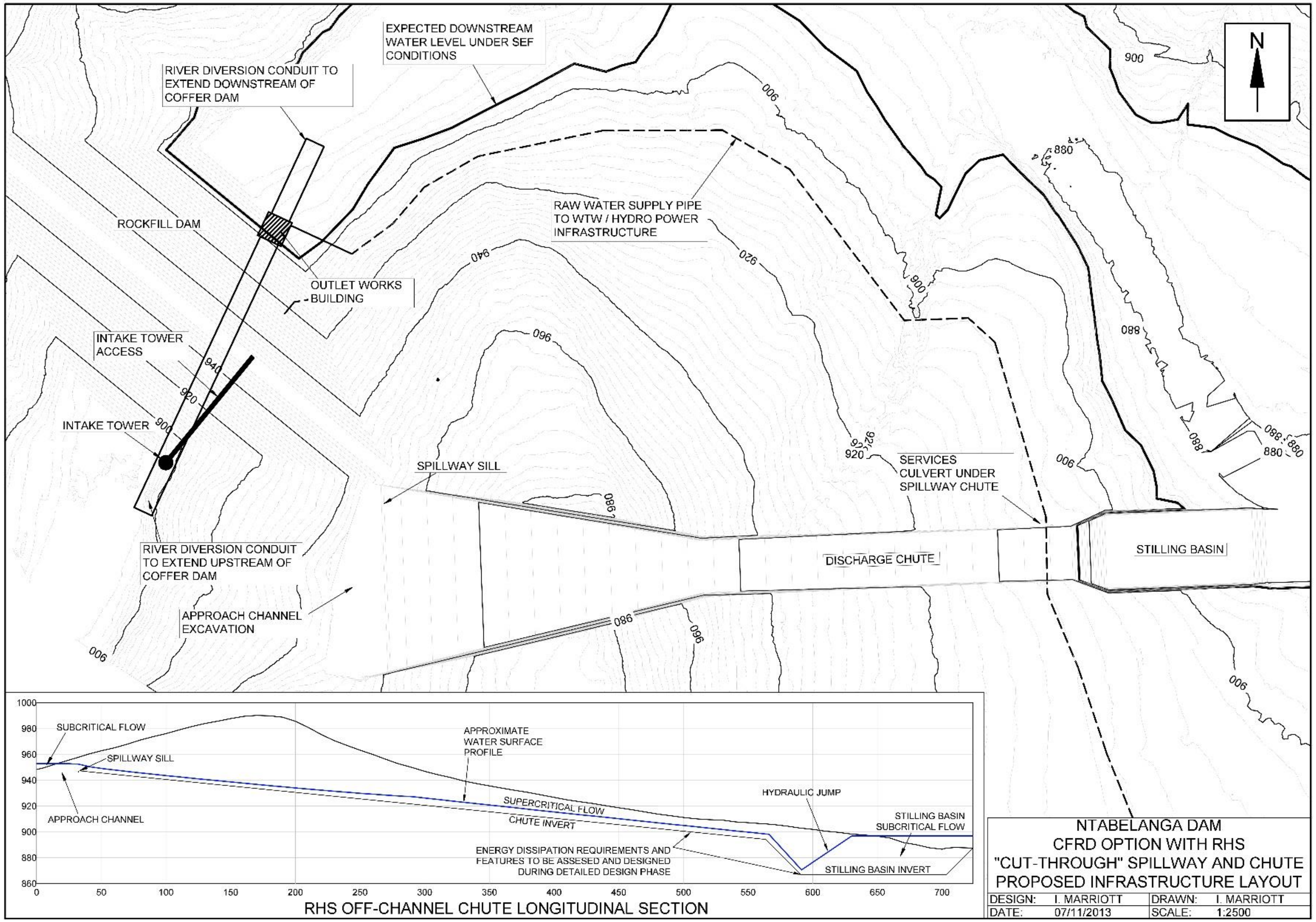


Figure 3-5: Off-Channel "Cut-Through" Spillway Option through Hill on Right Flank (Option 2)

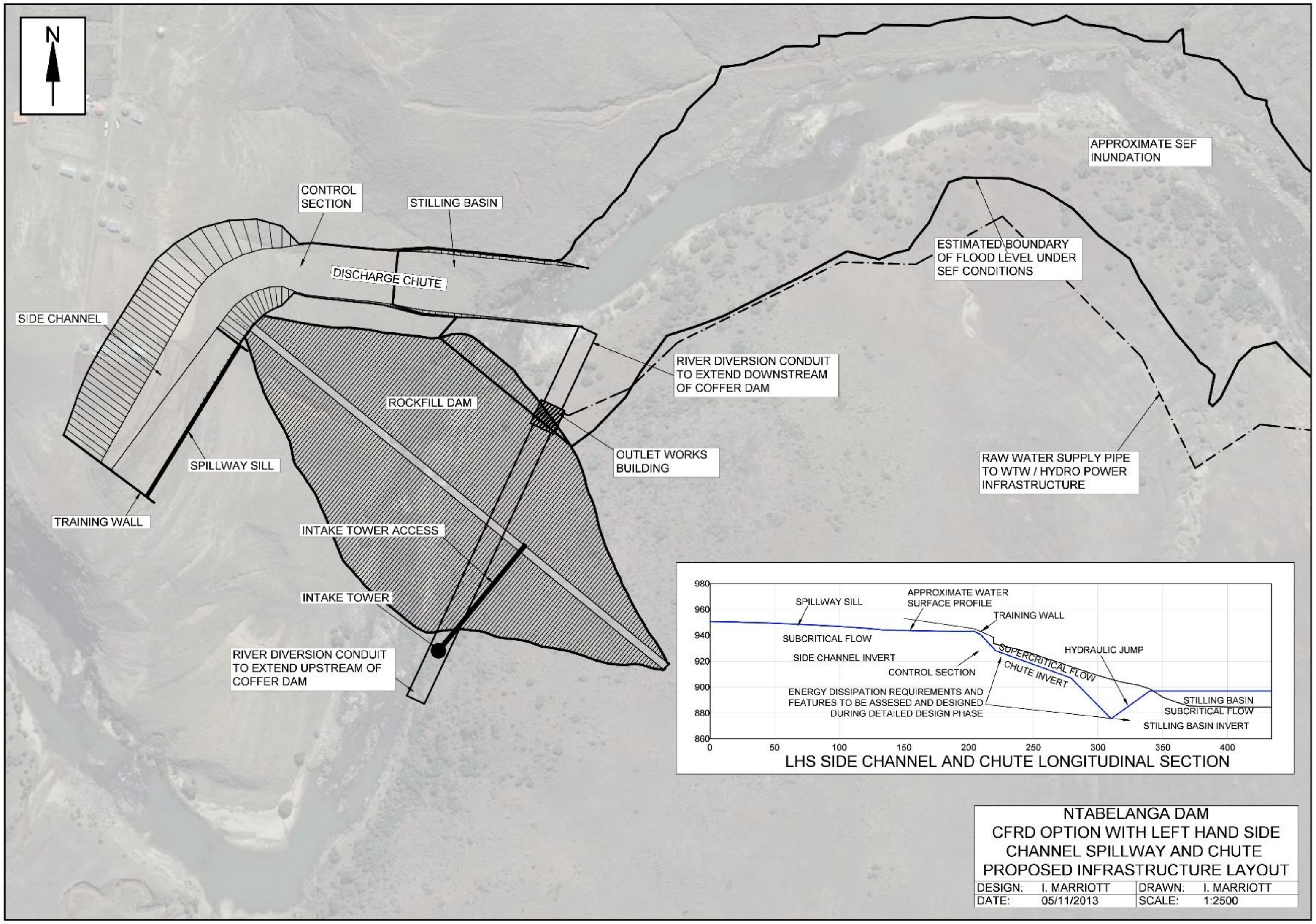


Figure 3-6: Side Channel Spillway Option Arrangement on Left Flank (Option 3)

c) RCC Dam Spillway Option

RCC construction lends itself very well to the situation where the spillway length is a significant proportion of the total crest length, and this option was therefore also investigated. Due to the slope of the downstream face of 1:0.7, the spillway can directly be incorporated into the dam body. This is a major advantage for dams which have to accommodate large floods and which are in need of proportionately large spillways.

At this feasibility study level, an ogee crest length of between 150 m to 200 m was considered which limits the SEF unit discharge rate to some 27 to 37 m^3/sec/m which is considered an acceptable figure, given the infrequency that such high flows would occur. For comparison with other dam type options, a 200 m spillway crest length was used.

A stepped spillway chute with 1.2 m high x 0.84 m wide steps and a slope of 1 to 0.7 was used to determine the costing of an RCC solution. Allowance was made for a significantly sized stilling basin with a gauged outlet weir.

As discussed above, if an RCC dam type is to be adopted for the implementation stage, it is recommended that physical laboratory hydraulic modelling be undertaken to optimise the crest shape, spillway, chute, energy dissipation, and stilling basin detailed design.

d) Composite Central Spillway Dam with Bathtub Spillway

One final option investigated was a composite dam with a bathtub spillway central section, and earth embankment flank walls (as used at DWS's Inyaka Dam), using similar methodologies described above.

3.3 Other Considerations

Other issues that were considered when deciding on dam type was the construction sequencing and the need to deal with wet season flood conditions during construction.

An earth fill or rock fill embankment solution normally requires extensive river diversion works, and could also require a longer construction period to enable the construction of certain sections within successive dry seasons. The risk of requiring an extended time for construction is higher than for an RCC solution.

RCC works are more resilient to such flooding events if they occur unexpectedly during construction, and can be designed to convey such floods without needing special diversion works to be constructed.

These considerations were taken into account when determining the geotechnical and materials investigations. The next section of this report describes the findings of these investigations.

3.4 Dam Construction Materials Requirements

For each dam wall type described above, cross-sections where prepared, based on a full supply level (FSL) for the proposed dam capacity of 490 million m^3, a spillway crest length of 200 m, plus a freeboard allowance of 5.5 metres[1], to determine the non-overspill crest level (NOCL).

[1] *These figures were used for the comparative analysis but were revised to 150 m crest length and 6.6 m freeboard under SEF conditions on the recommended feasibility design solution described in Section 5 herein.*

This freeboard allowance was based on the un-routed Safety Evaluation Flood of 5 530 m^3/sec and a spillway crest length of 200 metres. Once again, these factors will be revisited in the detailed design stage, but are considered suitable for feasibility study purposes.

The typical dam sections and arrangements shown in Figures 3-7 to 3-11 were used to calculate the quantities of the various construction materials. These were based on previously designed and constructed dams of similar materials, capacities and types that were being investigated in this study.

Cross-sectional "slices" for each dam wall were generated at regular intervals along the dam wall axis to calculate the quantities. The quantities for the outlet works, spillways and temporary construction works were also determined using standard measurement methods.

As a guide to the site investigations, approximate volumes of the various potentially locally sourced materials for the alternative dam options were determined as listed in Table 3-2.

Table 3-2: Estimated Material Volumes for Alternative Dam Types

Dam Type	Crushed Rock/Rock fill	Shell (General Fill)	Core	Sand
Concrete-faced Rock fill (CFRD)	1 300 000 m^3	n/a	n/a	100 000 m^3
Earth Core Rock fill (ECRD)	1 100 000 m^3	n/a	260 000 m^3	100 000 m^3
Earth Core Earth fill Embankment (EF)	65 000 m^3	2 100 000 m^3	500 000 m^3	25 000 m^3
Roller Compacted Concrete (RCC)	500 000m^3	n/a	n/a	200 000 m^3
Composite Central Bathtub Spillway (CCS)	1 000 000 m^3	20 000 m^3	200 000 m^3	150 000 m^3

Concrete aggregate

Concrete aggregate, rock fill

Concrete aggregate, rock fill, filters

Concrete aggregate, filters

Concrete aggregate, rip-rap, filters

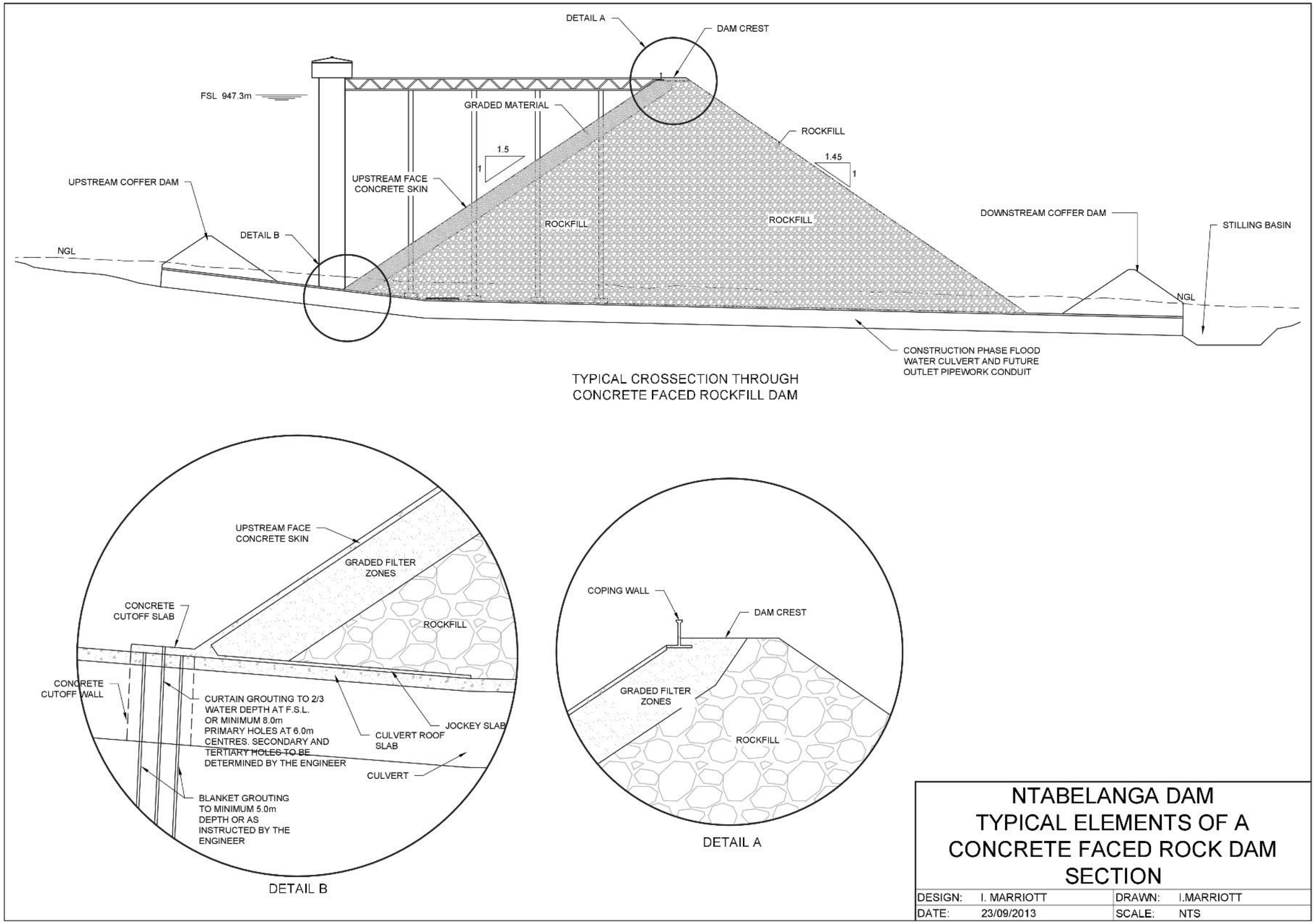


Figure 3-7: Typical Sections and Details for CFRD Type Dam

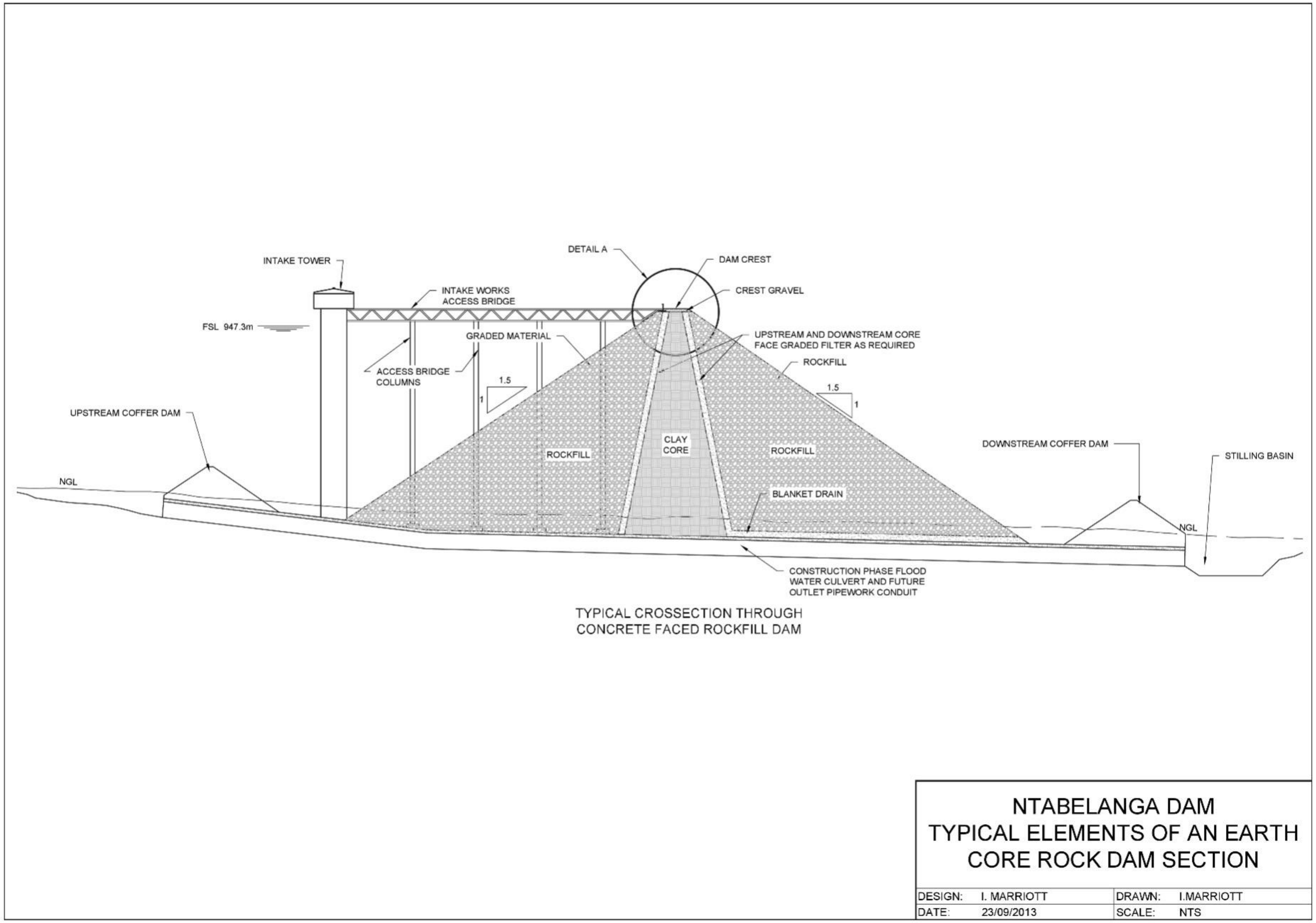


Figure 3-8: Typical Sections and Details for ECRD Type Dam

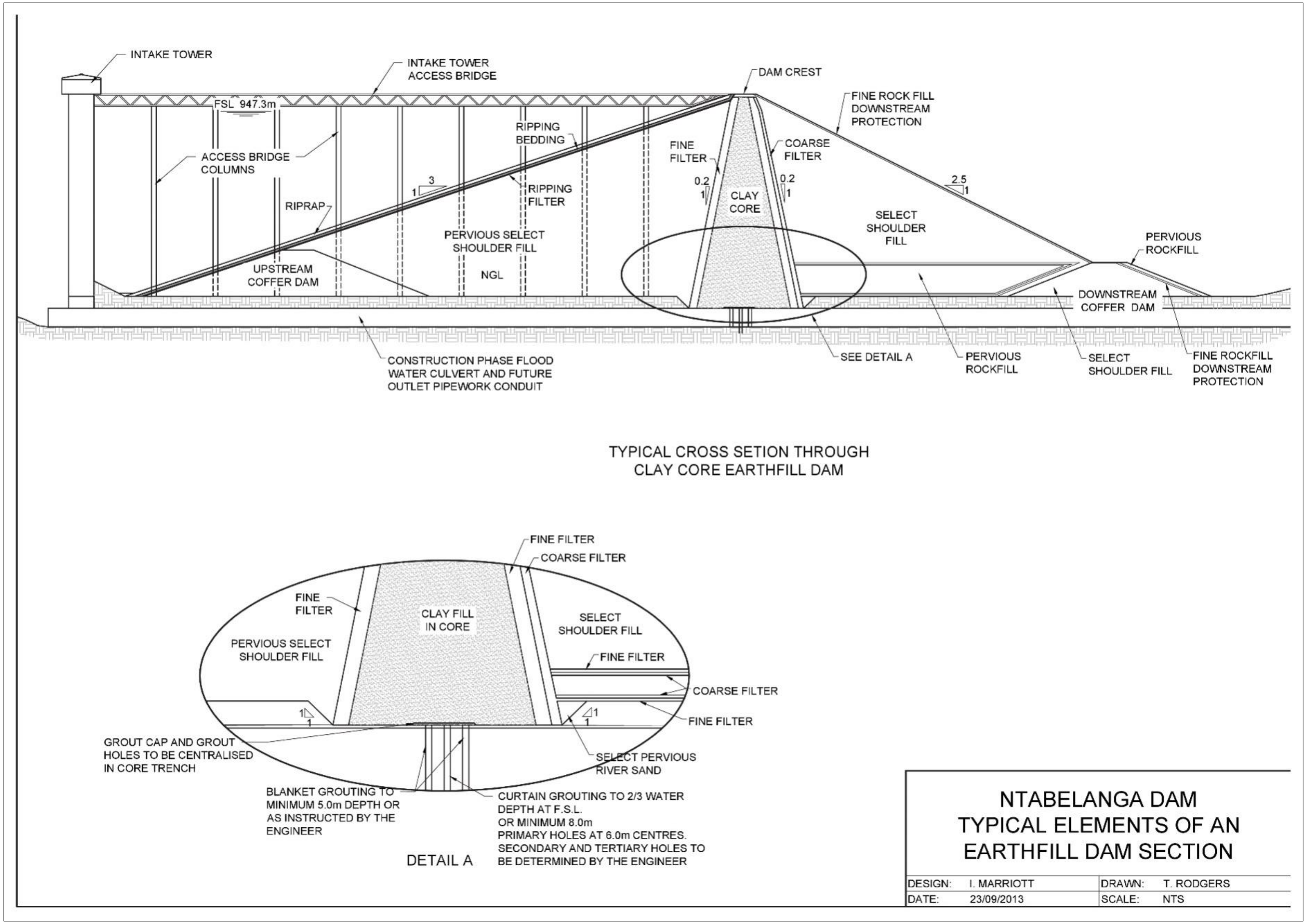


Figure 3-9: Typical Sections and Details for EC Earth fill Type Dam

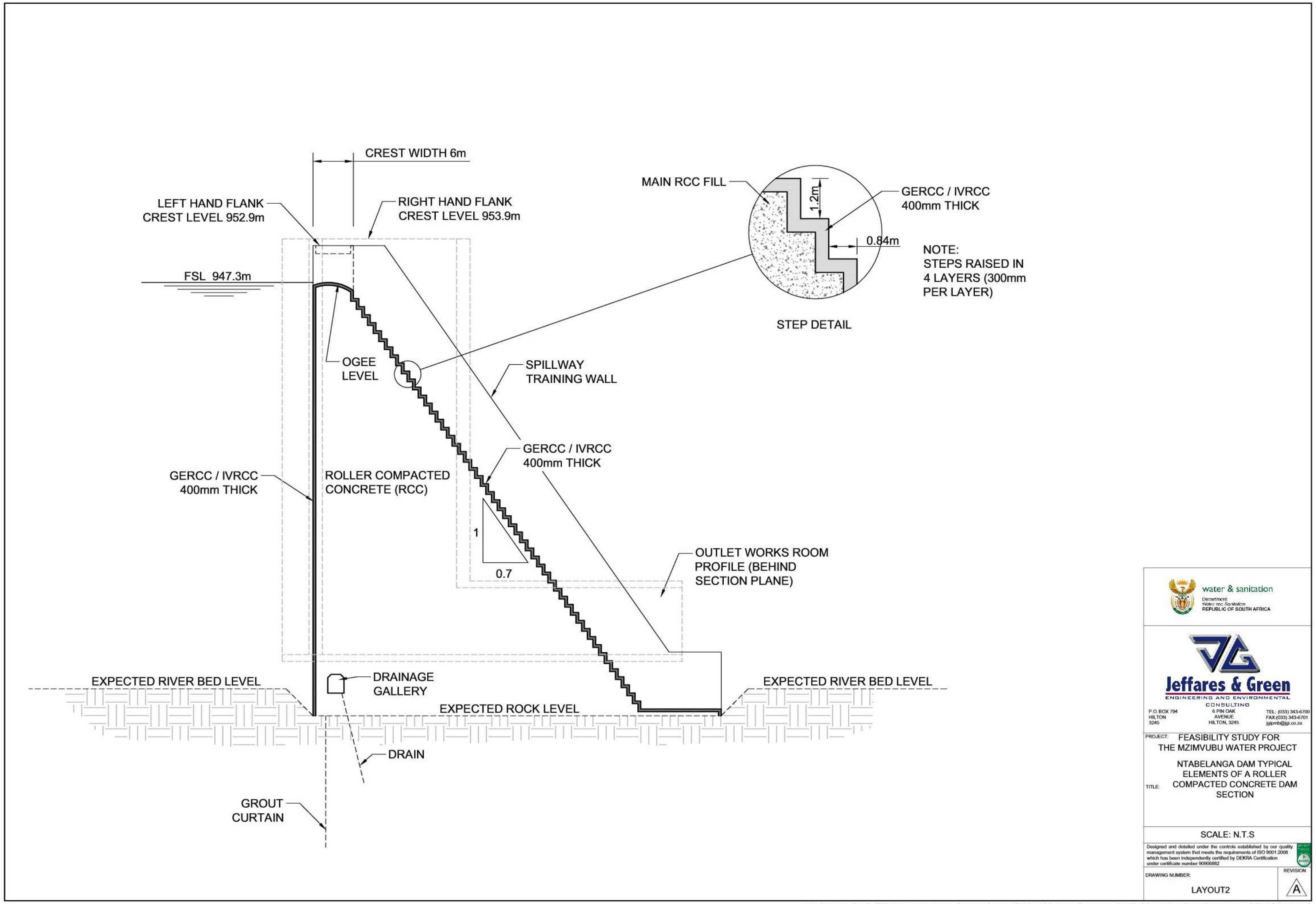


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Figure 3-10: Typical Sections and Details for RCC Type Dam

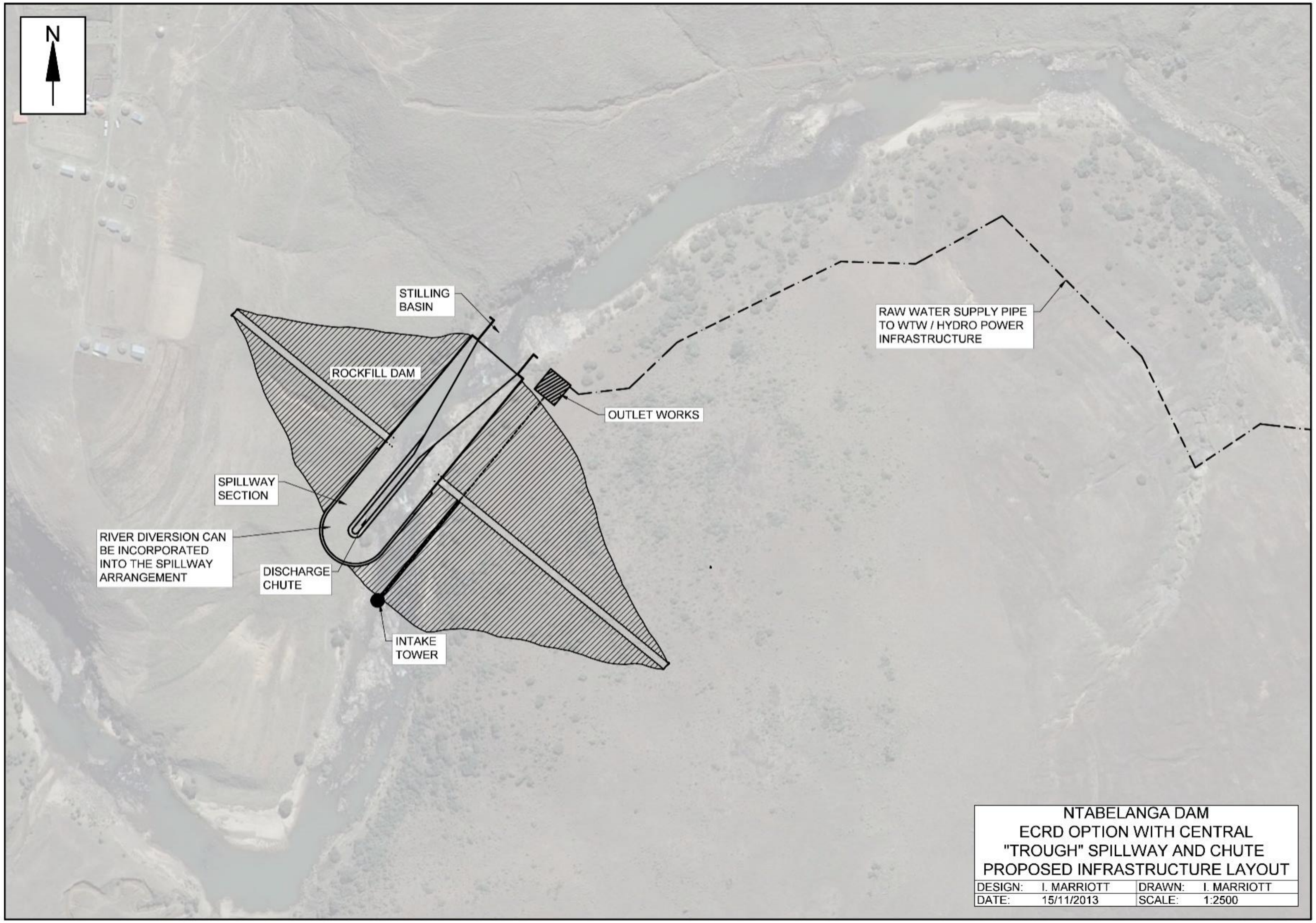


Figure 3-11: General Arrangement of a Composite Dam with Central (Trough) Spillway (CCS)

3.5 Construction Materials and Foundation Investigations

As reported in the Geotechnical Investigations Report Number P WMA 12/T30/00/5212/10, various site investigations have been undertaken, including core drilling, trial pit excavation, laboratory testing of samples, and seismic refraction geophysics.

This has provided adequate information on founding conditions, construction materials quantities and quality, and key design parameters.

Figures 3-12 and 3-13 below show the interpretation of the founding conditions as identified through the core drilling undertaken on the alignments shown in Figure 2-3.

3.5.1 Quarry for the Production of Concrete Aggregate, Rock-fill, Rip-rap, and Coarse Filters

Competent, hard dolerite rock underlies the middle to upper right flank, either near-surface or as an outcrop. The positions of boreholes drilled for the evaluation of dam foundations and spillway excavations are indicated in Appendix B. The depths to competent dolerite, as encountered in the boreholes drilled on the middle to upper right flank are summarised in Table 3-2.

Table 3-3: Middle and Upper Flank Boreholes

Borehole Number	Depth to Competent Dolerite	Comments
N4	2.43 m	
NL2/6	0.98 m	
NL2/7	11.23 m	Spillway Option 1. Drilled on side of hill
NL2/8	0.01 m	
NL2/9	0.66 m	
NL2/11	0.75 m	
SP1	0.41 m	Spillway Option 2
SP2	1.0 m	Spillway Option 2
SP3	8.5 m	Spillway Option 2. Drilled on side of hill

Samples of core material were retrieved from the core boxes and submitted for petrographic analysis to evaluate rock mineralogy, texture, degree of alteration and identification of alteration products, as well as unconfined compressive strength tests to determine intact rock strength. These have demonstrated that this material has low alteration, would provide very good foundations, and would be very suitable for both rock fill and concrete aggregate purposes.

The reserves of potentially good quality dolerite in the hill to the east and south east of the dam, of which the right flank is a part, are very extensive and are far in excess of the required quantities for any of the above listed dam alternatives.

Drilling indicates that a quarry located on the right flank upstream of the dam and within the basin would yield adequate rock aggregate for both dam and concrete structures construction purposes provided that the spillway configuration is designed with this in mind.

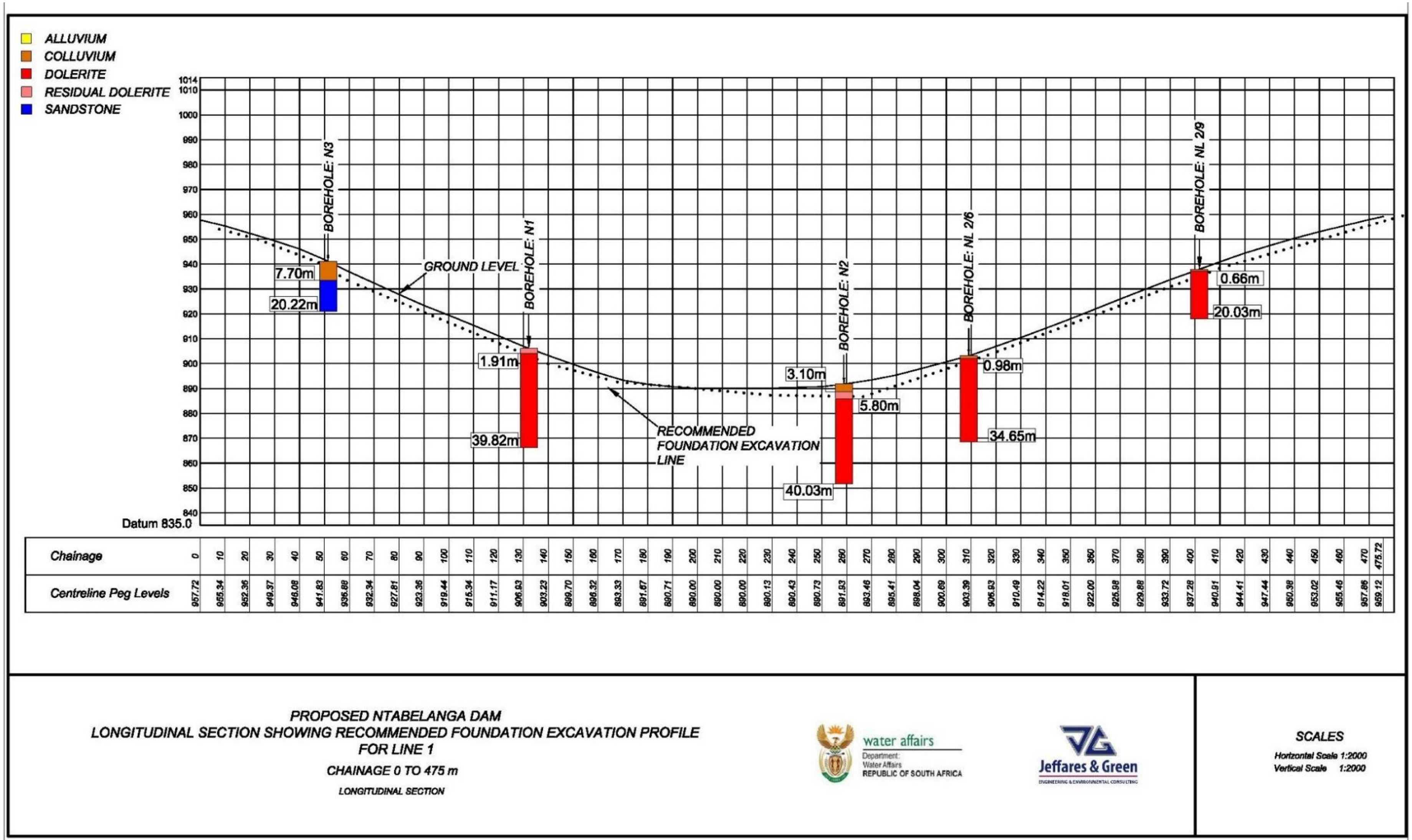
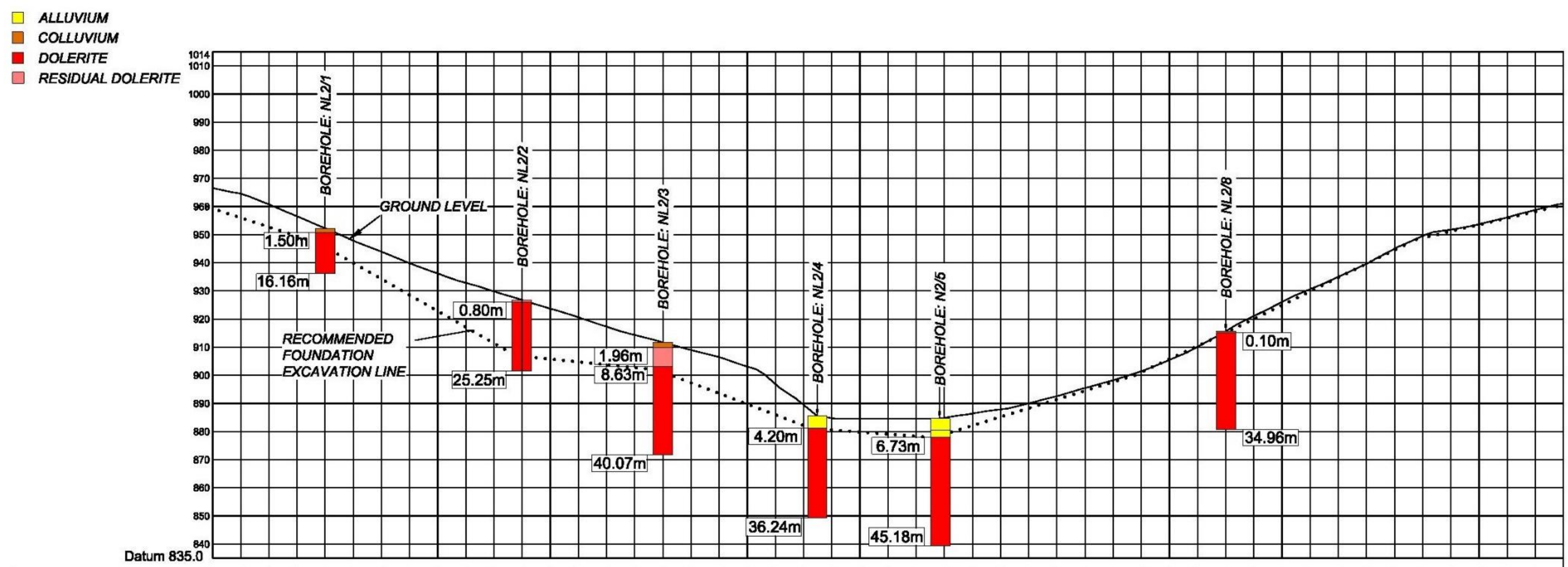


Figure 3-12: Longitudinal Section - Alignment 1 – Showing Interpretation of Core Drilling Logs



Chainage	0	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160
Centreline Peg Levels	966.57	964.53	960.75	956.52	952.27	947.88	943.97	939.85	936.23	932.85	929.79	926.74	923.69	920.63	917.27	914.33	911.70

Chainage	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320
Centreline Peg Levels	908.96	906.46	903.10	896.87	889.49	884.76	884.53	884.53	884.51	884.88	886.40	887.88	890.01	892.56	895.49	898.26

Chainage	330	340	350	360	370	380	390	400	410	420	430	440	450	460	470	480
Centreline Peg Levels	901.42	905.47	910.16	915.69	921.09	926.09	930.63	935.01	939.62	945.00	949.53	951.81	953.69	956.12	958.75	961.02

PROPOSED NTABELANGA DAM
LONGITUDINAL SECTION SHOWING RECOMMENDED FOUNDATION EXCAVATION PROFILE FOR LINE 2
CHAINAGE 0 TO 480 m
LONGITUDINAL SECTION

water affairs
Department: Water Affairs
REPUBLIC OF SOUTH AFRICA

Jeffares & Green

SCALES
Horizontal Scale 1:2000
Vertical Scale 1:2000

Figure 3-13: Longitudinal Section - Alignment 2 – Showing Interpretation of Core Drilling Logs

3.5.2 *Sand for Concrete Aggregate and Filters*

Sand deposits along a section of the Tsitsa River upstream of the dam site were sampled, as indicated by the yellow hatching on the drawings in Appendix B. The Tsitsa River in the project area generally flows in a relatively incised channel with sand deposits confined to the river channel. Therefore these deposits are relatively narrow and would require selective seasonal exploitation during the dry season.

Estimated reserves within the areas investigated were approximately 130 000 m³, but additional reserves that could be sourced by exploiting other nearby sections of the Tsitsa River within the impoundment basin would be far in excess of this and would meet the required volume of 200 000 m³ of sand required for the RCC and CCS dam alternatives.

3.5.3 *Clay Core Material*

Reddish brown, clayey hill-wash deposits associated with dolerite occur in relative abundance throughout the project area. The investigation targeted two areas within the impoundment basin within close haulage distance of the dam, as shown in Appendix B. The combined estimated volume of core material from the two areas is approximately 220 000 m³. Test results indicate that this material would be suitable for core material.

Other areas identified during the reconnaissance are expected to more than triple the volume proved above. In addition, small amounts of core quality material could be procured from the dam foundation excavations on the lower flanks, if an embankment wall solution were to be adopted.

3.5.4 *Embankment Shell Material*

Two areas were investigated within the impoundment basin, cross-hatched orange on the figures in Appendix B.

The shell requirements for the earth fill embankment dam (EF) option are of the order of 2.1 million m³. Sedimentary rocks comprising mainly mudrock with intercalated sandstone are widely distributed within the basin and were tested for suitability as embankment shell. These results proved these materials would be unsuitable for use as pervious fill, and only marginally suitable for use as semi-pervious fill.

Consideration could be given to the investigation of extensive sandstone deposits in the surrounding hills or weathered dolerite, but these occur well outside of the future impoundment basin and the exploitation of the large quantities required would have long haul distances and could have significant environmental impacts. These significant haul distances required have been allowed for in the rates used in the cost estimates, and would significantly increase the cost of this particular dam option.

The paucity of suitable shell material within the basin is viewed as a significant constraint to the construction of an earth fill embankment (EF) alternative.

3.5.5 *Spillway Materials Investigations*

Two alternative spillway alignments on the upper right flank were initially proposed, as illustrated in Figures 3-4 and 3-5 above. A third alternative was proposed on the left flank, also illustrated on Figure 3-6 above.

a) *Spillway Option 1*

Spillway Option 1 proposes a spillway channel cut into the upper right flank and orientated south to north, as indicated on Figure 3-4.

Dolerite outcrops and sub-outcrops are visible along the first approximately 330 m of the spillway axis. The logs of boreholes and trial pits in close proximity to the spillway axis are shown in Table 3-4.

Table 3-4: Spillway Option 1 Boreholes and Trial Pits

Borehole / Trial Pit No.	Description
SP1	Competent dolerite from 0.41 m
NL2/9	Competent dolerite from 0.66 m
N4	Competent dolerite from 2.43 m
NL2/8	Competent dolerite from 0.01 m
NL2/7	Colluvial soil to 2.7 m Residual and completely weathered dolerite to 11.23 m Competent dolerite from 11.23 m
SSP1	Residual dolerite to 1.7 m Highly weathered dolerite to below 3.5 m
SSP2	Weathered mudrock to 2.2 m Weathered dolerite to below 3.9 m
SSP3	Colluvial soil to 4.9 m Residual mudrock to below 5 m
D42*	Colluvial soil to below 2.6 m
*trial pit	Dolerite outcrop in the river

The boreholes show that surface and near surface competent dolerite occurs along the hill crest, with deeper soils and weathering profiles down the hill slope. The transported and residual soils are particularly deep towards the end of the spillway chute before the outfall into the river. This implies a need to concrete-line the spillway chute to provide protection against excessive erosion. Dolerite outcrops are visible in the river.

b) *Spillway Option 2*

Spillway Option 2 proposes an excavation cutting through the hill upstream of the dam as indicated in Figure 3-5. Dolerite outcrops and sub-outcrops are visible along the first approximately 190m of the spillway axis. The logs of boreholes and trial pits in close proximity to the spillway axis are shown in Table 3-5.

Table 3-5: Spillway Option 2 Boreholes and Trial Pits

Borehole / Trial Pit No.	Description
SP1	Competent dolerite from 0.41 m
SP2	Competent dolerite from 1 m
SP3	Completely to highly weathered dolerite to 5.5 m Medium weathered dolerite to 8.5 m Competent dolerite from 8.5 m
SP4	Weathered sandstone to 1.2 m Weathered mudrock to below 1.7 m
SP5	Colluvial soil to 2.4 m / Weathered mudrock to below 3.3 m
SP6*	Excavator refusal at 1 m on slightly weathered sandstone
SP7*	Excavator refusal at 1.2 m on slightly weathered sandstone
*trial pit	Sandstone outcrop in the left hand side river terrace. Dolerite outcrop in the river

Spillway Option 2 offers better founding conditions along the alignment of the lower chute than spillway Option 1, but the large quantities of rock excavation would be far in excess of the quantities required for the embankment construction and concrete aggregates. This would create the problem of disposal and spoiling of the excess quantities.

c) *Spillway Option 3*

Spillway Option 3 proposes a side channel cut into the left flank, perpendicular to the dam axis on the upper left flank, then curving just in front of the downstream dam toe to intersect the river. There is sub-outcrop of sandstone on the upper left flank, but the remainder of the spillway alignment is underlain by a relatively thick mantle of transported and residual soils. The logs of boreholes and trial pits in close proximity to the spillway axis are shown in Table 3-6.

Table 3-6: Spillway Option 2 Boreholes

Borehole No.	Description
N3	Unconsolidated materials to 7.7 m Highly to medium weathered, soft rock, sandstone to 8.83 m Below 8.83 m slightly becoming unweathered, medium hard to hard rock, sandstone.
NL2/10	Unconsolidated to partially consolidated, transported and weathered dolerite to 3.45 m. Slightly weathered, closely jointed, hard rock, dolerite to 5.05 m. Competent dolerite below 5.05 m.
NL2/3	Unconsolidated to partially consolidated transported and residual materials to 8.63 m. Competent dolerite below 8.63 m.
NL2/4	At river outfall. Competent dolerite below 3.6 m

This upper spillway side-channel would be excavated in sandstone. From mid-slope, the chute and stilling basin excavation would be in dolerite. Being located on the steeper left flank, the depth of excavation, particularly along the western face would be deeper than the corresponding spillway option on the right flank, namely Spillway Option 1.

The sandstone cores derived from the boreholes failed some durability tests and would not be suitable for rock-fill purposes, and would also not suitable for use as crushed aggregate.

Dolerite derived from excavation would be suitable for use as rock-fill and concrete aggregates, although it is doubtful that this option would provide sufficient hard rock dolerite for the project requirements, necessitating an additional hard rock source to supply the shortfall. This would ideally be located on the right flank, where two spillway options are situated.

d) *RCC or CCS Option Materials Sources*

An RCC or CCS dam alternative would be designed with a central in-channel spillway. The aggregate for the RCC dam and for the spillway of the CCS dam would require a separate rock aggregate source, again ideally located on the mid to upper right flank, where the other spillway options are sited.

3.5.6 Site Investigations and Materials Requirements Conclusion

The conclusions drawn are that the founding conditions at the dam site and the materials availability within the impoundment basin would be suitable for the construction of most of the alternative dam types mentioned above. The exception is the earth fill option for which large quantities of embankment shell material would have to be sourced from outside of the basin, with significant haulage cost and potential environmental impacts.

Further site and materials investigations will be required to properly inform the detailed design process. A draft scope of work has been prepared for DWS, and is included herein as Appendix C.

3.6 Dam Type Analyses

3.6.1 Embankment Stability and Seepage Analyses

As part of both the geotechnical investigation and the dam type analyses, feasibility level assessments of dam stability and seepage were undertaken for the following three possible dam types, earth fill embankment with a clay core (EF), earth core rock fill dam (ECRD) and a concrete faced rock fill (CFRD) dam.

The roller compacted concrete (RCC) dam option has been checked for safety factors against overturning and sliding under SEF conditions, but in the case of seepage analyses of a concrete dam built on competent dolerite, the methodology relates more to the presence of seepage paths through weathered or jointed materials.

In this case, the foundations of the RCC dam are likely to be on competent dolerite, but the amount of jointing can only be determined by undertaking the additional geotechnical investigations recommended for the detailed design stage, and would then be fully dealt with by curtain grouting and drainage.

At the 2004 World Conference on Earthquake Engineering in Vancouver, Paper No. 3399 entitled: *Earthquake Aspects of Roller Compacted Concrete and Concrete-Face Rock fill Dams, by Martin Wieland and R. Peter Brenner* was presented. The conclusion was as follows:

“The main disadvantages (of RCC) are the following:

i) Water tightness: Due to the construction of the dam in thin horizontal layers, in the case of high hydraulic gradients, water may percolate along the horizontal construction interfaces. Special measures may be needed at the upstream face of the dam to improve the water tightness, i.e. layer of high paste monolithic mass concrete or a surface sealing by a geomembrane.

ii) Limited experience of engineers and contractors: Few designers and contractors have extensive experience with the design and construction of RCC dams. The design and construction practice are still in development. It should be noted that, at this feasibility study level, these analyses were undertaken with the main objective of determining if there are any fatal flaws with the use of the materials as found in the vicinity of the proposed dam site, for any of the dam types investigated, as well as determining the cross-sectional shape of the dam embankments for feasibility design purposes.

iii) Limited experience with safety and long-term performance: No large RCC dam has been exposed to extreme loadings like strong ground shaking during an earthquake or large floods.

iv) Galleries: Placement of RCC around formwork, which is needed for access galleries in the dam body, is tedious and slows down the construction process.

The main weaknesses of RCC dams are the water tightness under high hydraulic gradients, ageing mechanisms and the unknown performance under seismic loading."

The intervening years have shown an upsurge in the construction of several large RCC dams around the world, as well as significant research into overcoming the perceived disadvantages listed above.

More experience has been gained by engineers and contractors in this period, (including the DWS in-house design and construction divisions themselves), and improvements in RCC construction methodology, resilience to earthquake stresses and movement, mix design, and special treatment of surfaces to improve water tightness, have all combined to improve the confidence in RCC as a dam type, as recently demonstrated at the De Hoop dam, Spring Grove dam in South Africa, and Metolong dam in Lesotho.

The stability and leakage analyses undertaken on other dam types have made use of the available information on the geotechnical properties of the available materials, as has been derived through the geotechnical investigations, but should be reviewed again with a more in-depth analyses as more information becomes available during the detailed design phase.

For all dam types it has been assumed that the foundations would be grouted. Grouting quantities have been adjusted to take into account the likely requirements of each dam type, which have different seepage cut-off arrangements.

The stability scenarios that have been analysed are:

a) Rapid Drawdown

This is when the reservoir level is rapidly reduced from the Full Supply Level (FSL) to the minimum operating level, and is generally only used in an emergency case when there may be some initial signs of failure or distress to the embankment.

It is not possible to 'instantaneously draw down the reservoir level as the outlet works would usually be designed to empty the dam over a period of 4 weeks. In terms of stability, rapid drawdown (RDD) is deemed to be a critical case, as it is assumed that with the rapid reduction in reservoir water level pore water pressures within the upstream shoulder of the embankment do not have sufficient time to dissipate, yet the shoulder loses the support in terms of loading of the reservoir water itself.

b) Seismic Event

An earthquake event would cause cyclic dynamic loading of the embankment, predominantly in the horizontal direction and may cause damage to the embankment but must not cause a total failure of the dam.

According to the seismic hazard map published in 2003 by the South African Council for Geoscience, Figure 3-14 (contained in draft SANS 10160-4), a peak horizontal ground acceleration of 50-100 cm/s^2 has been recorded, with a 10% probability of this being exceeded at least once in a 50 year period.

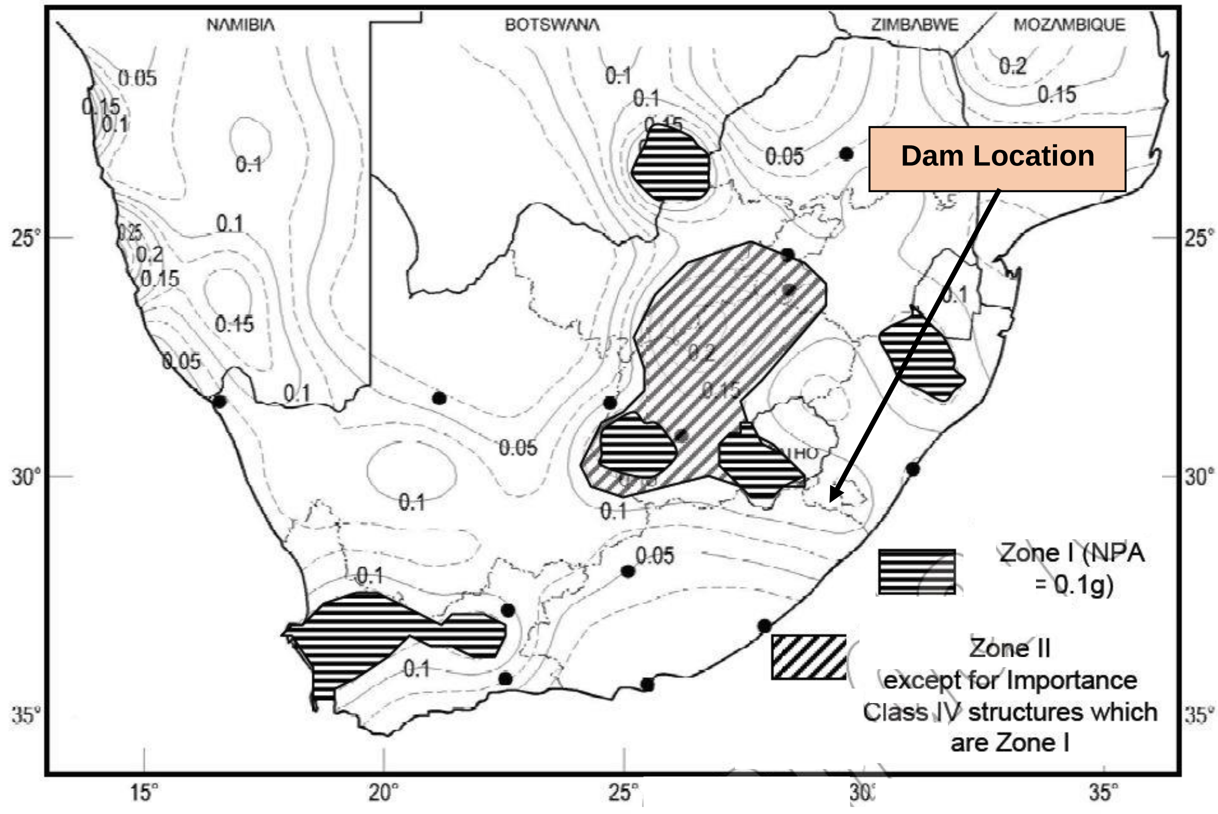


Figure 3-14: Peak horizontal ground acceleration

Taking only this guideline into consideration, this would be considered as a low risk zone, and a value of 0.1g would therefore have normally be applied as a horizontal loading in the design of the embankment.

Prof Andrzej Kijko, the Director of the University of Pretoria Natural Hazard Centre, was assigned to perform a detailed earthquake hazard assessment. From the research undertaken, indications are that there have been some historical earthquake events in the area of influence of this dam, which could merit the consideration of analysis using higher risk factors than those published in SANS 10160-4,2009.

The report and results of the above seismicity study are included as Appendix D to this report. A short extract of the findings of this study is as follows:

"For frequency of ground motion exceeding 1 Hz, the analysis used 1,574 records from 58 earthquakes in the distance range of 0 km to 400 km. (Boore and Atkinson, 2008).

The probabilistic seismic hazard analysis (PSHA) was performed using conventional, Cornell-McGuire procedure (Cornell, 1968; McGuire, 1976; 1978), where the integration across the uncertainty in the peak ground acceleration (PGA) prediction equation is an integral part of the methodology.

In accordance to the current seismic guidelines such as Euro code 8 (2004) and ASCE (2005), three seismic design levels were considered:

- Operating Basis Earthquake (OBE),
- Maximum Design Earthquake (MDE), and
- Maximum Credible Earthquake (MCE).

Given the existence of 594 tectonic faults in the vicinity of the dam site (information provided by Jeffares & Green (Pty) Ltd), an investigation of the effect of potential seismic activity of the faults on the seismic hazard assessment was performed.

The results of the PSHA are given in terms of mean return periods and probabilities of being exceeded for horizontal component of the PGA.

Based on the logic tree formalism, the expected values of horizontal component of OBE, MDE and MCE for the site of Mzimvubu Dam, Eastern Cape are:

- OBE (Return Period 144 years): 0.018 ± 0.003 g
- MDE (Return Period 475 years): 0.039 ± 0.012 g
- MCE (Return Period 10,000 years): 0.159 ± 0.043 g

According to the applied guidelines, the site of the future dam is rated as low risk."

Even though the results of this special study indicate a low risk rating, a conservative approach has been taken and the embankment stability analyses have been undertaken for accelerations of both 0.10g and 0.15g. The analyses indicate that the different dam types will not fail as a result of a 0.15g earthquake loading. The results of these analyses, undertaken with the SLIDE software, are presented below.

c) Liquefaction

This is a loss of shear strength due to increased pore pressures caused by an earthquake. It can lead to catastrophic failure of embankments. Soils most susceptible to liquefaction are saturated sands, silty sands and gravelly sands.

Cyclic loading tends to cause densification of granular soils, just like compaction. However, the phenomenon of liquefaction occurs in certain saturated soils because they are not sufficiently permeable to allow drainage during cyclic loading. They do not allow a decrease in volume, and the tendency to decrease volume is counteracted by an increase in pore pressure with associated reduction in effective stress. The pore pressures gradually build up to equal the total stress and then a state of zero effective stress, or liquefaction, occurs.

Loose materials are more susceptible than dense materials. Materials with less than 5% fines are also thought to be more susceptible to liquefaction. An increase in fines reduces susceptibility.

Liquefaction of the embankment and foundation at Ntabelanga is unlikely given the density and physical properties of the construction materials in question, and the low seismicity of the region.

d) End of Construction

For embankment dams, the end of construction case can often be critical, as pore pressures in the lower half of an earth embankment rise with the additional loading of fill material as it is being placed.

Over time these pore pressures will dissipate but if the embankment is raised too quickly the build-up in pore pressure can result in a lowering of the effective strength of the materials and can lead to a failure.

e) *First Filling*

This analysis investigated the stability of the upstream shoulder during first filling of the reservoir, which would be undertaken shortly after the end of the construction phase. If done too quickly pore pressures in the embankment may not have had time to dissipate and could result in lower effective strengths, as for the end of construction phase. A major storm could potentially effect rapid filling in a matter of hours.

f) *Full Supply Level*

This was the first case to be checked, where the reservoir level is at its maximum operating level, and a steady state seepage condition exists within the embankment.

The recommended minimum factors of safety for each case analysis are presented in Table 3-7 below:

Table 3-7: Recommended Factor of Safety

Design Condition Analysed	Minimum Acceptable Factors of Safety*
End of construction: - downstream slope - upstream slope	 1.3 1.3
***Initial filling*:** - upstream slope	 1.2
***Steady state seepage*:** - downstream slope - upstream slope	 1.5 1.5
***Rapid drawdown*:** - upstream slope	 1.2
***Steady state seepage plus earthquake*:** - downstream slope - upstream slope	 1.0 1.0

** The results of the analysis are expressed as a factor of safety, which is defined as the ratio of available shear strength to that required for equilibrium.*

The slope stability programme *SLIDE version 06*, which is part of the *RocScience Suite* of geotechnical software programmes, was used for the analyses, uses both the Morgenstern-Price and Bishop Limit equilibrium methods.

As discussed earlier, the laboratory test results available for the various construction materials at the time of writing this report were used in this analysis, and more detailed site investigations during the detailed design stage will significantly improve the information available on the materials properties.

Following a precautionary approach, a degree of conservatism has been used in the selection of material properties used in the analyses. Table 3-8 summarises the values used.

Table 3-8: Summary of Material Properties

Material Type	Unit Weight (Kg/m³)	Cohesion (kPa)	Internal angle of friction (degrees)	Permeability (m/s)
Core Material	1 800	2.0	26	1×10^{-9}
Shoulder Material (earth fill)	2 000	2.0	28	1×10^{-7}
Rock fill	2 200	0.0	42	1×10^{-3}
Alluvium	1 800	5.0	23	1×10^{-7}

It has been assumed that the shear strength of the foundation bedrock exceeds that of all other construction materials and that the embankment is rigidly bounded at this interface on the models. Discussion on the findings for the three dam types analysed follows.

3.6.2 *Embankment Dams Stability Analyses Findings*

a) Earthfill Embankment with Clay Core

The earth fill embankment with a clay core was analysed with an upstream shoulder at a slope of 1V: 3H, and the downstream shoulder at a slope of 1V: 2.5H. The crest width was6m, and the height of the dam above river bed level was 65 m. The following cases were analysed:

- The upstream and downstream shoulders for full supply level with steady state seepage conditions;
- The upstream and downstream shoulders with a horizontal seismic loading of 0.1g and 0.15g applied;
- The rapid drawdown case.

The following plots illustrate the failure planes with the minimum Factors of Safety

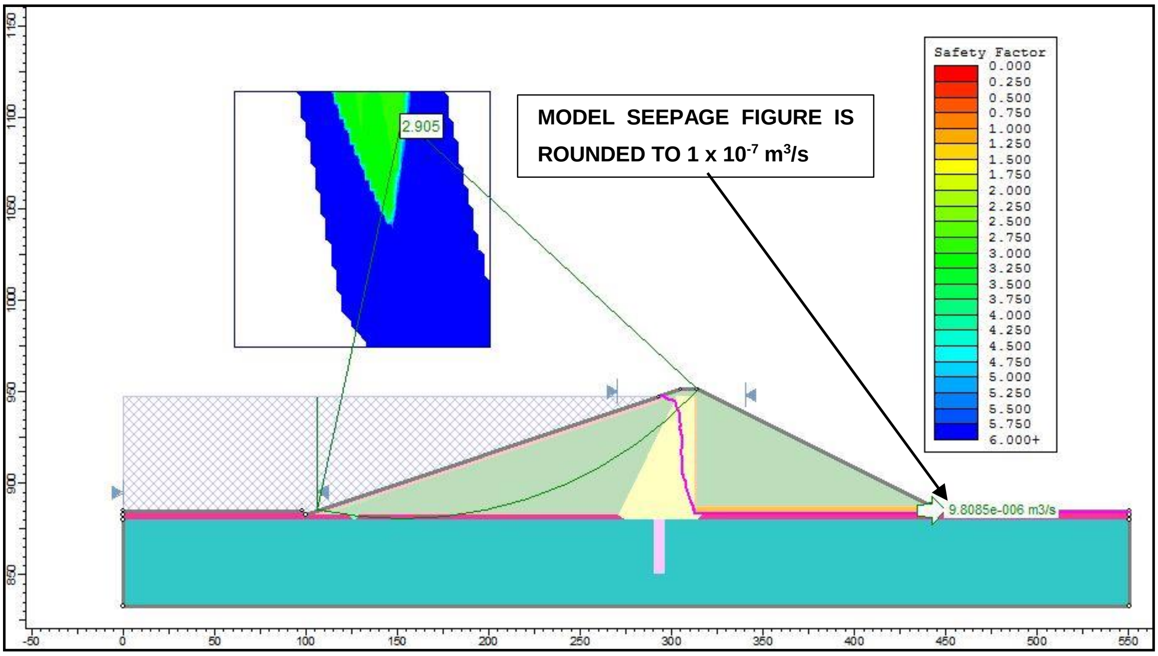


Figure 3-15: Earth fill Embankment: Upstream Shoulder
Steady State Seepage condition at Full Supply Level (FSL): Factor of Safety (FoS) = 2.905

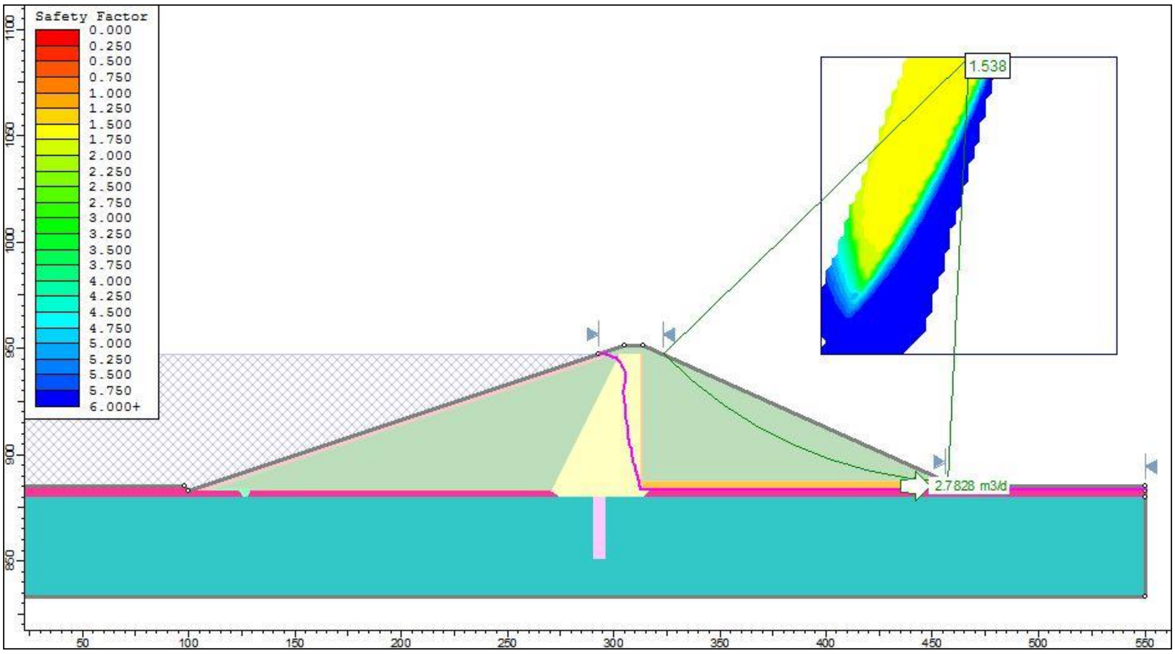


Figure 3-16: Earth fill Embankment: Downstream Shoulder
Steady State Seepage at Full Supply Level (FSL): FoS = 1.538

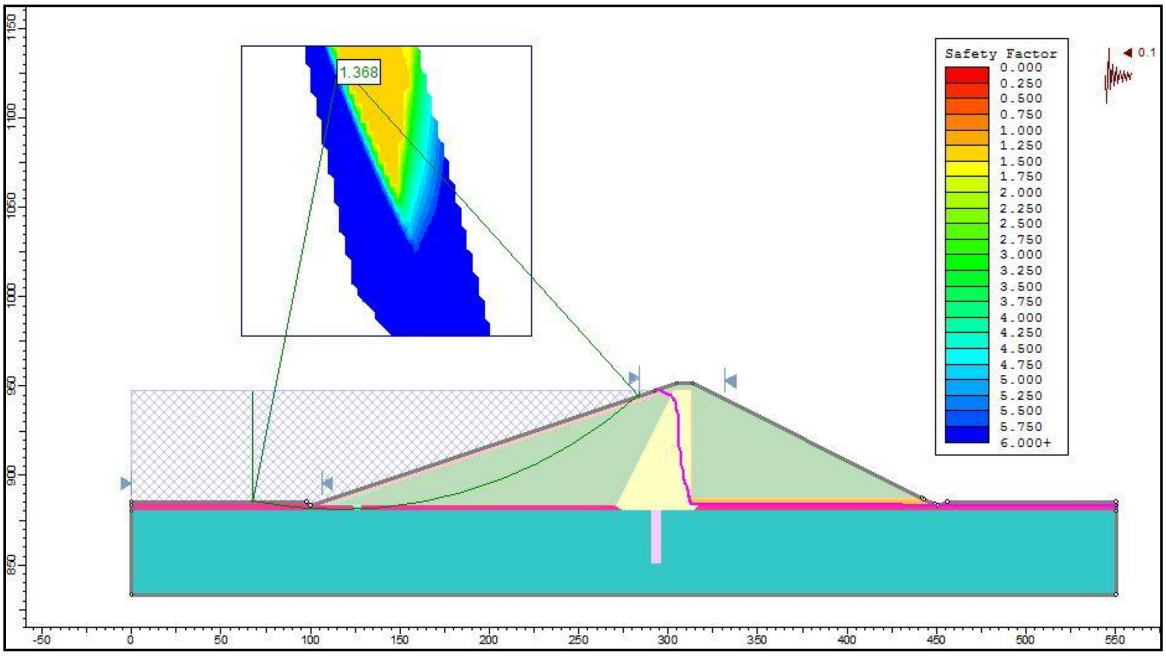


Figure 3-17: Earth fill Embankment: Upstream Shoulder
Full Supply Level with seismic loading 0.1g: FoS = 1.368

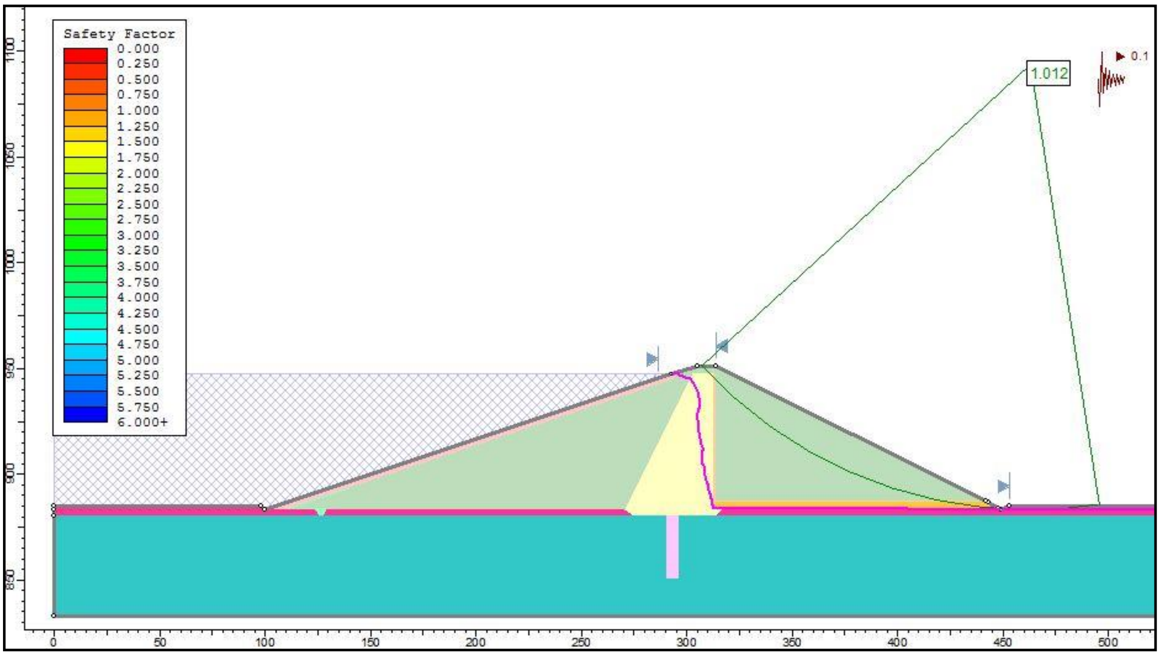


Figure 3-18: Earth fill Embankment: Downstream Shoulder
(Full Supply Level with seismic loading 0.1g: FoS = 1.012)

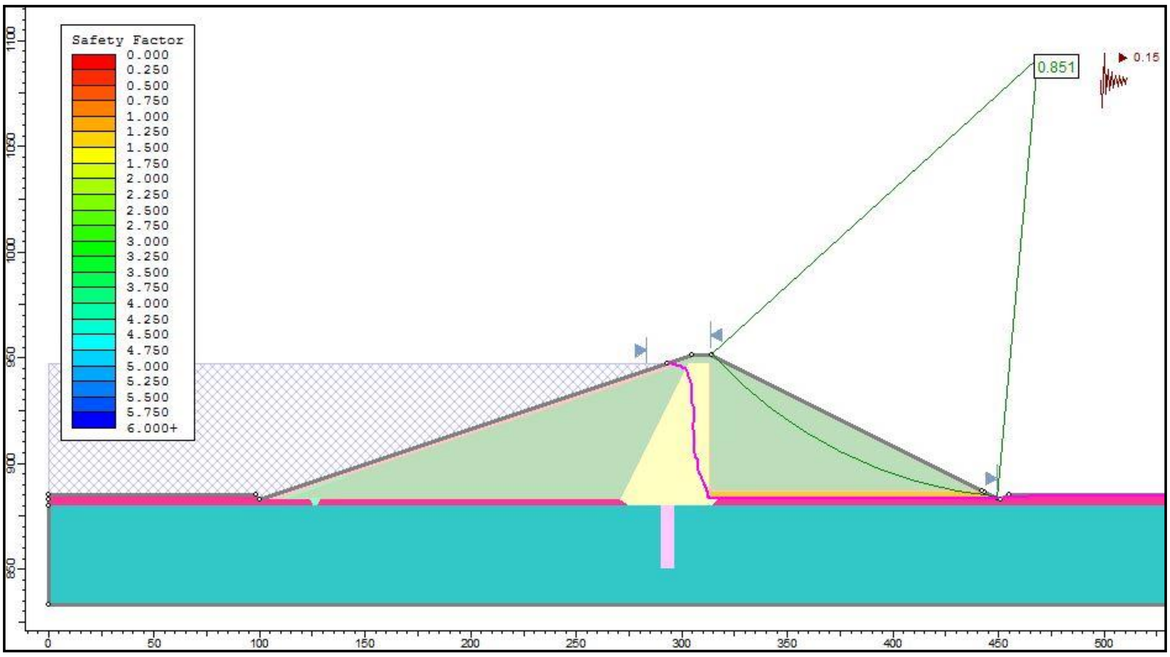


Figure 3-19: Earth fill Embankment: Downstream Shoulder
(Full Supply Level with seismic loading 0.15g: FoS = 0.851)

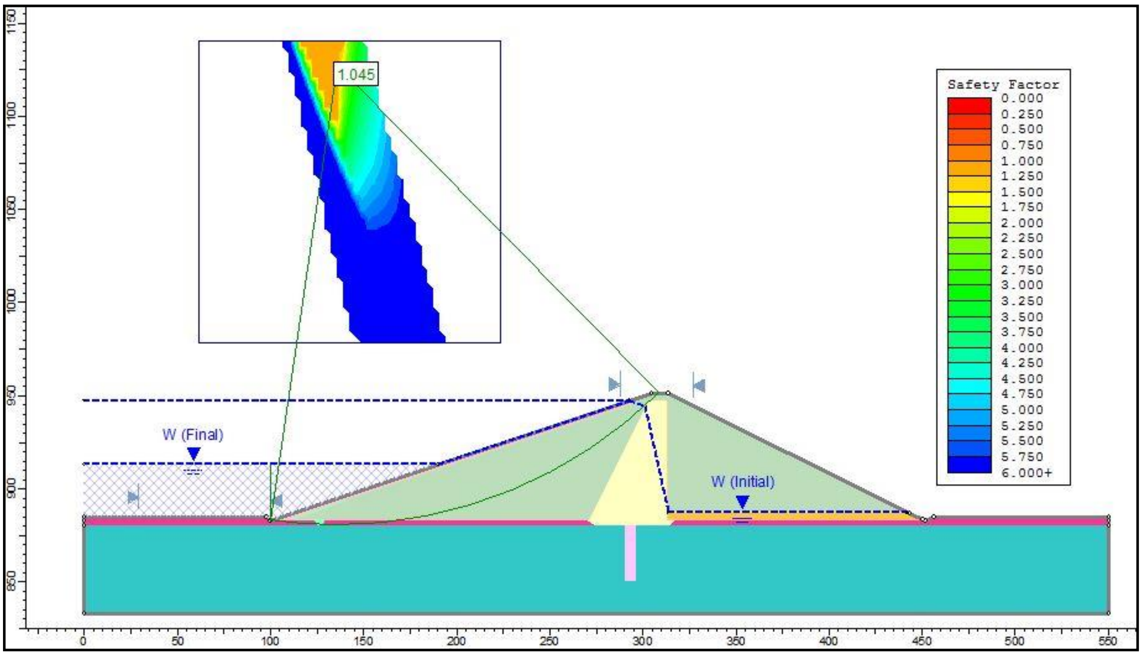


Figure 3-20: Earth fill Embankment: Upstream Shoulder
(Rapid Drawdown (RDD): FoS = 1.045)

b) *Earth Core Rockfill Dam*

A cross-section was analysed with the following geometry, upstream shoulder 1V:1.5H, and downstream shoulder with a slope of 1V: 1.5H. Filters were incorporated on either side of the core. The same loading conditions were applied to the Earth Core Rock fill Dam as for the Earth fill Dam with a clay core.

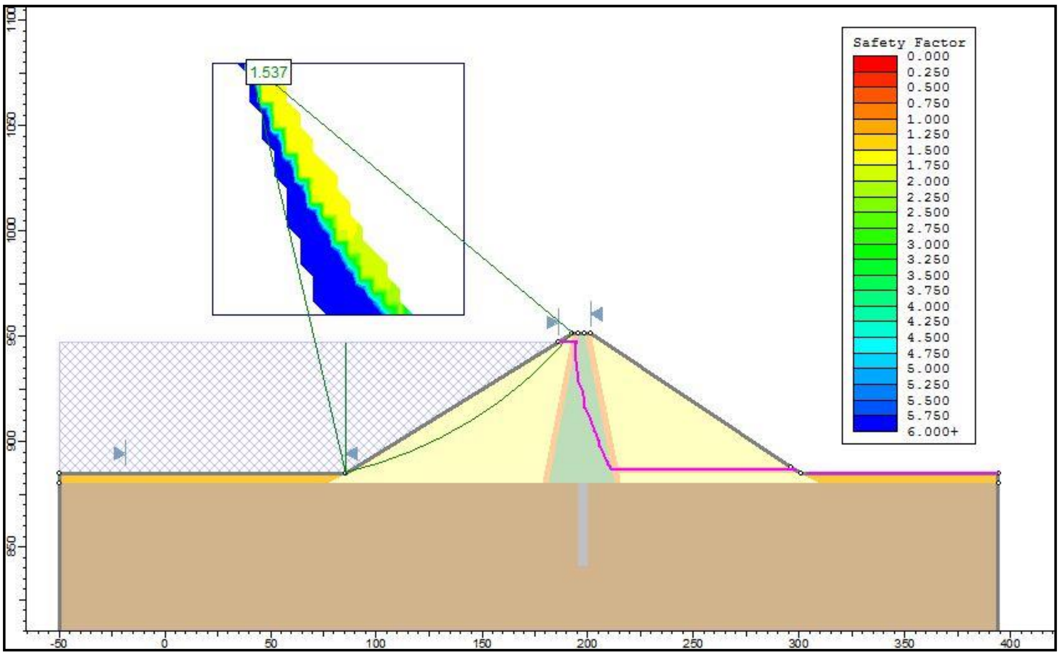


Figure 3-21: Earth Core Rock fill Dam: Upstream Face
(Full Supply Level (FSL): FoS = 1.537)

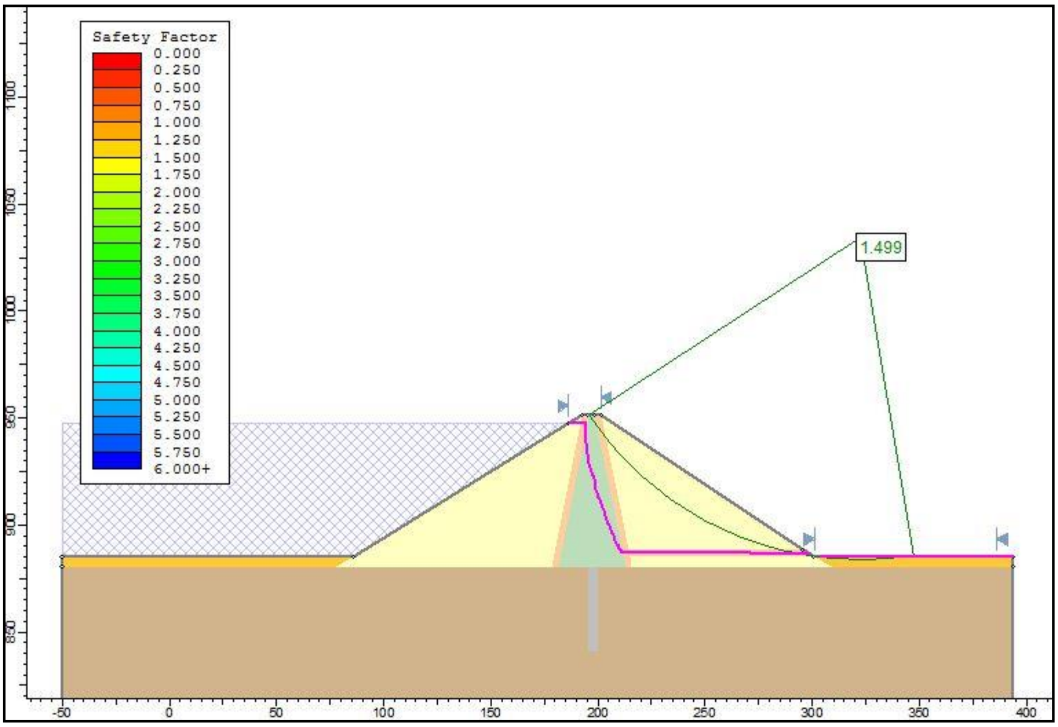


Figure 3-22: Earth Core Rock fill Dam: Downstream Shoulder
(Full Supply Level (FSL): FoS = 1.499)

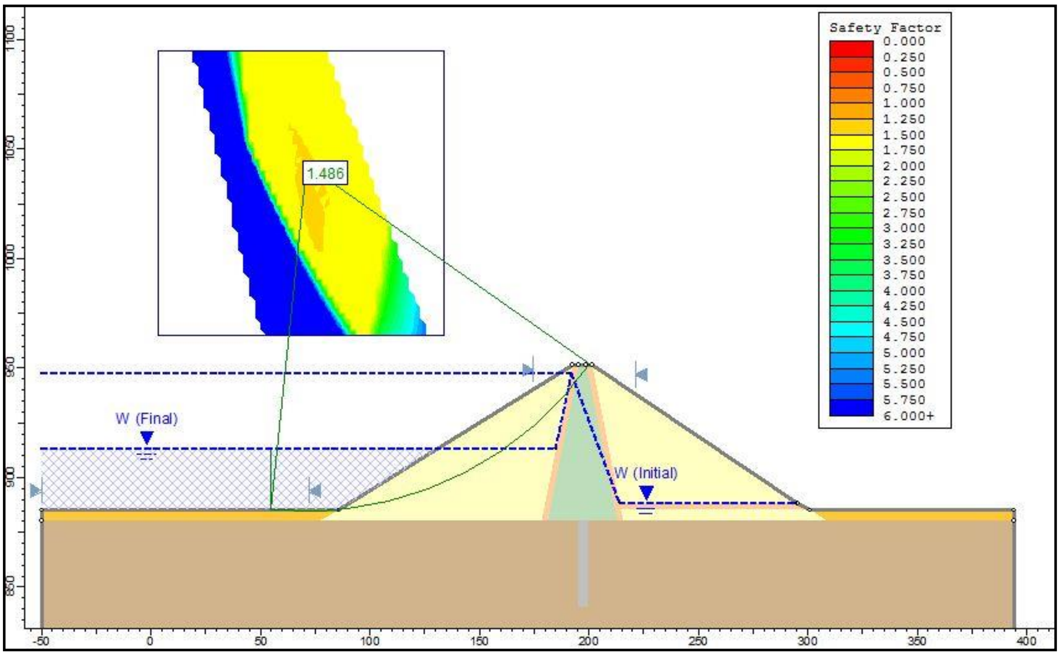


Figure 3-23: Earth Core Rock fill Dam: Upstream Shoulder
(Rapid Drawdown: FoS = 1.486)

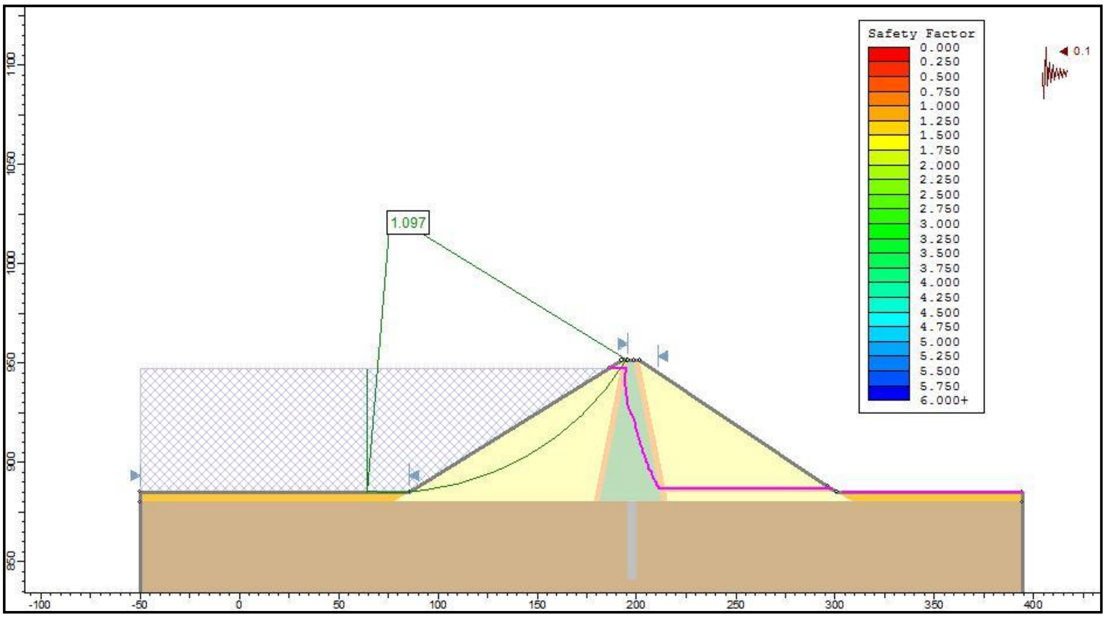


Figure 3-24: Earth Core Rock fill Dam: Upstream Shoulder
(Seismic Loading of 0.1g: FoS = 1.097)

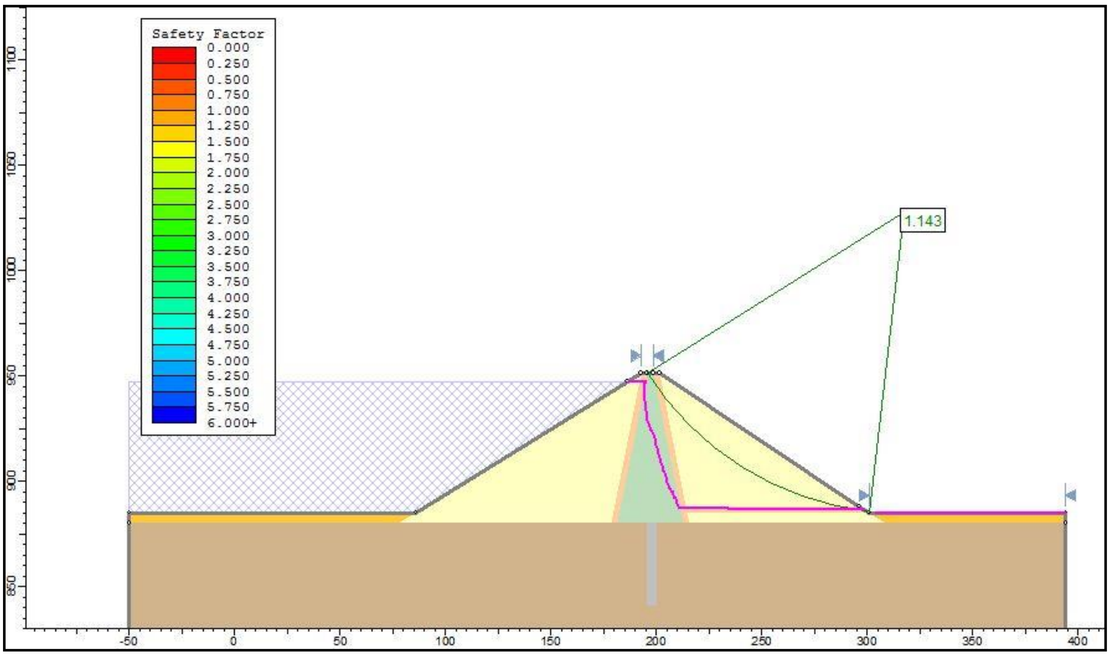


Figure 3-25: Seismic Loading of 0.1g: FoS = 1.143
(Earth Core Rock fill Dam: Downstream Shoulder)

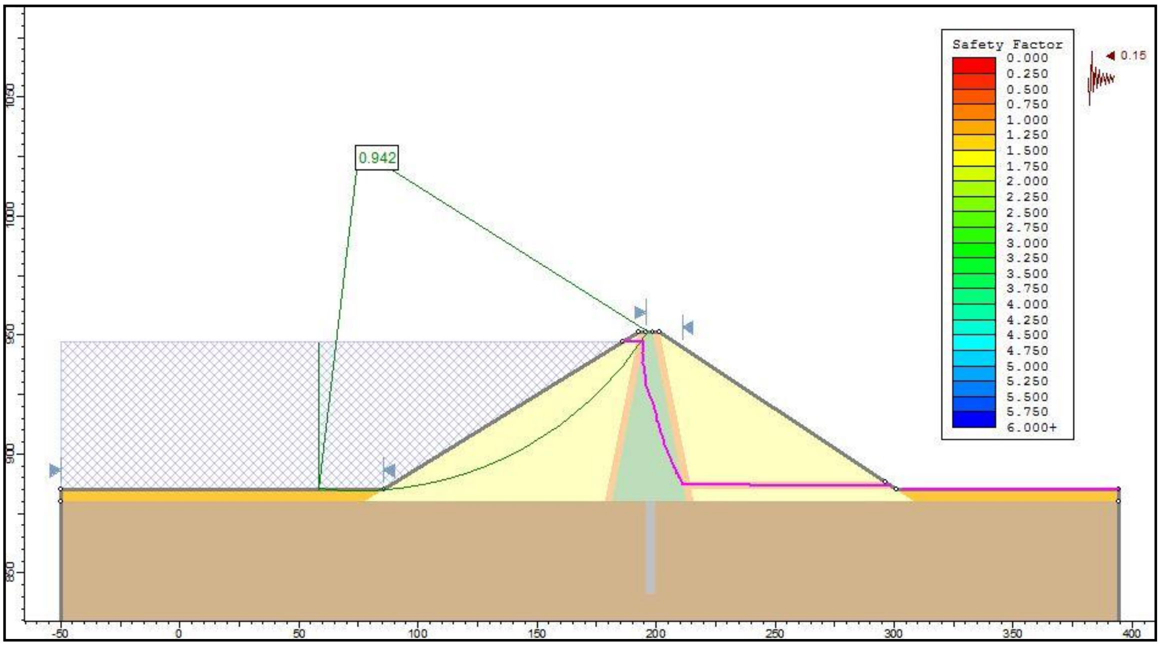


Figure 3-26: Seismic Loading of 0.15g: FoS = 0.942
(Earth Core Rock fill Dam: Upstream Shoulder)

c) *Concrete Faced Rockfill*

The third dam type analysed was a concrete faced rock fill dam with an upstream concrete face, with a cut off at the toe of the upstream shoulder. This cross-section was not modelled for the Rapid Drawdown loading condition as the concrete face, and filter zone on the outer face will prevent the build-up of pore pressures within the body of the dam.

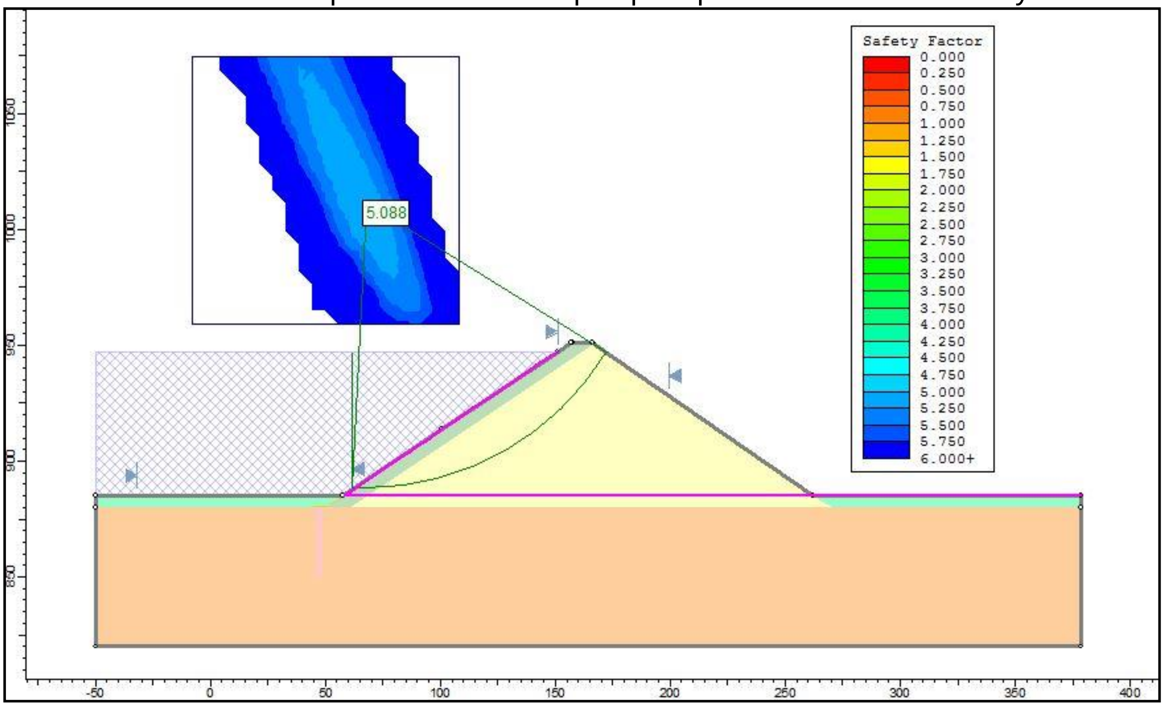


Figure 3-27: Full Supply Level: FoS = 5.088
(Concrete Face Rock fill Dam: Upstream Shoulder)

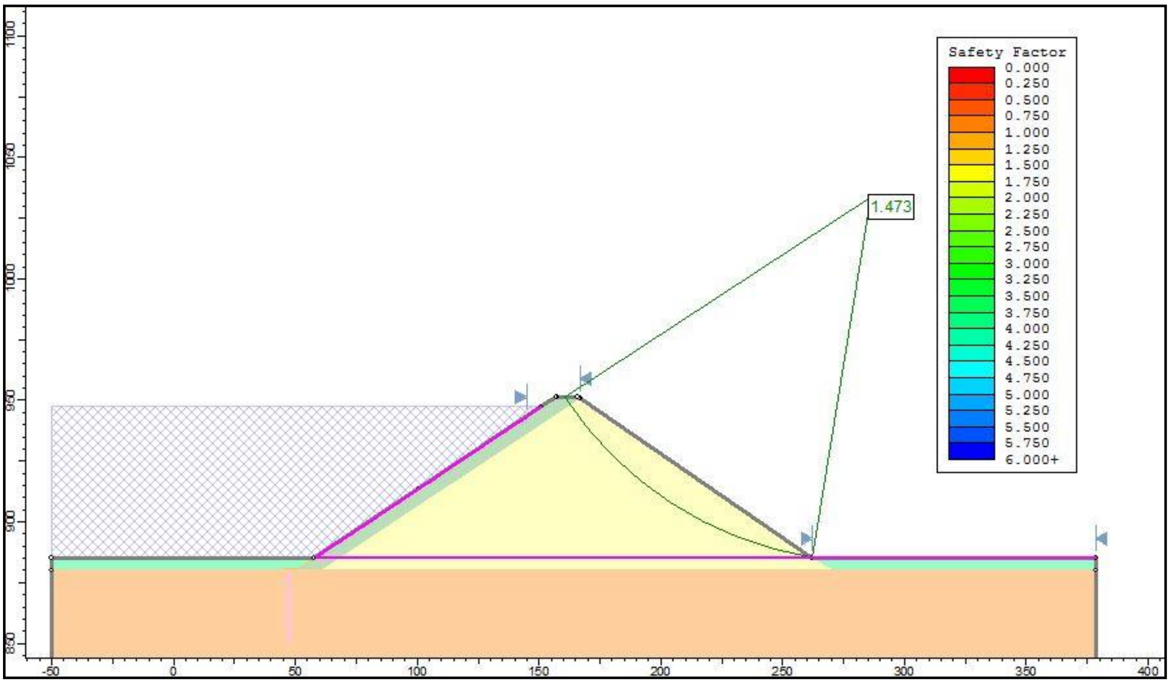


Figure 3-28: Full Supply Level (FSL): FoS=1.473
(Concrete Face Rock fill Dam: Downstream Shoulder)

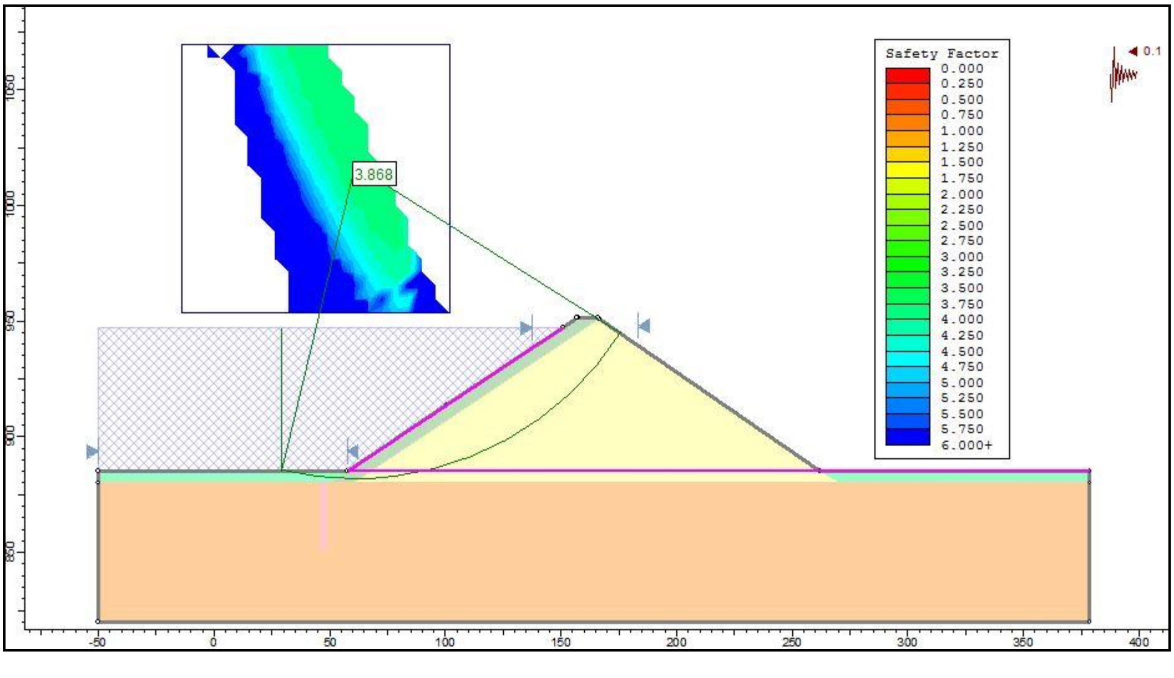


Figure 3-29: Concrete Face Rock fill Dam: Upstream Face
(Seismic Loading of 0.1g: FoS = 3.868)

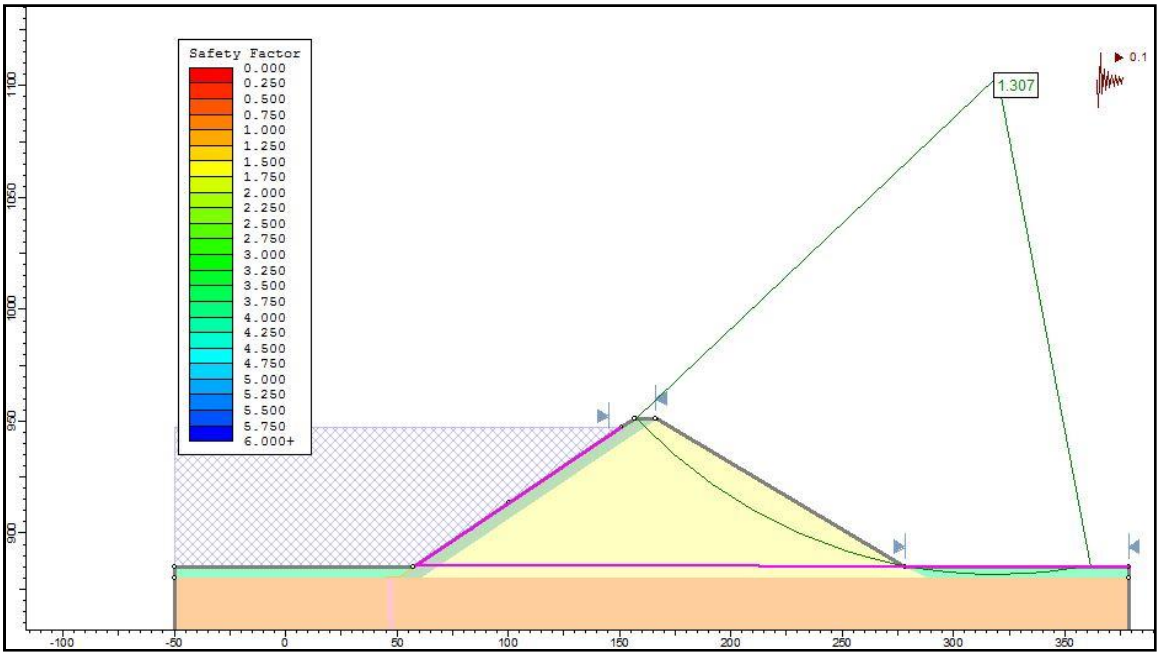


Figure 3-30: Concrete Face Rock fill Dam: Downstream Face
(Seismic Loading of 0.1g: FoS = 1.307)

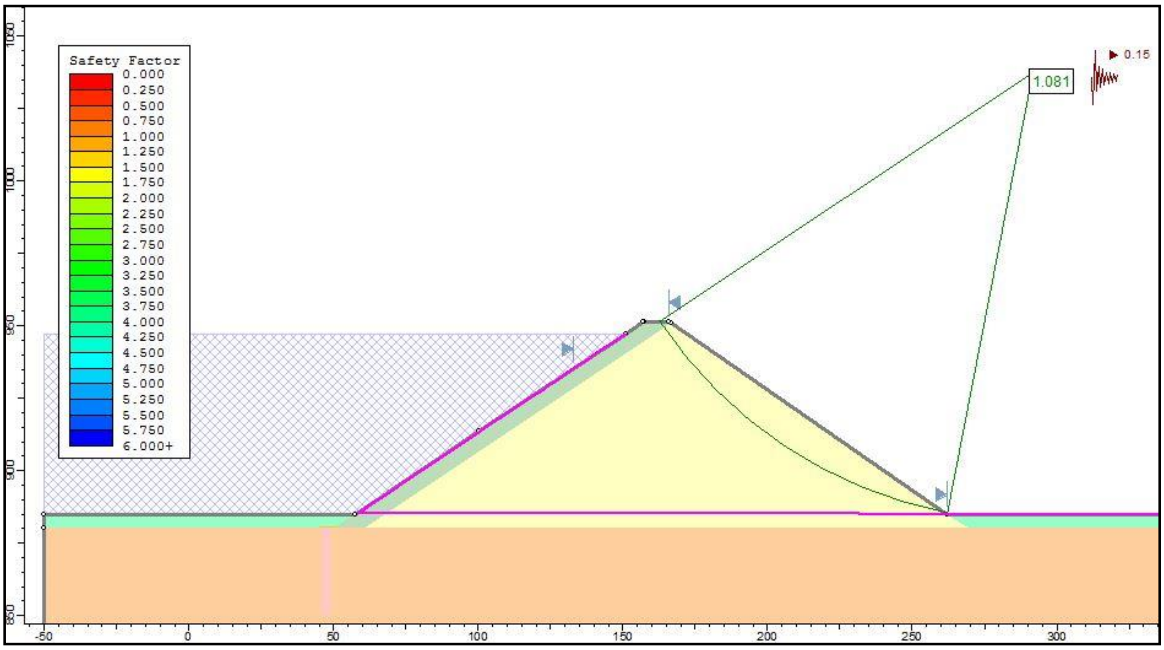


Figure 3-31: Concrete Face Rock fill Dam: Downstream Face
(Seismic Loading of 0.15g: FoS = 1.081)

3.7 Summary of Stability Analyses Results

Refer to Table 3-9 for the summary of the calculated factors of safety.

The only case which showed a Factor of Safety below the recommended minimum is the earth fill embankment dam for the Rapid Drawdown case, where the Factor of Safety was below 1.2.

The available material for use in the construction of the embankment shoulders is a mudstone, which although available in sufficient quantity, when broken down on site and compacted, results in an increase in the percentage of fines and results in a material with a permeability in the order of 10^{-9} m/s.

A permeability of 10^{-9} m/s is generally deemed to be unsuitable for use in the shoulder zones of an earth fill clay core embankment, as it would take a significant amount of time for pore pressures to dissipate with fluctuating reservoir levels which would result in lower shear strengths during any drawdown scenario. As will be shown below, this particular option is shown to not be a preferred solution.

The analyses for the ECRD embankment with clay core, indicated sufficient factors of safety for all cases. The ECRD option also makes good use of the available materials on site from the proposed spillway excavation, and would obviate the need to find an alternative use for the rock or a suitable spoil site. A suitable source of impermeable core material has been identified within the dam basin, which would be economically utilised in the embankment.

Likewise, the option of a concrete faced rock fill embankment (CFRD) passes all the stability criteria, and makes good use of the good quality dolerite available from the spillway excavation. These types of dams are inherently unlikely to fail due to the high permeability of the rock fill body of the dam, i.e. should the concrete face leak, the rock fill, if correctly graded and placed, will accommodate large leakages without jeopardising the stability of the dam.

Pore pressures inside the dam would remain relatively constant irrespective of reservoir levels as the impermeable zone is on the upstream face. The filter zone behind the concrete face should be graded so that the fine particles are on the outer edge of the filter, and the coarser material on the inner side. This will also provide a low permeability material under the concrete face and will assist in limiting leakage should cracks develop in the concrete.

The conclusion was that the above embankment profiles were viable feasibility designs and were suitable for usage in dam type comparative analysis.

Table 3-9: Summary of Calculated Factors of Safety

Dam Type	Analysis Description	Factor of Safety
Earth fill Embankment with Clay Core	Full supply level with steady state seepage conditions, for the upstream shoulder (US) and downstream shoulder (DS)	US: 2.90 DS: 1.53
	Full supply level with steady state seepage conditions and an applied seismic loading, for the most critical failure plane	0.1g: 1.00 0.15g: 0.85
	Rapid Drawdown	1.06
Earth Core Rock fill Dam	Full supply level with steady state seepage conditions, for the upstream shoulder and downstream shoulder	US: 1.53 DS: 1.50
	Full supply level with steady state seepage conditions and an applied seismic loading, for the most critical failure plane	0.1g: 1.05 0.15g: 0.94
	Rapid Drawdown	1.48
Concrete Faced Rock fill Dam	Full supply level with steady state seepage conditions, for the upstream shoulder and downstream shoulder	US: 5.00 DS: 1.50
	Full supply level with steady state seepage conditions and an applied seismic loading, for the most critical failure plane	0.1g: 1.30 0.15g: 1.08

3.8 Embankment Dams Seepage Analysis Findings

Using the models developed for the SLIDE slope stability package, a preliminary seepage assessment was undertaken for each of the three dam types.

The seepage analysis was undertaken prior to undertaking the stability analyses, and was used to determine the phreatic level through the embankment, using the hydraulic properties of the various layers. This phreatic level was then used in the stability analyses described above.

Permeabilities for the various materials were generally estimated from the material grading, apart from the core and shoulder material for the earth dam option for which the properties were determined from the laboratory test results.

These models should be re–run once more information is available at the detailed design stage.

Plots of the anticipated seepage pore pressures are presented below.

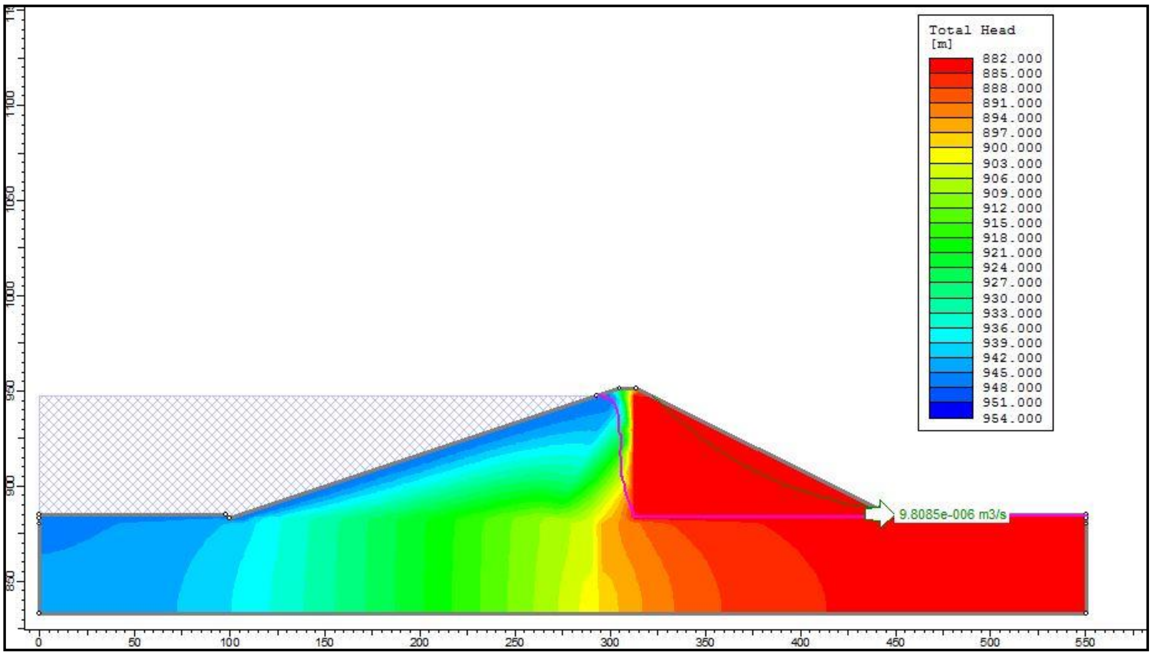


Figure 3-32: Earth fill Embankment: Seepage Analysis

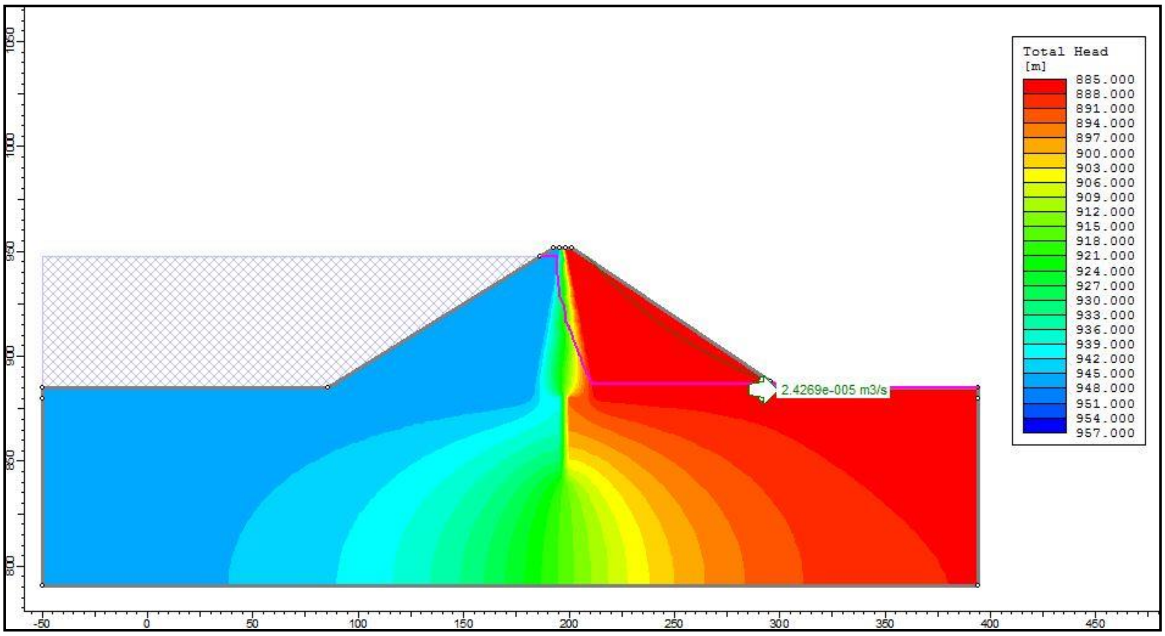


Figure 3-33: Earth Core Rock Fill: Seepage Analysis

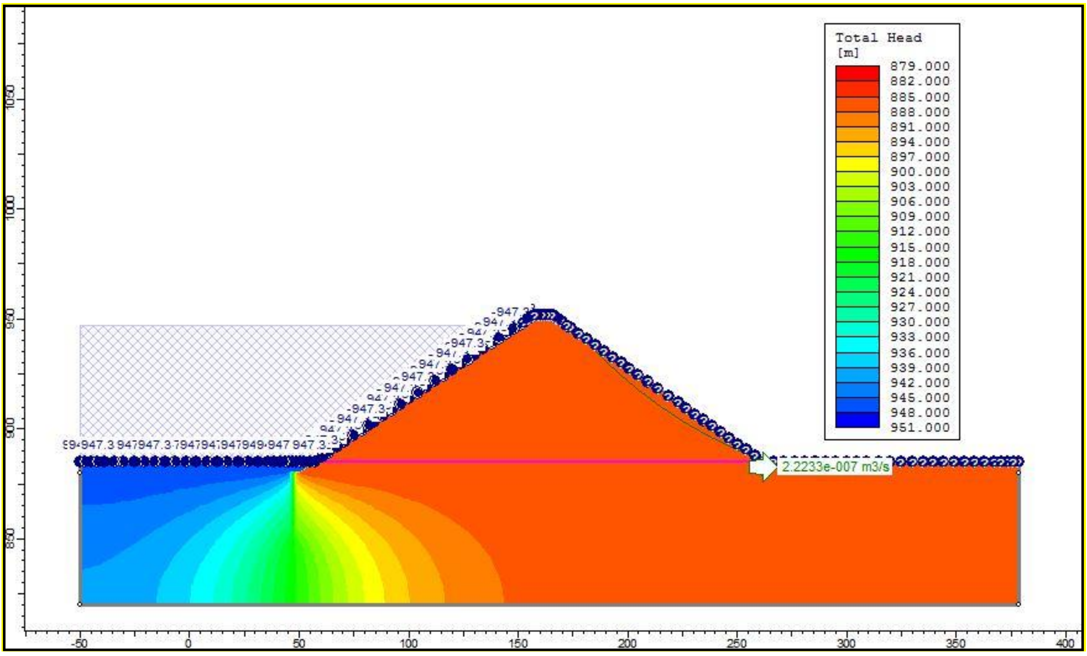


Figure 3-34: Concrete Faced Rock fill Dam: Seepage Analysis

Table 3-10: Summary of seepage per dam type

Dam Type	Seepage (per metre length of wall)
Earth fill Embankment	9.8 x 10^{-6} m^3/s
Earth Core Rock fill	2.4 x 10^{-5} m^3/s
Concrete Faced Rock fill	2.2 x 10^{-7} m^3/s

3.8.1 RCC Dam Option Analysis

CADAM software was used for the stability analysis.

Figure 3-35 shows a general layout of the proposed RCC Dam and its juxtaposition with the associated infrastructure. This also includes recommendations for construction site area allocations.

The model was set up based on simple beam theory. This is a methodology mainly used for gravity dam design. Figure 3-10 shows the proposed cross section of the central uncontrolled ogee spillway. That is considered to be the deepest and for which the structural analysis was performed.

Figure 3-10 shows a configuration with a 1:0.70 (V:H) downstream slope for the wall and spillway, but model runs were also undertaken for a downstream slope of 1:0.75.

The following information and assumptions were used in undertaking the analysis:

- Ntabelanga Dam wall would have a maximum height of 67 m from the river bed level and a total crest length of 440 m;
- Floods would be discharged by means of un-controlled ogee stepped spillway;
- Concrete density of 2 400 kg/m^3;
- Concrete grade C15/53 would be used mainly for the RCC;

- [2]Solid dolerite founding condition with minimum cohesion of 0.3 MPa and minimum angle of friction of 35°;
- The horizontal component of peak ground acceleration is 0.15 g; and
- The vertical component of peak ground acceleration is 0.08 g.

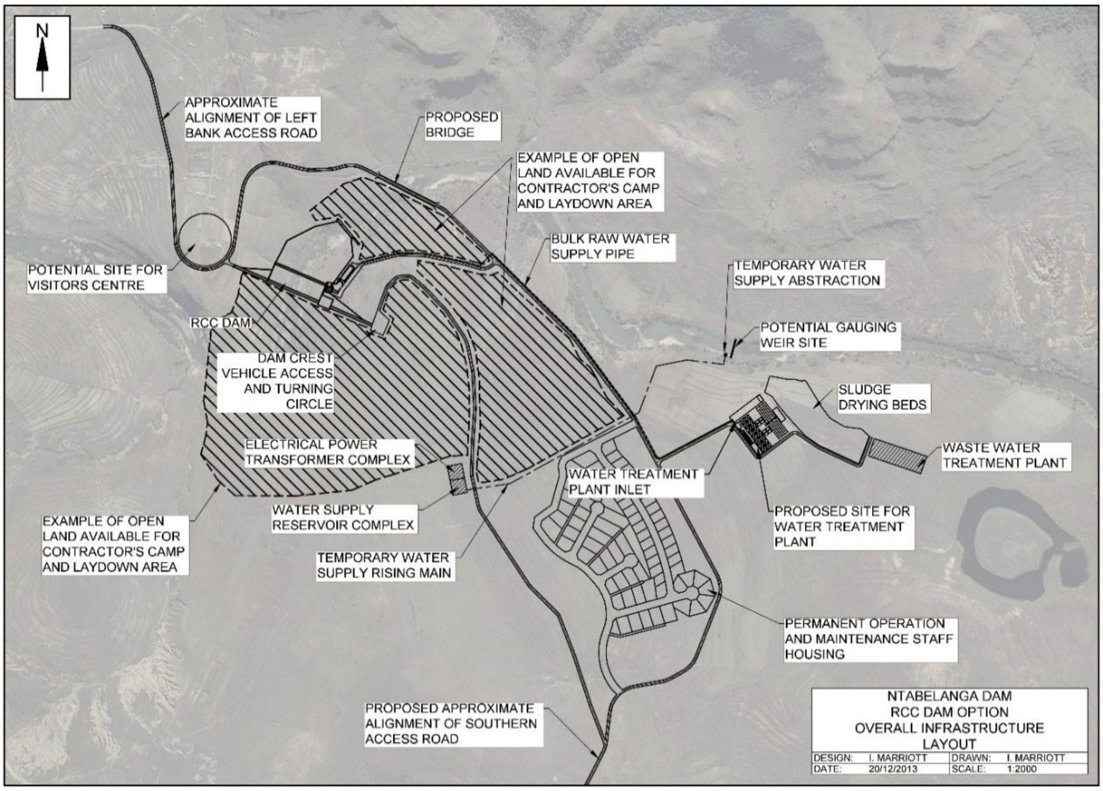


Figure 3-35: General Layout of the Proposed Dam and Associated Works

The loading conditions to be investigated were discussed and agreed with the Department of Water and Sanitation and are shown in Table 3-11.

Table 3-11: Loading Conditions

Type	Case	FSL	RDF	SEF	Silt (S)	Tail water(TW)	Drained (D)	Undrained (UD)	Seismic (SM)
Normal	1	√			√		√		
	2		√		√	√	√		
Abnormal	3		√		√	√		√	
	4			√	√	√	√		
	5	√			√	√	√		√
Extreme	6		√		√	√	√		√
	7			√	√	√		√	

Tables 3-12 and 3-13 present the results obtained from the various load cases in Table 3-11. The analysis results are compared with the allowable factors of safety and maximum stresses according to various international guidelines. Analysis was run for downstream wall slopes of both 1:0.70 and 1:0.75.

[2] *Literature on rock mass properties state cohesion can be in the range of 0.3 to 30 MPa (but this is not a sensitive parameter in this analysis) and an angle of friction up to 55º.*

Table 3-12: Analysis Results and Comparison (1:0.70 d/s Slope)

Type	Case	Tensile Stress (MPa)		Compressive Stress (MPa)		Sliding (residual) Factor of safety (FOS)		Downstream overturning Factor of safety (FOS)	
		R	A	R	A	R	A	R	A
Normal	1	+0.19	0.0	-1.2	-3.0	1.5	1.5	1.5	1.5
	2	+0.4	0.0	-1.4	-3.0	1.3	1.4	1.3	1.4
Abnormal	3	+0.61	0.2	-1.4	-4.5	1.1	1.1	1.1	1.2
	4	+0.56	0.2	-1.5	-4.5	1.1	1.1	1.2	1.2
	5	-0.27	0.2	-0.88	-4.5	2.2	1.1	1.7	1.2
Extreme	6	-0.07	0.35	-1.04	-4.5	1.9	1.0	1.5	1.1
	7	+0.77	0.35	-1.5	-4.5	1.0	1.0	1.0	1.1

Legend - ***A*** *= Allowable - = Compression* ***R*** *= Result + = Tension*

Table 3-13: Analysis Results and Comparison (1:0.75 d/s Slope)

Type	Case	Tensile Stress (MPa)		Compressive Stress (MPa)		Sliding (residual) Factor of safety (FOS)		Downstream overturning Factor of safety (FOS)	
		R	A	R	A	R	A	R	A
Normal	1	+0.03	0.0	-1.1	-3.0	1.62	1.5	1.54	1.5
	2	+0.22	0.0	-1.9	-3.0	1.4	1.4	1.4	1.4
Abnormal	3	+0.43	0.2	-1.9	-4.5	1.1	1.1	1.2	1.2
	4	+0.36	0.2	-2.0	-4.5	1.2	1.1	1.3	1.2
	5	-0.4	0.2	-1.2	-4.5	2.4	1.1	1.83	1.2
Extreme	6	-0.22	0.35	-1.4	-4.5	2.1	1.0	1.65	1.1
	7	+0.57	0.35	-2.0	-4.5	1.0	1.0	1.1	1.1

Legend - ***A*** *= Allowable - = Compression* ***R*** *= Result + = Tension*

These feasibility level results show that factors of safety for sliding and overturning are very close to those allowable for the 1:0.70 downstream slope option, and are conservative for the 1:0.75 downstream slope option. In both options, some of the tensile stress results are higher than allowable.

The eventual geometry of the dam wall would be determined following an extensive detailed design process including finite element and numerical elastic analyses, and this is normally a balance between minimising cost and meeting all of the allowable safety criteria.

This would include consideration of various cross section profiles, mix designs, and tensile crack control/induction methodologies. This will also include considering whether a sloped (rather than vertical) upstream face, or horizontally arched upstream face option is a beneficial and economic solution.

Typically RCC dams are built with downstream slopes of between 1:0.70 and 1:0.80, but this can be steeper on the upper part of the embankment if a non-symmetrical slope approach (base slope shallower than higher up the wall) is adopted.

For the feasibility design and costing of the Ntabelanga Dam, a simple symmetrical profile as given in Figure 3-10 has been adopted, with a slope of 1:0.70.

3.9 River Diversion Works

For each dam type and spillway options analyzed, consideration was given to the construction methodology and sequencing with particular attention to water diversion during construction.

The average flow of the Tsitsa River at the proposed Ntabelanga dam site is 13.5 m^3/s. Two different flood events were considered for the design of the diversion works. The 1 in 5 year flood of magnitude 500 m^3/s was used for RCC dam type while 1 in 20 year flood of magnitude 1 000 m^3/s was used for embankment dam types.

A diversion tunnel is a possibility but this might cost significantly more than the temporary diversion conduits described below. The diversion tunnel option could still be considered, but would require additional geotechnical investigations to verify ground conditions adjacent to the dam wall.

For the purposes of the comparison of dam types, the flood control works design focused on making as much use as possible of required permanent works. These aspects will be revisited during the detailed design phase, and it will also be an option for the contractors to propose alternative methodologies in their bids if this project goes out to tender.

a) RCC Option

In the case of an RCC dam option, minor overtopping during construction is acceptable. Given this, a 1 in 5 year flood event of magnitude 500 m^3/s was considered adequate for the design of the diversion works. The diversion conduit would be contained within the spillway section adjacent to the proposed permanent outlet works.

The diversion conduit would be designed so that when no longer required as a temporary river diversion, i.e. just before impoundment of the completed structure has commenced. The diversion section entrance would be permanently closed using stop logs, filled with pumped concrete and grouted.

b) Embankment Dam Options

For the embankment types of dam wall, the convention has been to construct an upstream outlet tower with multiple drawoff levels, linked to a steel pipe conduit encased in concrete to convey flow from this tower under the dam embankment to the outlet works, within, or near to, the toe of the downstream embankment.

It has also been common practice to design this outlet conduit as a river diversion system during construction. For this configuration, the conduit would extend under and upstream of the outlet tower base, to allow river diversion by cofferdam and through-flow during construction. The upstream conduit extension would be plugged permanently to commence impoundment

This conduit would be offset as far as possible to the side of the main river channel to minimize the impact of the diverted river flow on the conduit works during construction. In addition, the outlet works will convey water to the downstream works below the dam, which need to deliver raw water to the downstream works on the right-hand bank of the Tsitsa River, including a water treatment works, (possibly) a raw water pumping station, and a hydropower plant.

The approach proposed is to construct cofferdams firstly to divert river flow whilst the conduit itself is constructed, then later to divert flow through the conduit itself. Once the full river flow is diverted through the completed conduit, protection of the main works would be via upstream and downstream cofferdams, appropriately sized to cope with the temporary impoundment when dealing with a routed 1 in 20 year flood. This option would allow construction of most of the embankment and outlet tower works in dry conditions.

Once the dam embankment works are up to a level that can safely contain the rise in flood water level, then the cofferdam could be removed or lowered to provide access for construction.

Depending upon the approach and methodology chosen by the contractor, and the rate of construction progress, another option would be to construct a lower upstream cofferdam to deal with average flows, and to rely upon the partially completed dam wall works to contain larger floods up to the 1 in 20 year return period figure quoted above.

With regard to the latter condition, this would be when the embankment and core are at a safe height and state of completion to act in the same way as fully completed. In the case of a concrete faced solution, this would be when the upstream face of the partially completed dam wall has been constructed to a safe height and is protected on the upstream face against damage that could occur during this flood condition.

In all cases, these initial works would be developed in the first dry season of lower river flows. The conduit would thus be sized primarily for its ultimate normal operational requirements, but checked to ensure that the 1 in 20 year return period flood (for the embankment dam type options) could also be routed through these temporary works with rise in water level limited to that which can be tolerated by the cofferdams, or a 1 in 5 year return period flood could be tolerated by completed-to-date works, in the case of RCC dam.

For this dam type analysis exercise, a twin conduit system is proposed, given that DWS normally require dual outlet systems under dams to provide for redundancy and backup in case one outlet conduit needs to be serviced, repaired, or becomes unserviceable.

In the long term, this outlet conduit system and outlet works would have three functions in the case of embankment dams, namely:

i. To deliver raw water to the outlet works supplying:
   a. 1.2 m^3/s peak flow to the water treatment works;
   b. 1.2 m^3/speak flow to the raw water pumping station (for irrigation – if this option is adopted); and
   c. Up to 25 m^3/sec flow to be released downstream for the EWR and through the hydropower plant
ii. To effect rapid drawdown if required for operational purposes.
iii. To convey flood waters away from the works during construction

In the case of the RCC dam option, it is proposed that the river diversion conduit is purely a temporary solution to divert flood waters around the works during construction. This diversion would be permanent plugged once not required, i.e. once impoundment commences.

Given the net head available under these conditions and designing the conduit as a single circular pipe culvert, this would require a minimum conduit diameter of some 4.5 m.
Such a conduit diameter would obviously be more than adequate to meet function i. above.

The size of conduit required to convey the flood condition in iii above would depend on the type of the dam and the construction sequencing as well as the eventual Contractor's approach and methodology.

c) *River diversion conduit sizing for embankment dams*

Using the depth verses capacity curve for the Ntabelanga Dam, and calculating the hydraulic capacity for various diameters and lengths of conduits below the dam wall, modeling was undertaken to route a 1 in 20 year (1 000 m^3/s) peak flood hydrograph through the reservoir with a duration of 72 hrs, in order to check on the maximum upstream water depth for various conduit sizes. This analysis was performed specifically for embankment dams river diversion works. The embankment dams require higher safety margin against overtopping during construction as opposed to concrete dams.

Figures 3-37 and 3-38 below show an example of the model outputs.

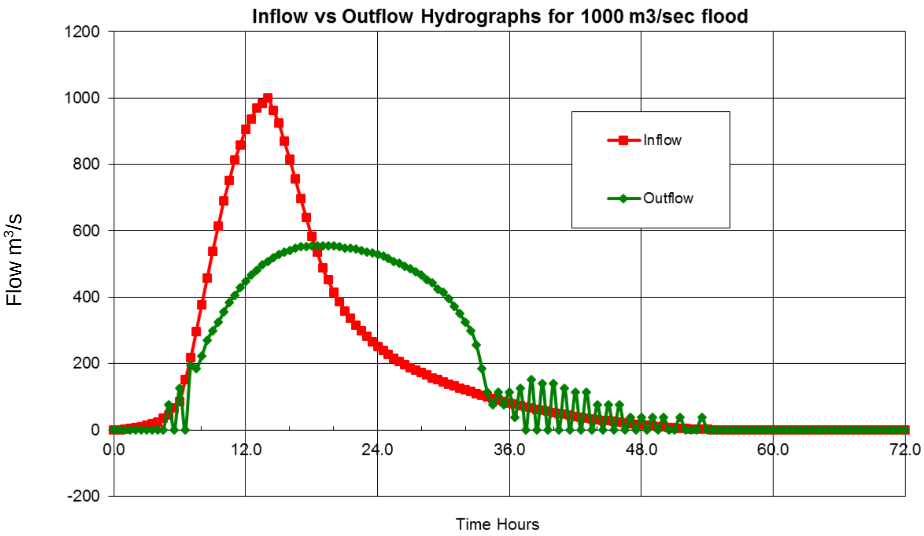


Figure 3-36: Inflow and Outflow Hydrograph for Conduit below the Dam

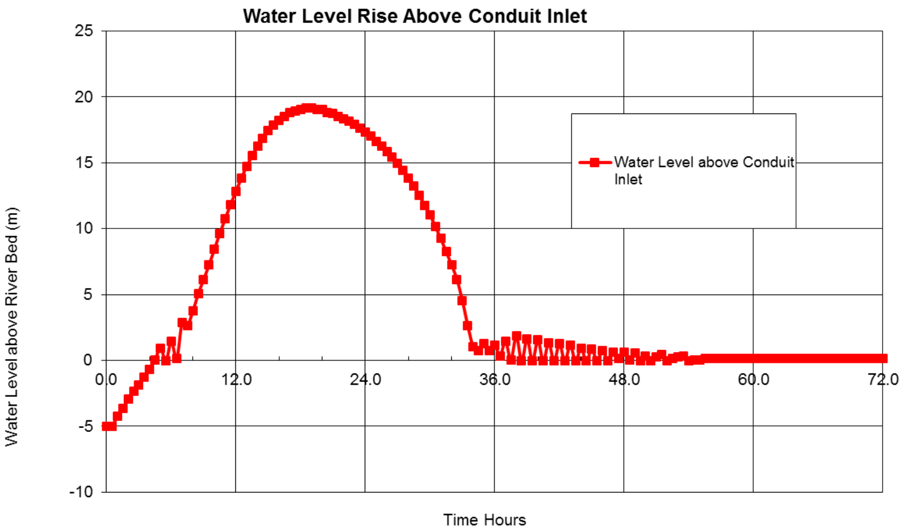


Figure 3-37: Rise in Water Level Upstream of Conduit for a Particular Conduit Size

The analysis was repeated for a range of conduit sizes from 3 m to 6 m diameter.

A pair of 4.5 m diameter conduits would, under the 1 in 20 year flood condition, produce a rise in upstream water level of some 25 m above river bed level. This requires a very high cofferdam.

A pair of 6.0 m diameter conduits would, under the 1 in 20 year flood condition, produce a rise in upstream water level of some 12 m above river bed level, requiring a much lower cofferdam.

At this feasibility level of analysis, for the purposes of comparison of embankment dam options, costings for all embankment dam options was based on the 2 x 6 m diameter conduit option.

d) *River diversion conduit sizing for RCC dam*

In the case of RCC dam construction, the ongoing works are normally more tolerant to overtopping and it is therefore in order to reduce the river diversion flood criteria to a 1 in 5 year return period flood.

This reduces the maximum flood flow rate from 1000 m^3/s for the embankment type of dam to some 400 to 500 m^3/s (depending upon the flood assessment method), and a 500 m^3/s figure was therefore used at this feasibility level of design.

The actual flood diversion approach and methodology should be revisited during the detailed design stage, as well as being a required method statement to be submitted by tenderers during the contract procurement process.

For feasibility costing purposes, it was assumed that the main outlet works structure would be constructed first, and that a diversion conduit would also be constructed in the river bed and alongside this outlet works whilst the river was diverted by cofferdam.

The conduit could be a reinforced concrete opening, passing through the spillway section, and would have to be carefully designed such that it could be permanently plugged and sealed once no longer required, and as impoundment commences.

The sizing of the river diversion works is dealt with in detail under Section 5.3.

4. SELECTION OF PREFERED DAM TYPE AND SPILLWAY OPTION

4.1 Comparison of Capital Costs

All cost estimates are based upon 2014 price levels. Please see Report No. P WMA 12/T30/00/5212/15 for details as to how these cost estimates were developed.

These consider not only the dam wall and spillway costs, but also take into account the costs of outlet works, routes of raw water pipelines, stilling basins, access roads, and temporary works requirements.

Haulage distances and costs of construction materials not available close to the dam and within the impoundment area were taken into consideration in the unit rates, as well as the additional cost implications of the removal and disposal of excess excavated materials, and the environmental costs of reinstating of those borrow pits and quarries which would not be inundated following impoundment.

Sensitivity to ranges of the major quantities unit rates was also tested to produce a ranking of total capital cost for the dam type options investigated.

In addition, for the highly sensitive cost of an RCC mix, a costing was developed for both low and high paste solutions from basic principles and taking into account all the individual processes required, as well as the cost of materials sourcing and processing, delivery of cement, fly ash and other special additives.

Bills of quantity were drawn up for each dam type and spillway arrangement, and these quantities were priced using costing information from several sources including internal cost estimation databases and the Department of Transport's annually published estimating rates, for past and ongoing dam construction projects, including the following dams:

- De Hoop
- Berg River
- Metolong (Lesotho)
- Braamhoek
- Bedford
- Spring Grove
- Ludeke
- Dikgathlong (Botswana)

The sensitivity analyses carried out on the major cost items included soft and hard excavation, reinforced concrete, steel, RCC, embankment material, clay core material, and filter material.

For each large volume item, a range of rates was developed based upon the contract rates sourced during research into the above projects. Some outlier values were ignored where special circumstances (e.g. very long haul for materials sources) did not apply to the particular situation at the Ntabelanga Dam site.

The cost estimates for all dam and spillway options were run using low, medium and high unit rate scenarios to test whether the ranking of different dam types changed with each scenario.

Table 4-1 below summarises the results of this analysis.

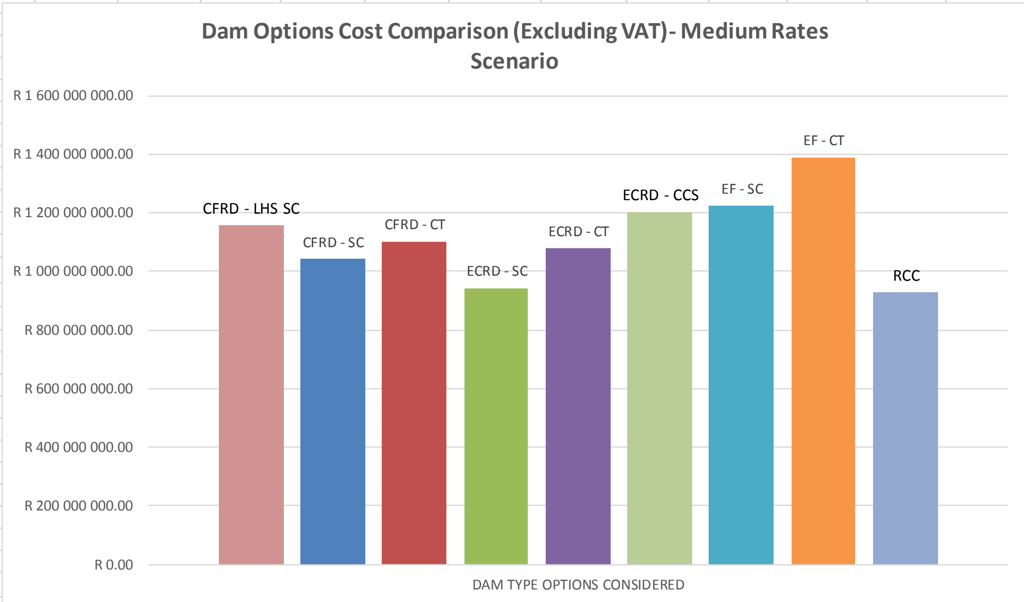
Table 4-1: Capital Cost Comparison of Dam Type & Spillway Options

Option No.	Dam Wall Type	Spillway Type	Capital Cost (R'million)		
			Low	Medium	High
1	CFRD	Side Channel (SC)on Right Flank	932	1 043	1 153
2	CFRD	Cut-Through (CT)	989	1 103	1 218
3	CFRD	SC Left	1 036	1 158	1 279
4	ECRD	SC Right	848	944	1 040
5	ECRD	CT	977	1 079	1 181
6	Earth fill	SC Right	1 147	1 224	1 301
7	Earth fill	CT	1 305	1 390	1 474
8	RCC	Central Ogee	769	929	1 089
9	CCS	Composite Central Channel Spillway	1 009	1 203	1 397

(green)	Lowest
(yellow)	Second Lowest

The green highlighted cells show the lowest cost option, which is, for the low and medium rate ranges of major quantity unit rates - Option No. 8 – an RCC dam, with Option No.4, the ECRD dam with a Side Channel Spillway cut through the Right-hand Flank, coming second lowest. Only for the highest rates does this ranking reverse.

Figure 4-1 shows the comparative costs of all the options for the medium rates case, as well as main materials quantity information and how much excavated material needs to be disposed of to spoil.



Percentage of lowest cost option	125%	112%	119%	102%	116%	130%	132%	150%	100%
Cost Excluding VAT R'millions	**1 158**	**1 043**	**1 103**	**944**	**1 079**	**1 203**	**1 224**	**1 390**	**929**
Total rock excavation used in embankment	450 000	780 400	1 186 179	815 600	927 600	166 700	0	0	382 381
Total rock excavation to spoil	0	0	452 500	0	737 300	0	858 400	1 716 650	39 413
Total all materials to spoil	187 700	79 000	610 500	79 000	895 300	23 000	978 200	1 915 350	0

Figure 4-1: Dam Options Cost Comparison

As can be seen for the “medium rates” scenario, which is considered to be a reasonable assumption given the nature of the dam site and proximity to construction materials, the RCC and ECRD (with right hand side channel spillway) options are ranked very closely, with all other options more than 10% higher in cost.

It is therefore concluded that there is little to choose between these two options as far as costs are concerned, and other factors were therefore considered to inform the decision-making process.

4.2 Other Factors Affecting Decision-Making

The following considerations were made:

- Ability to build in stages if a smaller dam is built first and then raised
- Speed of implementation to first water delivery
- Ability of DWS Infrastructure division to undertake detailed design in-house
- Ability of DWS construction division to undertake construction in-house
- Simplified infrastructure layout and access
- Low maintenance inputs
- Less risk when dealing with floods during construction, and
- Environmental impacts.

4.2.1 *Ability to build in stages if a smaller dam is built first and then raised*

After Phase 1 of the study, it was confirmed that Phase 2 of this study should focus on the conjunctive use option requiring the larger Ntabelanga Dam size, and that Phase 2 should also include feasibility level analyses of this option to finally determine its viability in terms of cost–benefits.

There was some discussion regarding a potential situation arising where either the conjunctive use scheme ceased to be a viable option, or the implementation of the Lalini Dam and hydropower component of the conjunctive scheme were to be significantly postponed, or even cancelled.

Whilst this situation is unlikely to arise, if this did happen then the alternative option would be to revert back to the "minimum sized" (approximately 0.15 MAR) Ntabelanga Dam that would still adequately supply potable and irrigation water requirements, but not hydropower. Such a dam would impound just 10% of the 1.5 MAR dam, and would be roughly half the dam wall height and two-thirds of the wall crest length, as has been previously described above.

As this situation may even still be unresolved towards the end of the feasibility study, it was considered to be prudent to investigate a dam type that could be built in two stages, thus deferring capital cost and avoiding possibly unnecessary capital expenditure.

Earth embankment or CFRD dams do not easily lend themselves to being raised in two stages, especially where a side channel spillway is to be installed. This is especially difficult if the degree of raising is to be nearly double the original dam wall height. Undertaking such raising works on a "live" earth or CFRD dam is not to be undertaken lightly, and such raising would also require a completely new side channel spillway to be constructed. Outlet works passing under the first stage embankment would also need to be carefully planned and designed for later raising, which could mean working over water, and a completely new access bridge to the outlet tower.

RCC construction offers much simpler options for construction in two stages, especially if construction plant access roads are planned for both stages from the outset.

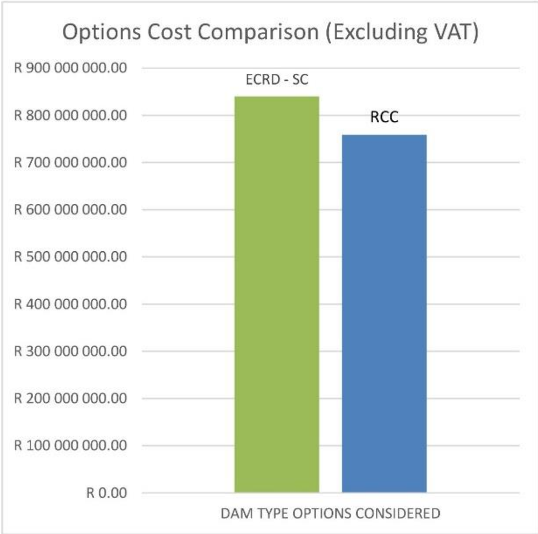
Given that an RCC dam would likely have a combined spillway and outlet works at the river centreline, there would be less alterations and additional works to undertake during the raising as the downstream spillway chute and stilling basin would not need any changes, and there would be no flood risks at all.

However, attention would have to be paid to ensure that the joint interface between the original and raised dam wall does not delaminate due to differential shrinkage between old and new concrete layers, and open up seepage paths.

In conclusion, if it were to be decided by DWS that the Ntabelanga Dam should be configured such that it can be developed in two stages (i.e. 0.15, and 1.5 MAR or larger), then it is a high probability that RCC would be the only practical and economic solution.

To determine whether this might affect decision-making, costing of the minimum Ntabelanga Dam scenario has been undertaken for both ECRD – right hand side channel spillway, and RCC option.

The chart in Figure 4-2 below shows the cost comparison of the Earth Core Rock fill and Roller Compacted Concrete options, including all of the same other ancillary works that would be required even with the smaller dam.



	111%	100%
Cost Excluding VAT R'millions	840	758
Total rock excavation used in embankment	438 000	147 856
Total rock excavation to spoil	303 000	184 285
Total all materials to spoil	372 300	0

Figure 4-2: Minimum Ntabelanga Dam Options Cost Comparison

In this case the RCC option is significantly less expensive than the ECRD option, and has better incorporation of excavated material in the works, without major disposal to spoil, which could have additional environmental impacts. The side channel spillway configuration makes the ECRD option less viable in terms of construction in stages.

4.2.2 Speed of implementation to first water delivery

One of the advantages of an RCC solution over the embankment dam is faster speed of construction and, provided that the outlet works can be completed in time, delivery of water could commence well before the main structure of the dam is completed.

4.2.3 Ability of DWS Infrastructure Branch to undertake detailed design in-house

This project falls under a Strategic Integrated Projects (SIP) category and is therefore a very high priority project. Should there be a need to undertake a procurement process to appoint a PSP to undertake the detailed design of the works, this fast-track implementation programme would be significantly delayed. The solution being considered is for DWS's Infrastructure Branch to undertake the design in-house. The Infrastructure Branch has good experience of designing RCC structures but has limited experience of rock-fill dams.

4.2.4 Ability of DWS construction unit division to undertake construction in-house

For similar reasons to those given above, DWS are also considering construction of the works using their in-house construction division, rather than further extending the implementation period by having to undertake a prolonged contractor procurement process. Once again, the in-house expertise of RCC construction is available whereas there is limited recent in-house experience of construction of rock-fill embankments.

4.2.5 *Simplified infrastructure layout and access*

The optimum ECRD dam solution would have a right-bank side channel spillway which discharges back into the stilling basin below the dam wall. Given that the outlet works and water treatment works would also be sited on the right bank of the river below the dam, the outlet works and access road would need to cross over the spillway discharge chute. This would limit the space available for locating the hydropower plant near to the dam wall, and more complicated access would be required across the spillway chute.

The RCC dam would have a central discharge spillway, and the outlet works on the right bank of the river, leaving the right flank area downstream of the dam clear for the efficient location and development of a hydropower plant, water transfer pipelines to the water treatment works, and access roads to these works and to the dam itself.

4.2.6 *Low maintenance inputs*

Generally, an all-concrete solution such as an RCC dam, may have lower maintenance requirements than an embankment dam, given the need to regularly monitor and maintain embankment slopes, the more complex outlet tower, and its access bridge. A side channel spillway would also be mainly unlined, and regular inspection and maintenance of the rock channel surfaces may be needed.

4.2.7 *Less risk when dealing with water during construction*

An RCC dam is more resilient to overtopping during construction than an earth core rock-fill dam, should unexpected flood events happen during construction, and temporary works fail to contain such floods. For example, both Ludeke and Dikgathlong dams mentioned above had unforeseen, and previously unrecorded flood events which damaged the works under construction, and delayed the completion of the works, with consequential increased costs.

4.2.8 *Environmental impacts*

An ECRD will require more rock excavation than the RCC dam option, and would source such rock from the right bank side channel spillway, whereas the rock for concrete for the RCC dam would be sourced from a quarry on the right bank, which quarry would be inundated when the dam fills.

The ECRD option also requires clay and filter sand sources, whereas the RCC dam requires sources of sand, all of which would be obtained from within the river basin above the dam wall. Once again, whilst the temporary environmental impacts of the abstraction and hauling of these materials would likely be higher for the ECRD option, it can be argued that the RCC option would have different temporary impact due to the need to transport other materials such as cement, fly ash and other additives from sources outside of the local area, via the national road network.

4.3 Conclusion on Dam Type Selection

Taking the above decision-making factors into consideration, it is concluded that the preferred dam type is the RCC solution.

This would provide for a simplified operational layout, and better aesthetics and less environmental impact than an ECRD dam with a side channel spillway, and would offer the better opportunity for implementation in a shorter time period.

The fact that the DWS Infrastructure Branch would be responsible for the implementation of the project in-house to reduce the implementation time, and that they have more experience with RCC technology than rock-fill, would further justify the preference of RCC as the dam type to be implemented.

Therefore the dam and ancillary works that will be further described in the following sections are based on the RCC solution.

The draft Scope of Work for detailed design of the works allows for a further review of the dam type and this decision will therefore be re-evaluated in the detailed design stage in the light of more detailed analysis based on additional geotechnical information.

A general arrangement of the RCC dam solution is given on Figure 4-3.

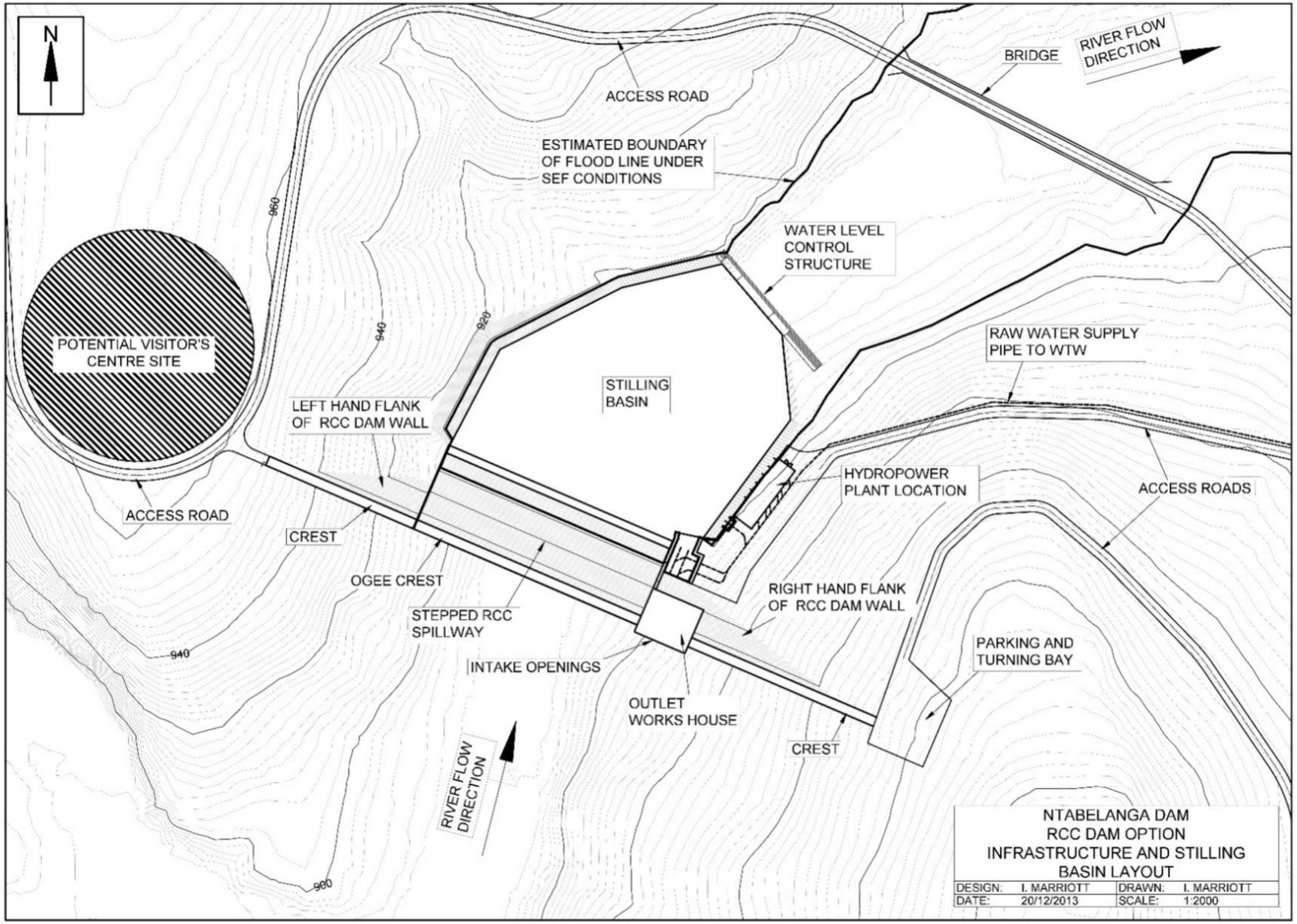


Figure 4-3: General Arrangement of the RCC Dam Option

5. FEASIBILITY DESIGN NTABELANGA DAM, SPILLWAYAND RELATED WORKS

5.1 Dam wall and spillway

As described in the preceding sections, an RCC gravity dam is recommended, with an ogee spillway with stepped downstream face, with a slope of 1 to 0.70, or step dimensions of 1 200 mm high by 840 mm wide.

During the undertaking of the feasibility design of this dam, the design process and relevant associated reports were reviewed by specialists on the Review Panel. The main review expert also visited the site with the study team, and fine tuning of dam centreline alignments and other details were agreed. This involved adjusting the axis of the dam wall to be squarer to the contours on both flanks, and this effectively moved the centre point of the dam very slightly upstream. This also has the advantage of reducing the maximum dam wall height by 1.7 m and the crest length by some 33 m.

Figures 4-3, 5-1 and 5-2 show the proposed layout plan, typical wall and spillway cross-sections, and longitudinal cross-sections for the recommended dam type and spillway.

The proposed Ntabelanga Dam has the following characteristics:

Full Supply Level (FSL):	947.3 m.a.s.l.
Non-Overspill Crest Level (NOCL) – right flank	953.9 m.a.s.l.
Minimum bed level in river at dam:	886.7 m.a.s.l.
Crest width:	6 m
Minimum operating level (MOL):	918.00 m.a.s.l.
Emergency drawdown minimum outlet level:	907.00 m.a.s.l.
Maximum dam wall height to NOC:	66.1 m
Wall crest length (incl spillway):	407 m
Spillway crest length:	150 m
Gross stored volume at FSL:	490 million m^3
Mean Annual Runoff at dam:	415 million m^3
Storage below MOL (V_{50} sedimentation):	37 million m^3
Surface area of lake behind dam:	31.5 km^2
Backwater reach upstream of dam:	15.5 km

The dam wall height, impoundment volume, and downstream risk factors for the Ntabelanga Dam put this structure into a Category 3 dam under Gazetted Dam Safety Guidelines.

As discussed in Appendix A, and as reviewed and accepted by the DWS Hydrological Services, the flood criteria for design of this dam are as follows:

1 in 200 year return period Design Flood:	2 500 m^3/sec
Safety Evaluation Flood (SEF):	5 530 m^3/sec

The dam capacity fully meets the potable and irrigation water requirements as well as providing regulated flow releases in the river below the dam to meet the EWR requirements, to generate an average of 1.6 MW of hydropower at the dam wall, and to assure sufficient river flow downstream for the Lalini Dam and Hydropower Scheme.

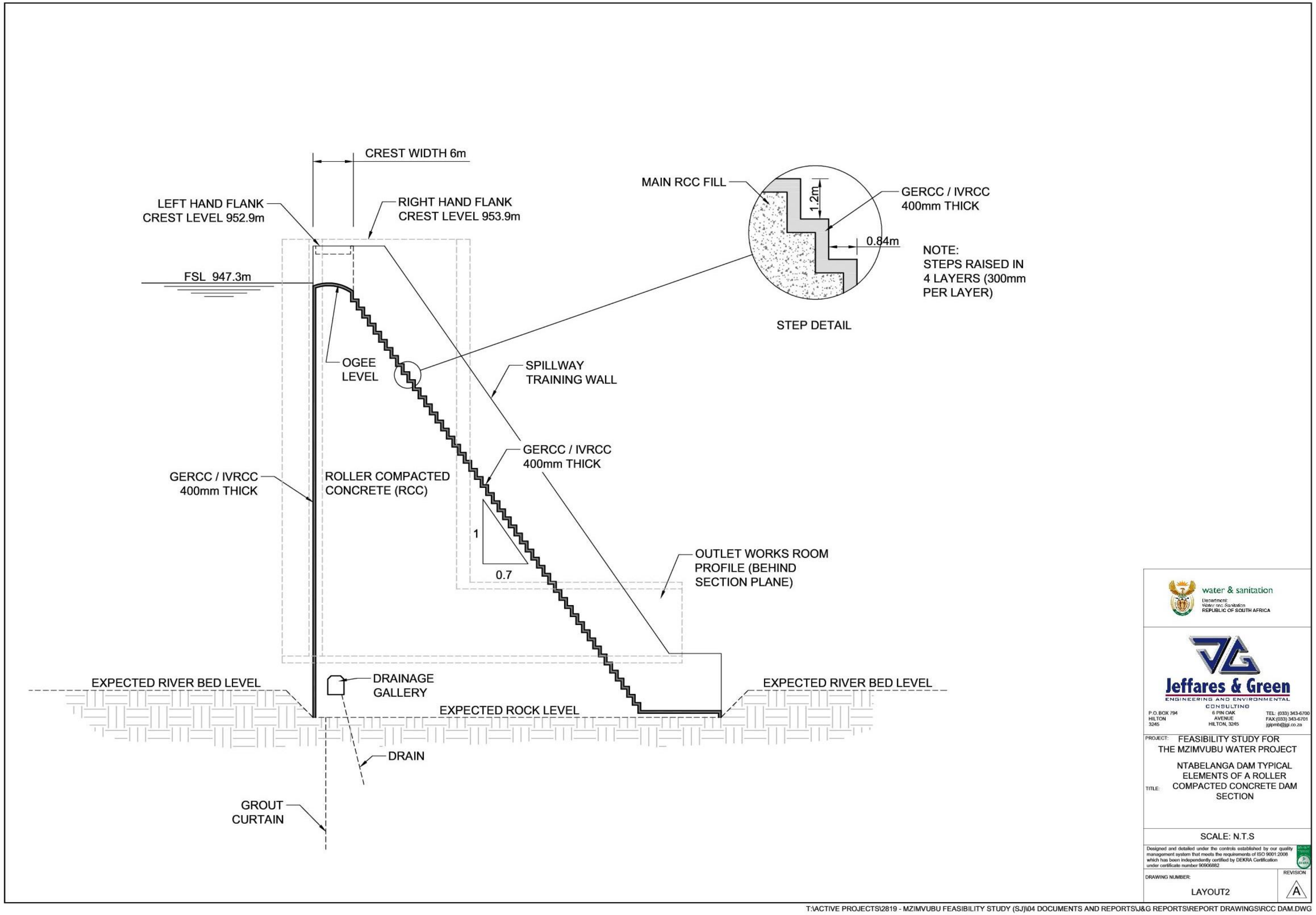


Figure 5-1: RCC Dam Wall and Spillway Typical Cross Section

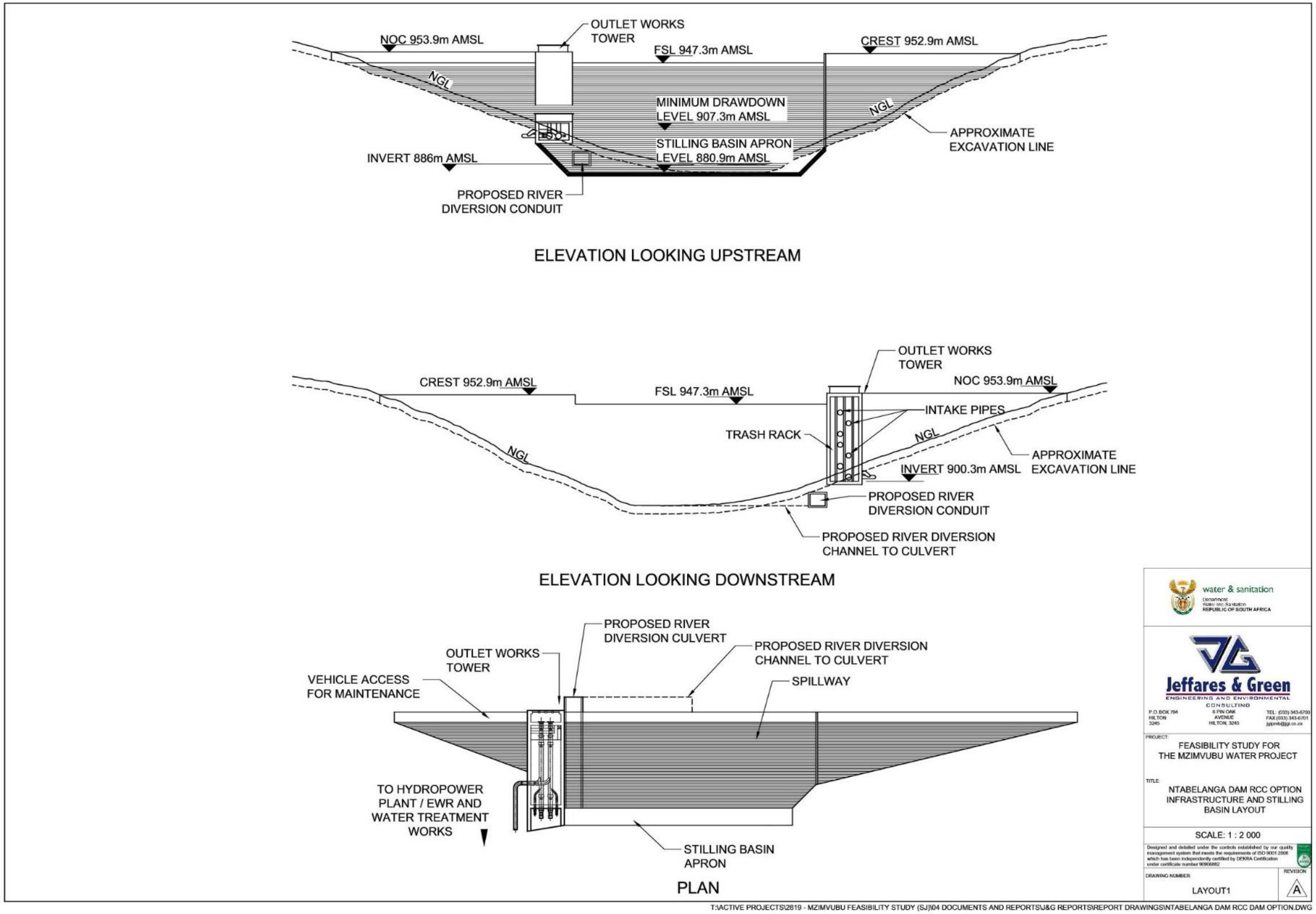


Figure 5-2: RCC Dam Embankment Plan and Longitudinal Elevations

The geotechnical investigations have indicated that the founding conditions of both dam wall and stilling basin are in competent dolerite, which will have a low erodibility. It was thus agreed with the review expert that the spillway width could be reduced to 150 m and the stilling basin accordingly reduced in width, length and depth, thus saving costs. It was also not considered necessary to install a flip bucket at the lower end of the stepped spillway chute.

Given that the dam wall is to be entirely of RCC construction, and is built on competent rock foundations, the wall structure can therefore tolerate some overtopping under both design flood and SEF conditions. It was therefore suggested to reduce cost that it would not be necessary to increase the non-overspill crest to the full SEF level of the dam on the left flank, and this would result in approximately a 1 m overtopping depth under the extremely rare SEF event, and some wave over-splash during a design flood event. For this reason, the NOC is set to 6.6 m above spillway crest on the right flank to avoid overtopping of the outlet works and 5.6 m on the left flank. This decision can be revisited during the detailed design stage if it raises concerns.

The hydraulic analysis was again undertaken using the normal ogee spillway crest formula described in previous sections, and using a spillway crest width of 150 m, which, under the 1 in 200 year return period 2 500 m^3/s design discharge, results in a flow depth over the crest of 3.9 m. This limits the unit discharge rate to an acceptable 16.7 m^3/s/m.

The resulting 2.7 m freeboard under design flood conditions is adequate to deal with wind run-up, weaves, surges and seiches.

The depth of flow over the 150 m spillway during the SEF event, which has a flow rate of 5 530 m^3/s, is 6.6 m, and there would therefore be a 1 m overtopping of the left flank during this event.

The SEF event flood produces a unit discharge rate over the spillway crest of 36.8 m^3/s/m, which is at the upper end of that recommended for stepped spillways to reduce nap separation and cavitation action.

The spillway, chute and stilling basin arrangement must be investigated in more detail and optimised during the detailed design stage, which should include both Computational Fluid Dynamics (CFD) and physical modelling.

CFD is optional, given that it requires very intense computational power and can be time-consuming, but physical modelling is considered essential. Research is currently being undertaken at the University of Stellenbosch regarding the impacts on discharge efficiency of high flows over ogee-crested stepped spillways, and it is evident that much attention must be paid to ensuring that the nap adheres to the ogee crest and does not separate. Physical modelling will therefore inform the design and, if necessary, a longer spillway crest length might result.

5.2 Outlet Works

As has been described above, the dam wall and spillway will be constructed using RCC, and it is proposed that the draw-off and outlet works be housed in a reinforced concrete structure running through the dam wall as is shown on the layout drawings.

The drawoff and outlet works will have a multi-purpose which functions are described in the following sub-sections. The dam outlet arrangements will be subject to review during the detailed design stage and may therefore change from this feasibility level design approach.

5.2.1 EWR Releases

The Reserve Determination Report No. P WMA 12/T30/00/5212/7 determines the Environmental Water Release (EWR) requirements to be released downstream of the dam. This is based upon running WRYM hydrological simulations and takes into account the expected spills during the same period of simulation.

The recommended total releases are those required to maintain an intermediate ecological Class C of 87.249 million m^3 per annum, which equates to an average of some 7.27 million m^3 per month, or 2.8 m^3/s.

The EWR is actually required to be released according to a seasonal pattern and this also depends on whether the river is in a status of flood or drought. EWR release rules are proposed in the Reserve Determination Report, and release criteria are based upon preceding inflows. This report also recommends that in order to be able to select the level from which to release water of best quality and temperature from the dam, the outlet works should have seven outlets spaced at between 6 and 7 m apart, with the top outlet located some 7 m below the FSL. This has been incorporated into the feasibility design of the outlet works as shown on Figure 5-3.

The monthly model simulation results are shown in Appendix E, and a statistical analysis has been undertaken to determine the probability of various release volumes that would likely be required. Figure 5-4 shows this in chart form. As can be seen, the EWR release requirement varies from almost zero to 23 m^3/s. The 2.8 m^3/s average figure given above is actually required only 25% of the time, with lower figures required 75% of the time, and flow rates above 16 m^3/s are required less than 2.5 % of the time.

Given that water released for EWR can also be passed through a hydropower generation turbine before release, it was decided to consider both EWR and hydropower releases together before making a decision on outlet conduit capacity.

5.2.2 Hydropower Generation

It is proposed that the Ntabelanga and Lalini Dams be operated conjunctively to generate hydropower. During the more detailed investigations of the Lalini Dam and hydropower scheme (see Report Nos. P WMA 12/T30/00/5212/18 and P WMA 12/T30/00/5212/19) a hydropower simulation model was developed and run which, in addition to the main Lalini hydropower plant, included mini-hydropower plants located at each of the two dams themselves which utilized EWR releases as well as flows that would have otherwise passed over the spillway of each dam.

Operating rules were set to ensure that minimum and maximum allowable EWR releases were maintained throughout.

The results of this modelling indicated that a hydropower plant of some 5 MW should be installed at Ntabelanga Dam and that this would be operated in accordance with the agreed EWR release rules.

As a result, average power outputs varied monthly, and in accordance with the pattern shown in Figure 5-5.

As can be seen in Figure 5-5, whilst the plant can produce up to 5 MW at peak (i.e. when sufficient flows are available), on average the wet season monthly output would be some 3.75 MW, and the dry season average monthly output would be some 1.4 MW (with some drier months operating at only 0.75 MW). Thus the hydroelectric plant (HEP) needs to be configured to be able to operate within this full range of outputs.

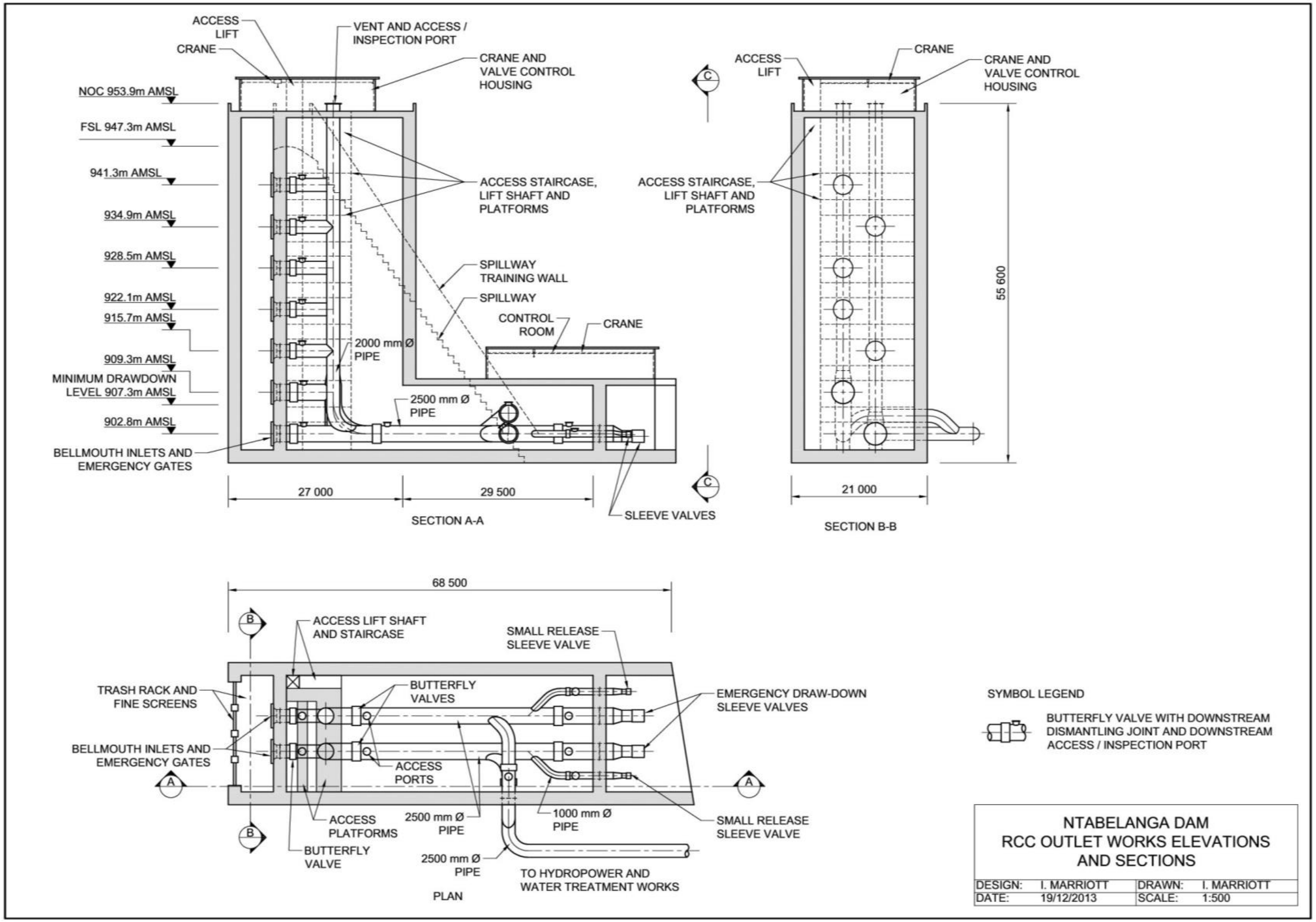


Figure 5-3: Outlet Works Elevations and Sections

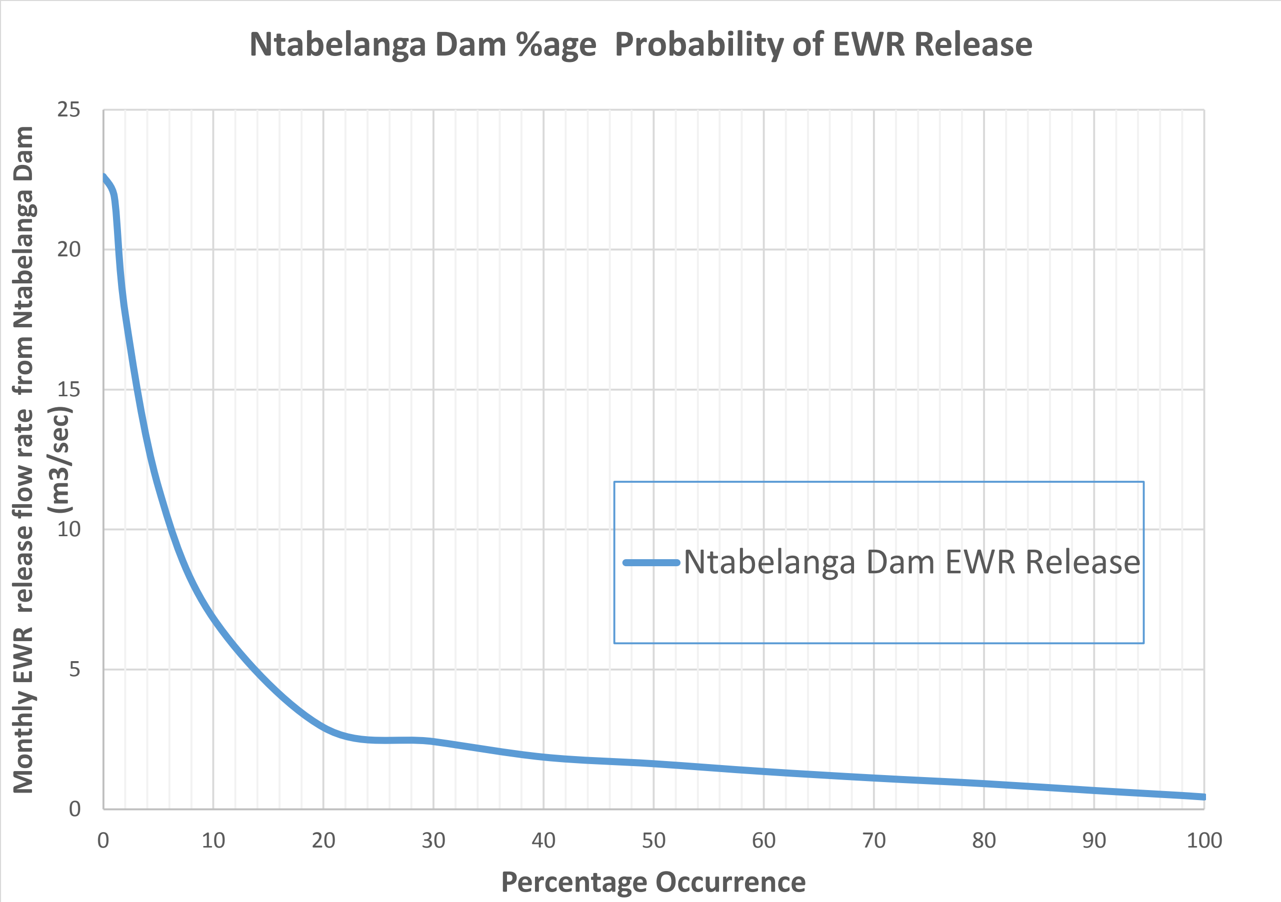


Figure 5-4: Probability of Required EWR Release Rates

MONTHLY HYDROPOWER REQUIREMENTS

Month	Operating MW	Avg HP Generated (MW)	Avg Assurance Baseload (KWh)
Oct	5.0	1.63	1 211 096
Nov	5.0	1.88	1 351 209
Dec	5.0	1.82	1 355 969
Jan	5.0	2.21	1 645 106
Feb	5.0	3.76	2 552 154
Mar	5.0	3.44	2 556 484
Apr	5.0	2.24	1 610 117
May	5.0	1.55	1 154 458
Jun	5.0	1.56	1 121 201
Jul	5.0	1.40	1 039 898
Aug	5.0	1.53	1 141 075
Sep	5.0	1.52	1 094 138
TOTAL	**5.0**	**24.5**	**17 832 905**
Average Power		**2.0**	**1 486 075**

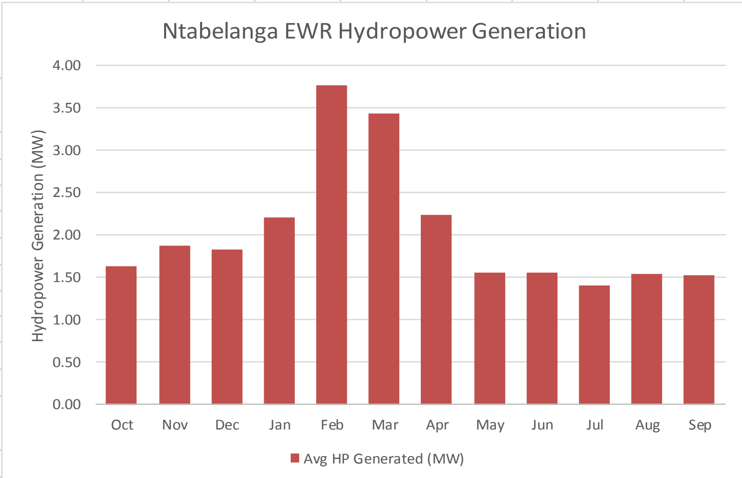


Figure 5-5: Average Monthly Power Outputs from Hydropower Generation at Ntabelanga Dam

The same analyses also produced a historical simulation of water levels in the Ntabelanga Dam which is shown in Figure 5-6.

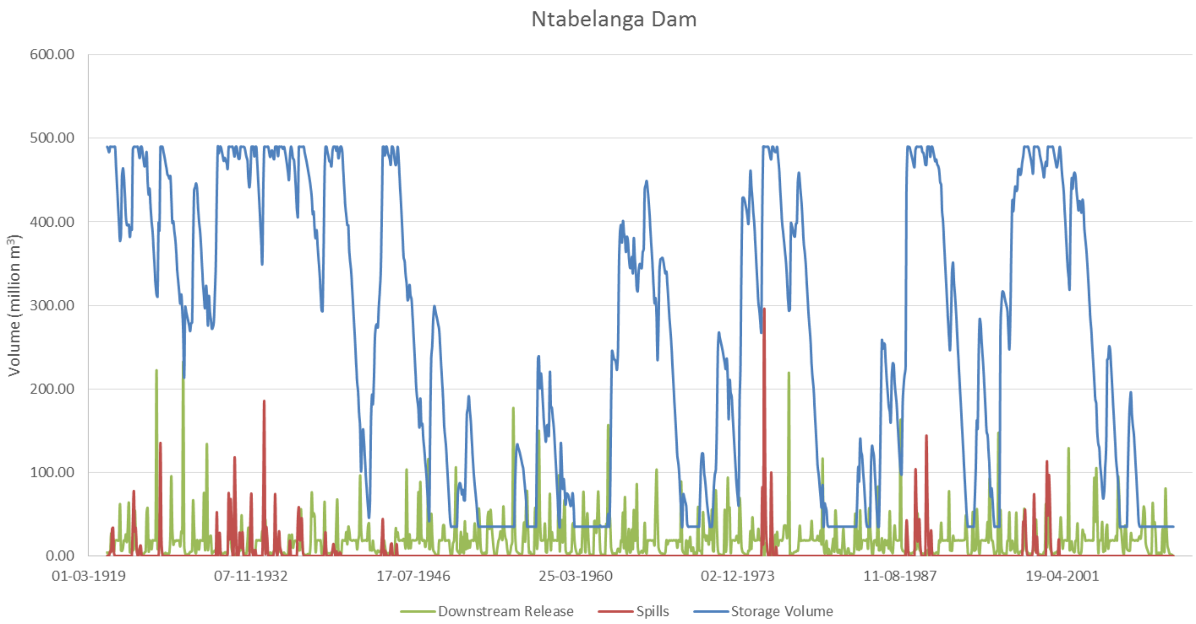


Figure 5-6: Water Level Trajectory for Conjunctive Use Simulation at Ntabelanga Dam

Figure 5-6 shows that the full capacity range of the dam will be utilized on a regular basis throughout, with only moderate incidence of spillage, which justifies the sizing of the dam for this conjunctive purpose. It should be noted that the operating rules of the dam have been set to ensure that EWR is always released as well as meeting the potable and irrigation water requirements with the required assurance of supply, as determined under this feasibility study.

The outlet works pipework configuration allows for large and small release discharges directly into the stilling basin. The off-take pipework to the Ntabelanga mini-hydropower plant and WTW is sized for the maximum hydropower output, WTW, and raw water requirements of 16 m^3/s, 1.2 m^3/s and 1.2 m^3/s respectively. In this case, a 2.5 m diameter pipe was deemed to be sufficient.

5.2.3 Pipeline to Water Treatment Works

A further function of the offtake and outlet works is to deliver water to the water treatment works. As described in the Bulk Water Distribution Infrastructure Report No. P WMA 12/T30/00/5212/13, the summer peak raw water demand for Domestic Requirements was 101 515 m^3/day + 10% WTW losses = 111 667 m^3/day, which is 1.3 m^3/s.

The water treatment works inlet is located approximately 1.2 km from the dam outlet works with an inlet level of 900.00 m.a.s.l. Given the recommended bottom operating level of the dam of 918.00 m.a.s.l., a minimum gravity head of 18 m is available to transfer water from the dam to the WTW.

Limiting the flow velocity in this transfer pipeline to less than 2 m/s would require a 1 000 mm diameter pipeline, which would have less than 10 m total head loss under this flow condition. However, this pipeline will also transfer water to the raw water pump station for the irrigation scheme, and therefore this transfer pipeline is sized at 1 600 mm diameter. The minimum recommended velocity for self-cleansing of raw water pipelines is 0.6 m/sec. At this velocity, a 1 600 mm diameter pipeline would convey some 104 000 m^3/day. Therefore, if the first stage development of the WTW and the irrigation scheme combined were to have less than this capacity, consideration should be given to installing two smaller raw water pipelines, and using one initially to maintain self-cleansing velocities above this minimum.

As can be seen on Figure 5-3, the proposal is to run twin 2.5 m diameter steel conduits through the outlet works. These will be supplied from six x 2 m diameter bellmouth draw-offs located on the front face of the outlet works, and positioned at various depths to allow water to be drawn at the best level for water quality purposes. In this respect, turbidity and suspended solids will be of importance for the treatment process, and temperature is also important for the EWR aspects.

In the unusual case of single outlet operation, each single 2 m diameter bellmouth draw-off and 2 m diameter conduit can convey flows up to the design peak flow of 16 m^3/s for hydropower or EWR purposes plus 2.36 m^3/s for the peak water treatment works and irrigation scheme output, at a velocity of 5.8 m/s, which is acceptable. In the very rare absolute peak flow periods, this velocity would still not exceed 8 m/s.

Under maximum hydropower production, peak water treatment works operation, and raw water pumping flow conditions, the head loss to the hydropower plant would be approximately 5 m.

The configuration shown in Figure 5-3 allows either or both conduits to be used at any time, to supply both water treatment works and hydropower plant/EWR outlet simultaneously.

5.2.4 Pipeline to Irrigated Areas

As described in the Bulk Water Distribution Infrastructure Report No. P WMA 12/T30/00/5212/13, the lowest unit cost solution is to abstract raw water at the Ntabelanga Dam and to pump a distance of 16.4 km to an intermediate storage reservoir before distributing onwards to the farming units located in the Tsolo area.

As described in the section above, an additional allowance for the 1.1 m^3/s peak flow rate required for irrigation has been allowed for in the Ntabelanga Dam outlet works design.

This arrangement has the advantage of centralising all operations at the Ntabelanga Dam site rather than having to operate and maintain a separate river intake works, settling channels or basins, and pumping station downstream of the dam.

As is shown in the sections below, the raw water pumping station required for the irrigation scheme has been located at the water treatment works site to simplify raw water inlet pipework arrangements, and to allow the WTW, treated water pumping and raw water pumping operations to be managed by the same operations staff. This will also simplify and centralise power transformers and supply lines.

Figure 5-7 shows a proposed arrangement for the irrigation raw water pump station, which pumping configuration is as described in the Bulk Water Distribution Infrastructure Report No. P WMA 12/T30/00/5212/13.

5.2.5 Emergency Drawdown Facilities

It is a normal requirement to be able to rapidly drawdown the dam water level in the case of an emergency. This requires that the dam water level be reduced from FSL to one third of its full water depth in 90 days.

Given that only a very small volume of water is stored in the first third of the dam's water depth, this means that some 480 million m^3 of water would need to be released in 90 days. This is an average flow of 62 m^3/s, with a peak flow of approximately 72 m^3/s. This is taken into consideration for the outlet works feasibility design.

Some dams have completely separate emergency drawdown systems, and given that these are very rarely used, can be a cause of problems if they silt up or are not maintained properly.

Under an emergency rapid drawdown situation, it is proposed that all seven outlet bell-mouths would be opened as well as the downstream discharge valves on both of the outlet conduits.

Under such conditions the required peak drawdown rate of 72 m^3/s and average of 62 m^3/s will be achieved, and the maximum velocity in each conduit would be 8.0 m/s, which is acceptable for limited periods and infrequent occurrences.

In addition to the upstream emergency gates and butterfly valves on each of the seven offtakes upstream, there would be sleeve valves at the outlet of each of the rapid drawdown and small release conduits. Given the velocities involved, these sleeve valves are suitable for flow control and tight closure.

It should be noted that total head loss in the system will increase under this rare emergency drawdown period. This will not affect the water treatment works and raw water pumping station output but the hydropower plant output will be down on its normal performance for equivalent dam water levels.

It is again recommended that such a system be modelled and optimised using physical modelling or possibly computational fluid dynamics modelling (CFD) during the detailed design stage, to ensure that surge, cavitation and vibration effects are minimised or avoided altogether.

5.3 River Diversion

A 1 in 5 year flood event (some 500 m^3/s) was used to design temporary diversion works for the RCC dam, since a RCC dam can accommodate minor overtopping during construction. Hence, it requires a lesser safety margin in terms of floods.

The first stage river diversion would require protecting the right flank by means of a cofferdam with diversion of the river flow to the left flank. The cofferdam is required to enable and protect the excavation for the outlet works, right flank non-overspill concrete section and a portion of the spillway to accommodate construction of the second stage diversion conduit and low notch.

For this RCC dam, it is proposed that a second stage river diversion conduit (opening) for low flows (up to 150 m^3/s) and an open channel (low notch) be constructed in the spillway section. An opening measuring approximately 4 m x 3 m needs to be constructed on the right flank of the river channel below the right hand section of the spillway, and adjacent to the outlet works structure. This would have a maximum flow capacity of 160 m^3/s and would limit flood water level rise upstream to just over 10 m. This will comprise an opening in the concrete spillway section at the appropriate level, which can be closed by steel gates/stop logs and filled with concrete when the dam starts impounding. An open channel (low notch), at a higher elevation than the conduit, is formed in the spillway section to accommodate the remainder of the flood within the second stage cofferdam.

Together with the above measures, the timing of the second stage diversion is essential so that it coincides with the dry season (low river flow). The excavation within this cofferdam needs to be completed as quickly as possible and the first concrete placed. This will protect the foundations and limit damage if the design flood for the diversion is exceeded. The risk of these proposed measures is regarded as acceptable for the construction of a RCC dam when compared to the alternative of providing much larger diversion works.

The eventual river diversion works will depend upon the design proposed by the design team and the construction contractor's proposed approach and methodology, which needs to be approved by the Approved Professional Person.

The primary issue is to ensure that such works are properly designed and do not form potential seepage paths in the longer term. The design of these works must include a method to seal and plug these works securely once it is time to start impounding water in the dam. This has been done successfully at similar DWS dams, such as Nandoni and De Hoop.

For the 500 m^3/s design flood, the maximum rise in water level should be limited to some 8 m, which is considered acceptable and can be contained by the second stage diversion cofferdam of about 10 m high.

In the context of the feasibility level analysis, the cost of the proposed upstream and downstream cofferdams and flood diversion conduit has been allowed for in the cost estimate and economic analysis.

5.4 Dam Foundations

The foundation levels for this RCC dam type are based upon borehole core log descriptions and seismic velocity profiles. *Van den Berg and Parrock (2009)* recommend the following foundation criteria for dams exceeding 60 m in height:

Table 5-1: Recommended Foundation Design Criteria for RCC Dams

Foundation Design Criteria						
E_{mod} (GPa)	**RMR**	**Weathering**	**UCS**	**RQD**	**Joint Spacing**	**Joint Condition**
>4.5GPa	>40	Medium to Slightly Weathered	>20MPa	>30%	>300mm	Rough, Unaltered

E_{mod}: Elastic Modulus
RMR: Rock Mass Rating
UCS: Uniaxial Compressive Strength
RQD: Rock Quality Designation

The longitudinal section in Figure 5-8 shows the recommended foundation excavation profile, which is based upon the results of the rotary core drilling and seismic refraction survey undertaken during this Feasibility Study.

This foundation profile targets the founding on medium hard to hard rock, complying with the parameters recommended in Table 5-1 as well as the 2 000 m/s seismic velocity profile.

This places the foundation in an intermediate to generally hard excavation category and it is likely that some blasting will be necessary to achieve excavation to good quality foundation rock.

However, blasting must be minimised so as to avoid excessive blast fracturing, which compromises the integrity of the foundation rock. *Van Schalkwyk et al (2009)* recommend stopping bulk blasting about 1 m above the expected founding level and proceeding below this with controlled blasting or powerful excavating equipment.

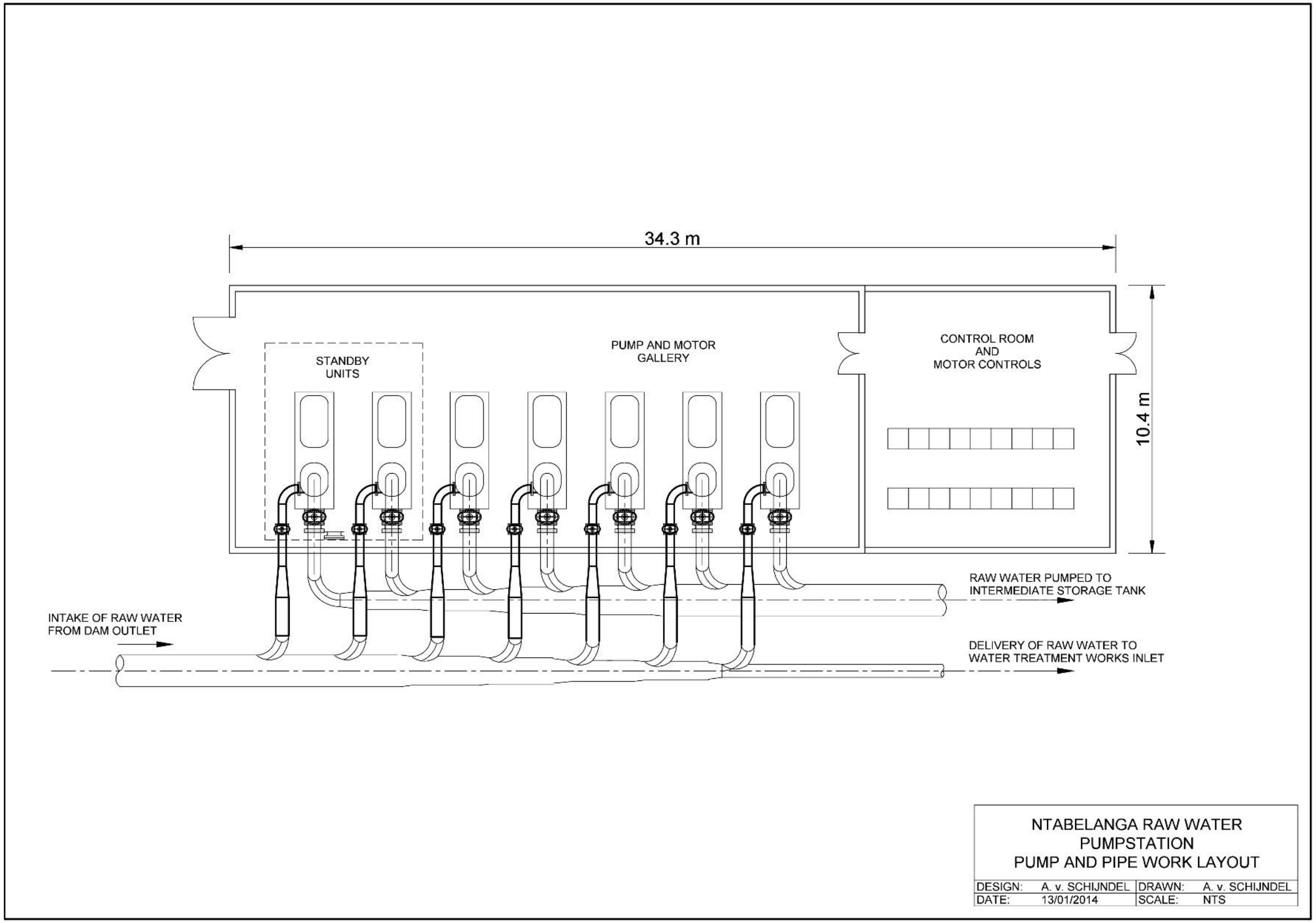


Figure 5-7: Layout of Raw Water Pump Station to Supply Irrigation Scheme

The longitudinal profile indicates an excavation depth of about 9 m on the upper left flank decreasing to about 7.5 m on the mid to lower left flank, about 4 m through the river section to again about 6.4 m on the lower right flank, decreasing to less than 1 m on the mid to upper right flank.

It is recommended that the profile is amended during the detailed design stage as more drilling information becomes available during the detailed design geotechnical investigations.

This further investigation should be planned to check for faults, fractures and lineaments below the dam footprint, although it is not expected that such problems will be identified.

Furthermore, all foundation excavations must be continuously monitored, verified, and the final excavation mapped by an experienced geotechnical professional during construction.

A budget has been allowed in the cost estimates for drilling, grouting and test drilling programme, covering the upstream heel areas of the dam foundation footprint, the outlet works, the spillway, and the temporary river diversion works conduit. Lugeon testing during the core drilling undertaken to date showed very low or no grout uptakes, and therefore only limited grouting is expected to be required.

5.5 Dam Stability Analysis

This has already been discussed above in Section 3, but is summarized herein. The following information and assumptions were used in undertaking the analysis:

- Ntabelanga Dam wall will have a maximum height of 67 m from the river bed level and a total crest length of 400 m.
- Flood will be discharged by means of un-controlled Ogee stepped spillway.
- Concrete density of 2 400 kg/m^3,
- Concrete grade C15/53 would be used mainly for the RCC;
- [3]Solid dolerite founding condition with minimum cohesion of 0.3 MPa and minimum angle of friction of 35°;
- The horizontal component of peak ground acceleration is 0.15 g; and
- The vertical component of peak ground acceleration is 0.08 g.

The loading conditions adopted are shown in Table 5-2.

Table 5-2: Loading Conditions

Type	Case	FSL	RDF	SEF	Silt (S)	Tail water(TW)	Drained (D)	Undrained (UD)	Seismic (SM)
Normal	1	√			√		√		
	2		√		√	√	√		
Abnormal	3		√		√	√		√	
	4			√	√	√	√		
	5	√			√	√	√		√
Extreme	6		√		√	√	√		√
	7			√	√	√		√	

Table 5-3 presents the results obtained from the various load cases in Table 3-11. The analysis results are compared with the allowable factors of safety and maximum stresses according to various international guidelines.

[3] *Literature on rock mass properties state cohesion can be in the range of 0.3 to 30 MPa (but this is not a sensitive parameter in this analysis) and an angle of friction up to 55°.*

Table 5-3: Analysis Results and Comparison (1:0.70 d/s Slope)

Type	Case	Tensile Stress (MPa)		Compressive Stress (MPa)		Sliding (residual) Factor of safety (FOS)		Downstream overturning Factor of safety (FOS)	
		R	**A**	**R**	**A**	**R**	**A**	**R**	**A**
Normal	1	+0.19	0.0	-1.2	-3.0	1.5	1.5	1.5	1.5
	2	+0.4	0.0	-1.4	-3.0	1.3	1.4	1.3	1.4
Abnormal	3	+0.61	0.2	-1.4	-4.5	1.1	1.1	1.1	1.2
	4	+0.56	0.2	-1.5	-4.5	1.1	1.1	1.2	1.2
	5	-0.27	0.2	-0.88	-4.5	2.2	1.1	1.7	1.2
Extreme	6	-0.07	0.35	-1.04	-4.5	1.9	1.0	1.5	1.1
	7	+0.77	0.35	-1.5	-4.5	1.0	1.0	1.0	1.1

Legend - ***A** = Allowable* *- = Compression* ***R** = Result* *+ = Tension*

At the detailed design stage, a detailed structural analysis should be performed on the finally selected dam, spillway and outlet works configuration using this and other available engineering methods and best practice, to optimise the dam structure.

5.6 Estimated RCC Dam Construction Materials

5.6.1 Rock Aggregate

Current feasibility design estimates indicate a volume of some 500 000m³ of crushed rock aggregates will be required for a low paste RCC dam construction. Extensive reserves of competent, hard solid dolerite rock occur on the right flank. This was estimated to be more than twice the required rock volume for construction. Boreholes drilled during geological investigations on the right flank upstream of the dam axis indicate hard rock dolerite occurring from depths of around 1m and in some places, dolerite occurs as sporadic outcrop and sub-outcrop.

Core samples retrieved from the boreholes were submitted for petrographic analyses and unconfined compressive strength (UCS) testing. The petrographic analyses indicate a relatively low degree of alteration and insignificant amounts of deleterious alteration products, such as smectite clay minerals. UCS tests on cores from the upper right flank indicate competent, high strength dolerite. The rock is suitable for use as crushed rock aggregate in RCC dam construction, and for reinforced concrete.

Surface mapping identified that the reserves of potentially good quality dolerite in the right flank to the east and south east of the dam are vast and are potentially far in excess of the required quantities for RCC dam construction.

It is expected that all of the required dolerite could be quarried from this right flank upstream of the dam wall, and below the dam full supply level. Thus there should be no permanent environmental impacts, or significant quarry closure requirements.

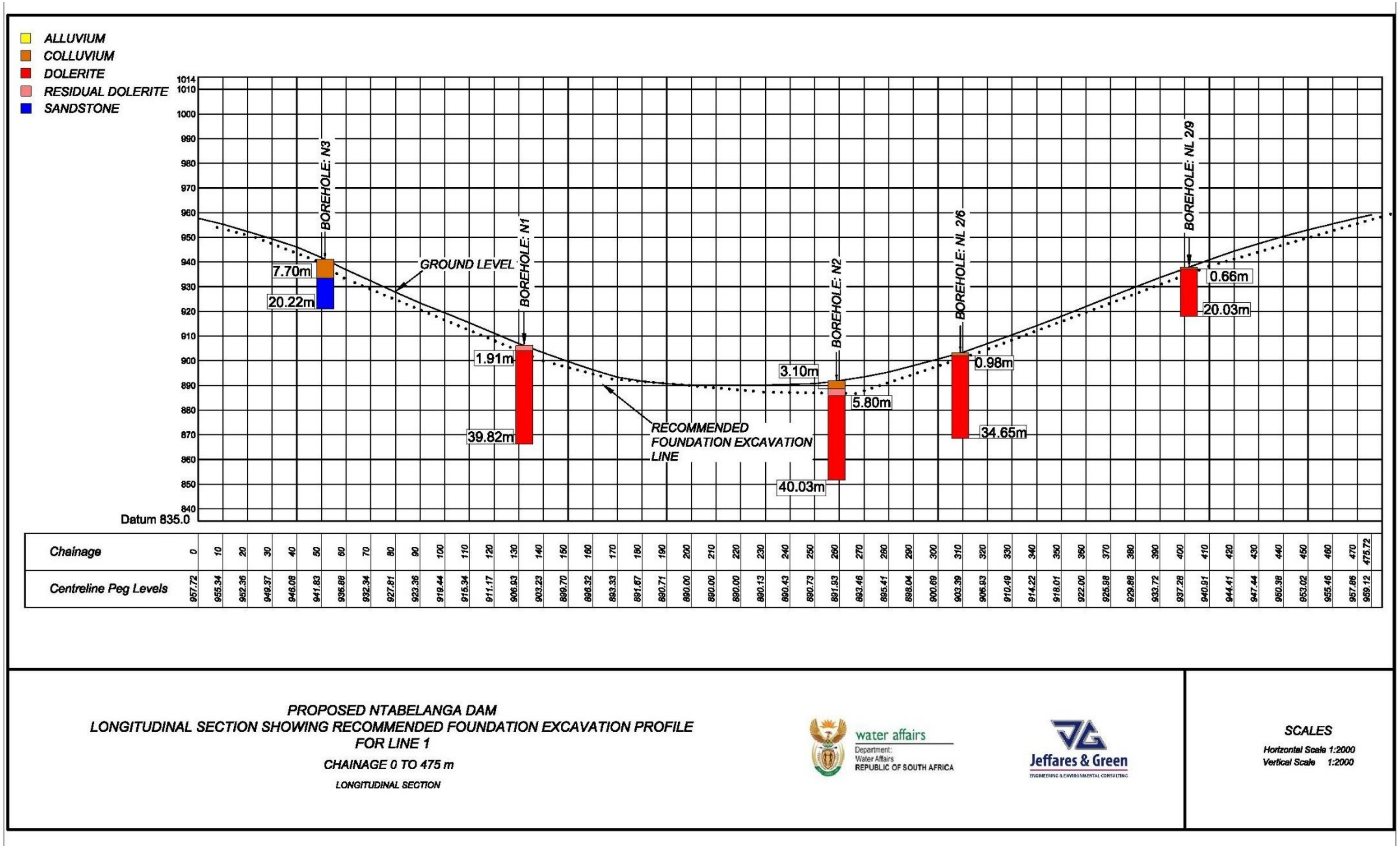


Figure 5-8: Recommended Foundation Profile for Competent Rock Levels

5.6.2 Sand

Sand along a section of the Tsitsa River upstream of the dam was sampled, as indicated by the yellow hatching on Figure B.3 in Appendix B. The Tsitsa River in the project area generally flows in a relatively incised channel with sand deposits confined to the river channel. Over-bank deposits on inside meanders are of a restricted and localised nature.

Therefore sand deposits in the Tsitsa River are relatively narrow and will require selective seasonal exploitation during the dry season. Screening will be required to remove gravel (mudrock fragments), pebbles and boulders. Furthermore, the sand requires blending using crusher sand to achieve the grading required for the concrete mix. Test results available do not indicate the presence of deleterious chemical constituents.

Estimated reserves within the area investigated are approximately 130 000 m³. Considering that this sand will need to blended with crusher sand to provide required grading for RCC construction, the volume of the available sand for construction can be stated in excess of the above value. Furthermore, visually the actual feasibly exploitable reserves in the Tsitsa River, available within the impoundment basin, and within economic haulage distance of the dam, will be far in excess of two times the required volume of some 200 000 m³ of sand for the proposed RCC dam.

5.6.3 Other Concrete Constituents

As a part of the detailed costing of the RCC concrete mix, an analysis was undertaken of the sources of fly-ash, cement, and concrete additives from the South African major suppliers of these materials. These companies included Lafarge, Ash Resources, etc.

All of these materials are readily available albeit with significant transport costs. The costs of these materials as provided by the manufacturers have been taken into account when building up the cost estimates for the project.

This is reported further in the Cost Estimates and Economic Analysis Report.

5.6.4 Recommendations for Further Detailed Geotechnical Investigations

Based upon the results of the feasibility level investigations, founding conditions are suitable for an RCC dam. Additional, detailed investigations considered necessary to bring the level of detail up to that required to undertake the detailed design and tender documentation for the proposed construction of the dam and appurtenant works are described in the Geotechnical Investigations Report.

It is recommended that the detailed rotary core drilling investigation concentrates on infill drilling of the foundation footprint on both dam flanks, spillway components, appurtenant structures and to prove sufficient reserves of rock aggregate for construction.

It is recommended that an inclined borehole be drilled through the dolerite / sandstone contact on the mid left flank and that another inclined borehole is drilled beneath the river section from the left river bank. Provision must also be made for additional drilling on both the upstream and downstream dam foundation footprints.

Following design confirmation of the locations of the appurtenant works such as spillway, intake tower, outlet works, pipelines, hydropower plant, water treatment plant, roads, downstream river bridge and other related infrastructure, drilling and trial pitting will be required to augment the feasibility level investigations in proving suitable founding conditions and to prove adequate reserves of rock aggregate and sand.

5.6.5 Stilling Basin

As is shown on Figure 4-4, water passing over the spillway will be channelled into a stilling basin cut into the existing rock downstream of the dam, to a depth of 5 m below the existing river bed level.

This starts at the base of the spillway where flow into the stilling basin is transitioned over a 20 m reinforced concrete apron to protect the rock at the toe of the spillway from scouring.

The stilling basin width gradually reduces from 150 m to 50 m over a distance of about 200 m, where a bunding and outlet weir controls the stilling basin water level.

The twin emergency drawdown outlet conduits discharge into the same stilling basin from a chamber on the right hand side of the spillway chute side wall.

The hydropower plant and the EWR release valve also discharge flow back into the stilling basin.

It is again recommended that physical and CFD modelling is undertaken to optimise the spillway performance, and the stilling basin shape and depth.

5.7 Outlet Pipelines

Figure 4-4 shows the routing of the raw water outlet pipeline which delivers gravity flow to the Water Treatment Works and Hydropower Plant.

Figure 5-9 shows the overall site layout plan for the dam and its appurtenant works.

The diameter and capacity of these pipelines are as follows:

- Branch line to hydropower plant and Water Treatment Works (WTW): 2.5 m diameter at 18.5 m^3/s
- Branch line to WTW: 1.0 m diameter at 1.2 m^3/s

The proposed pipeline material is welded steel pipe with external protection and epoxy lining, laid in a trench. Cathodic protection will probably be required due to the presence of power lines and stray currents near to the pipeline routes.

The alignments have been selected to follow contours and avoid high points, and to stay outside of the SEF flood line in the river downstream of the dam.

This flood line has been calculated by modelling the downstream river sections using HEC-RAS software.

EWR requirements can still be released in a controlled manner from the bypass and cone valve in the hydropower station, when the hydropower station is off-line.

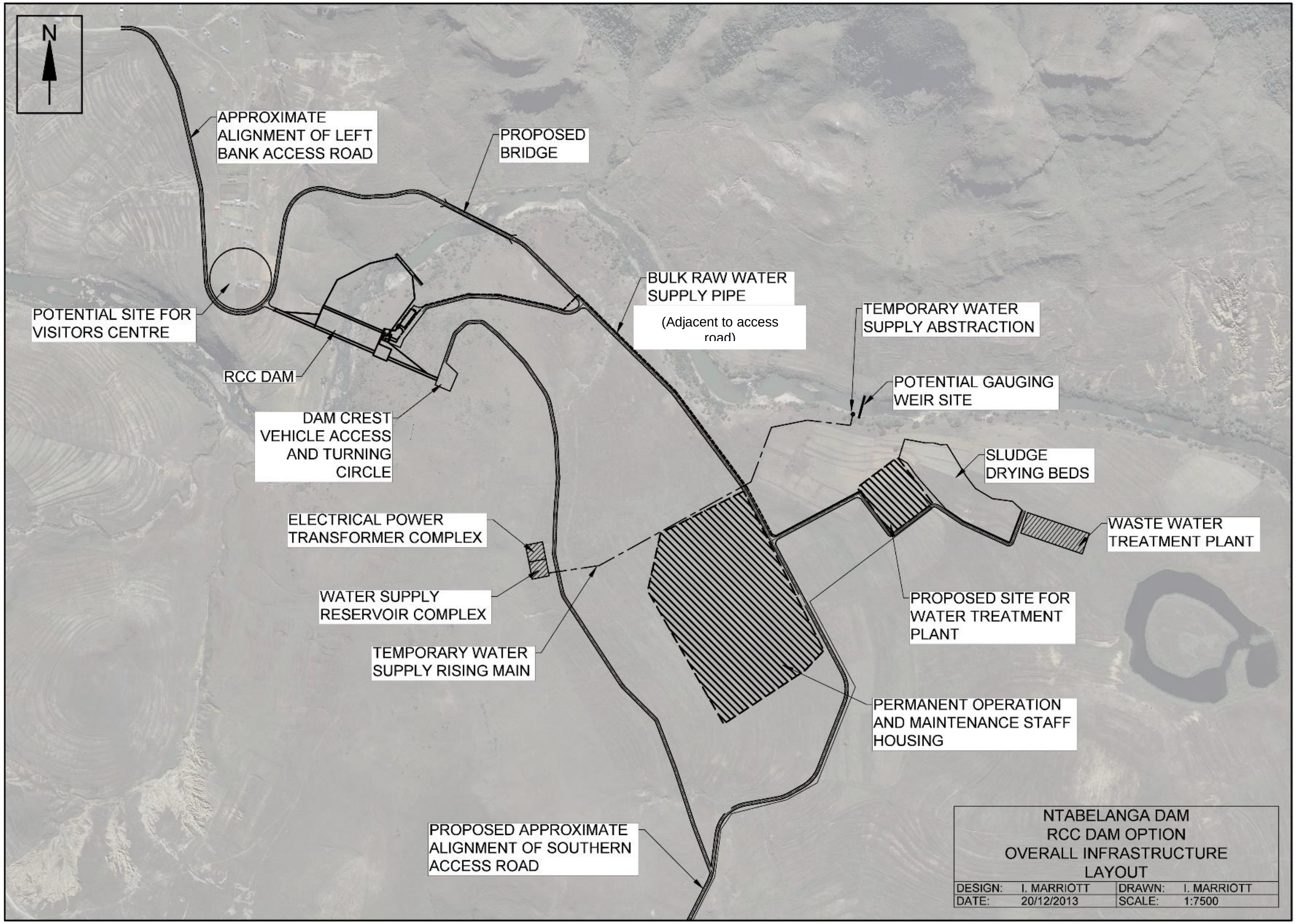


Figure 5-9: Overall layout of the Ntabelanga Dam and associated infrastructure

5.8 Ntabelanga Hydropower Plant

The hydropower plant configuration has been based upon an operating range of between 0.75 MW and 5 MW.

Hydropower plant suppliers were asked to suggest which types of turbines should be used for this application and provided the following options:

The operation of 6 turbines in parallel - 3 pairs with one synchronous and one asynchronous generator. The synchronous generator of each unit is started in the beginning (black start capability, able to run in island mode), the asynchronous unit follows later depending on available flow.

For easy maintenance and stable operation all turbines are of the same size. The speed of asynchronous units will be 750 rpm. The synchronous units speed has to be defined depending on the efficiency expectations (600 rpm or also 750 rpm).

Each turbine set is equipped with a tachometer for speed control, two PT100 sensors (one per bearing) to check bearing temperature and also two vibration sensors (one per bearing).Typical pump-turbine units suggested were:

Pump - Turbine FPT40-700 T1, T3 & T5 with asynchronous generator:

- "Andritz" double suction Pump Turbine
- Type: FPT40-700, with stuffing box sealing
- Casing of cast iron EN-GJL250
- Impellers made from 1.4460 Duplex stainless steel
- Head range 22 - 52 m
- Flow range 1450 litres/sec -2400 litres/sec
- Nominal speed: 750 rpm
- Max. turbine output: 990 kW
- Turbine efficiency max. 84%, actual : 82%
- Power factor: 0.9

Pump - Turbine FPT40-700 T2, T4 & T6 with synchronous generator:

- "Andritz" double suction Pump Turbine Type: FPT40-700, with stuffing box sealing
- Casing of cast iron EN-GJL250
- Impellers made from 1 .4460 Duplex stainless steel
- Head range 22 - 52 m
- Flow range 1200 litres/sec -2300 litres/sec
- Nominal speed: 600 rpm
- Max. turbine output: 825kW
- Turbine efficiency max. 84%, actual: 82%
- Power factor : 0.9

The total number of installed turbine units can produce the following performance:

Table 5-4: Hydropower Plant Output Performance

Scenario	Head (m)	Flow (m^3/s)	Duty	Power (Water kW)	Power (Electrical kW)
Minimum	22	6.0	T1/T2/T3/T4	1 062	956
Average	40	9.0	T1/T2/T3/T4	2 896	2 606
Maximum	45	16.0	T1/T2/T3/T4/T5/T6	5 792	5 212

Figure 5-10 shows a proposed layout of the hydropower turbine house together with the inlet and outlet pipework arrangements.

The building structure would be of similar design to that of the larger pump stations proposed in this study.

This arrangement allows for the whole hydropower plant to be by-passed when not in use, whilst still allowing for release of water for EWR purposes via a sleeve valve outlet.

The inlet system also conveys raw water onward to the water treatment works. If one pair of turbines needs to be taken out of service for maintenance or repair, then the other sets can be run at higher flow rates to maintain power output during that period.

The options for utilisation of the hydropower produced at the Ntabelanga Dam are further discussed in detail in the Cost Estimates and Economic Analysis Report No.P WMA 12/T30/00/5212/15.

5.9 Associated Infrastructure

5.9.1 Roads

The local gravel roads on the north and south banks of the basin (shown in purple on Figure 5-11) are existing low quality access roads to the local settlements, and are normally affected by inclement weather. Some sections of the existing tracks will be inundated by the reservoir water level and will need to be realigned. The main bridge across the river linking the two sides will also be inundated and a new bridge will be constructed just downstream of the dam wall, to restore this main crossing route.

All of these tracks and drainage structures will be upgraded to all-weather gravel roads so that the affected settlements will have improved transport links, which are at a higher elevation and unaffected by the raised water level. These particular upgrades will total some 32 km of road, which will have a servitude width of some 10 m. As all of these improvements will be aligned along existing tracks, or on currently unoccupied areas, this should have only limited or no resettlement or compensation implications.

The two existing gravel access roads shown in yellow and green are currently low quality roads albeit wider than the above existing gravel roads. It is proposed that both these roads are upgraded to secondary surfaced standards, in order to provide all-weather access to heavy vehicles during construction, as well as leaving behind upgraded transport routes to the larger centres of Maclear, Tsolo, and beyond, for those most affected by the project. These two route upgrades will also contribute to improvement of the economy in the area by improving speed and ease of access for business and private travel as well as opening up tourism in the area. Better road quality also reduces wear, tear and maintenance to vehicles using the road.

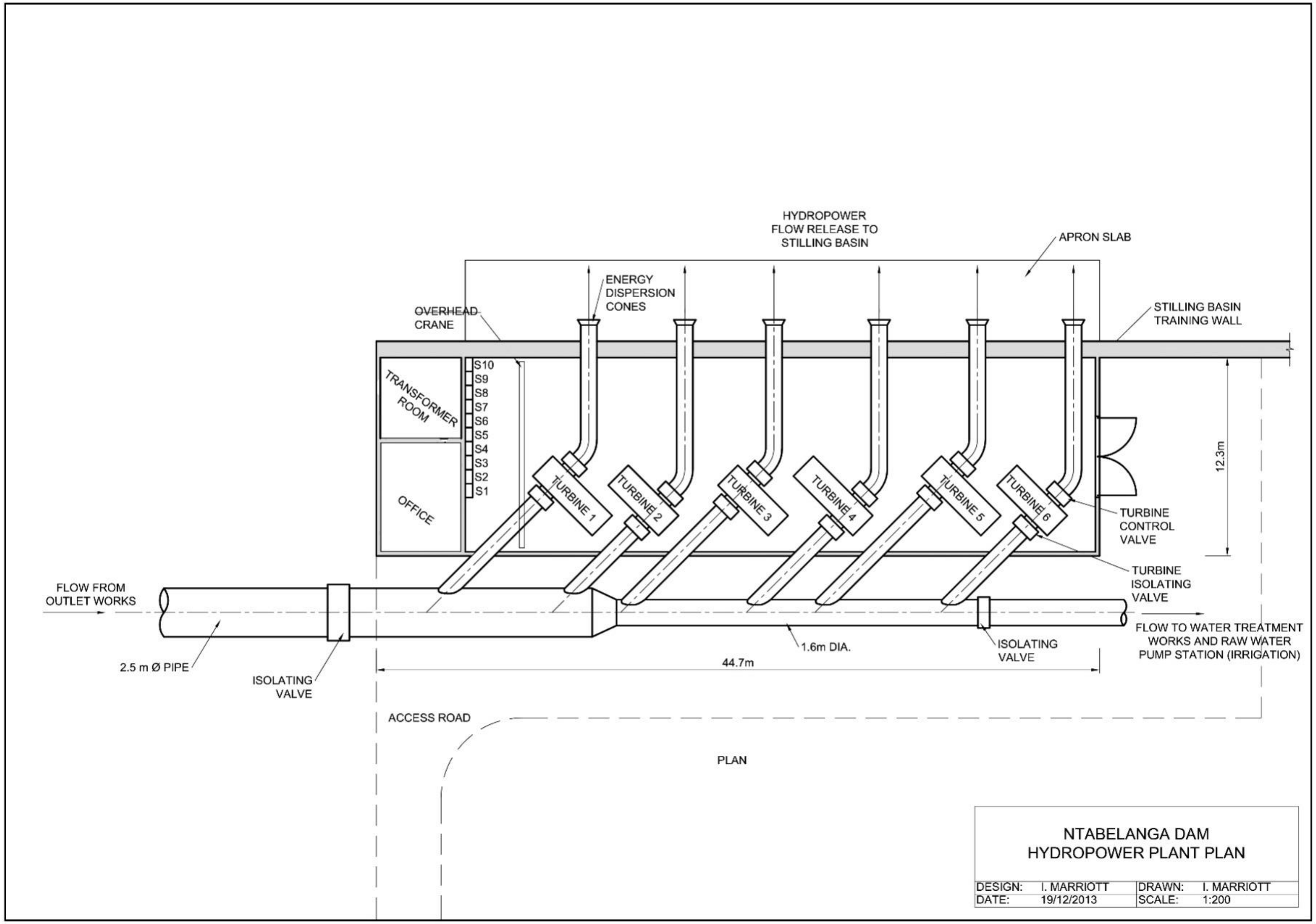


Figure 5-10: Proposed Layout of Ntabelanga Dam Hydropower Plant and EWR Discharge Point

These upgrades will be to a higher standard than the other roads above, and will be two lane carriageways (one each way) with a servitude width of between 20 m and 30 m (depending on terrain). The Maclear route would be some 18.9 km long and the Tsolo link some 12.9 km long. Once again, these improvements will be primarily aligned along existing routes, and this should have only limited or no resettlement or compensation implications.

Figure 5-11 shows new roads that will have to be constructed at the dam wall itself, and its appurtenant outlet works, hydropower plant, water treatment works and offices, staff housing, and pumping station site.

A new dam site access road will be required which will connect with the above upgraded road in from the Tsolo direction, and will run through the new operational works as shown. This road will have service roads branching off it to the temporary water works, the staff housing, the hydropower plant, the water and wastewater treatment plants, the pumping stations, accesses to the dam wall and outlet works, and then across the new river bridge to link with the upgraded existing roads on the north bank of the scheme.

The length of this new road will be approximately 5 km, and will have a servitude width of approximately 20 m. The existing land use features some subsistence agriculture which fields are fenced, but no habitable structures.

The dam site as a whole would need to be expropriated in its entirety, as well as the associated water treatment works, accommodation, access roads, and construction works areas shown on Figure 5-12. This will include a site for a proposed visitor’s centre, which will required resettlement involving two or three existing dwellings that can be seen on the figure.

5.9.2 Camps and Permanent Staff Accommodation

Several construction contracts are likely to be awarded to undertake the various components of this project. The construction of the works will provide employment opportunities for between 300 and 500 people for varying periods. Most of these jobs will be filled with labour commuting or being transported from local communities including the small villages close to the works as well as from the urban areas such as Maclear, Tsolo and Mthatha. It is not therefore expected that a significant amount of permanent camp accommodation would be required. The contractors will normally make this decision at tender stage in their approach and methodology, and costs for these requirements are included within the P&G items. There will, however, need to be some permanent staff accommodation built for the operational staff and their families, who will need to live close to the works.

Provision has therefore been made for a housing estate containing some 75 stands on which one-, two- and three-bedroomed staff houses can be built, as is indicated on Figure 5-9 above. These will also have fitted kitchens, bathrooms, lounge and dining rooms, and will have mains electricity, water, and waterborne sanitation.

Allowance has been made in the project budget for immediate construction of 20 x one-bedroom, 10 x two-bedroom, and 2 x three-bedroom houses in the first phase of the project. These requirements can be reviewed during the design stage.

Electricity will be via ESKOM connection, water supply from the Ntabelanga WTW, and a wastewater treatment facility will also be built. The housing complex will also have street lighting, tarred roads and surface water drainage.

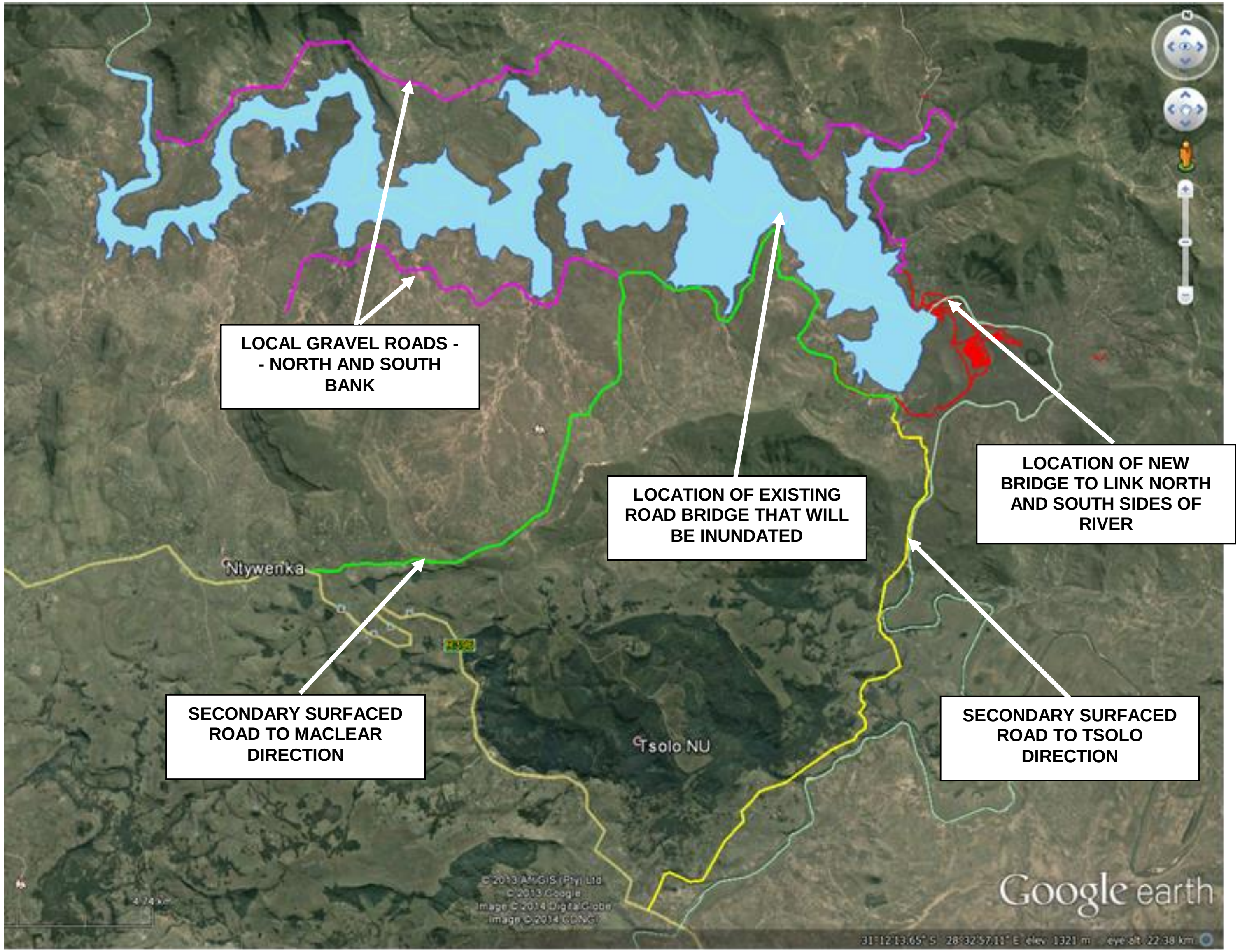


Figure 5-11: Roadways to be Permanently Upgraded Before and During Construction

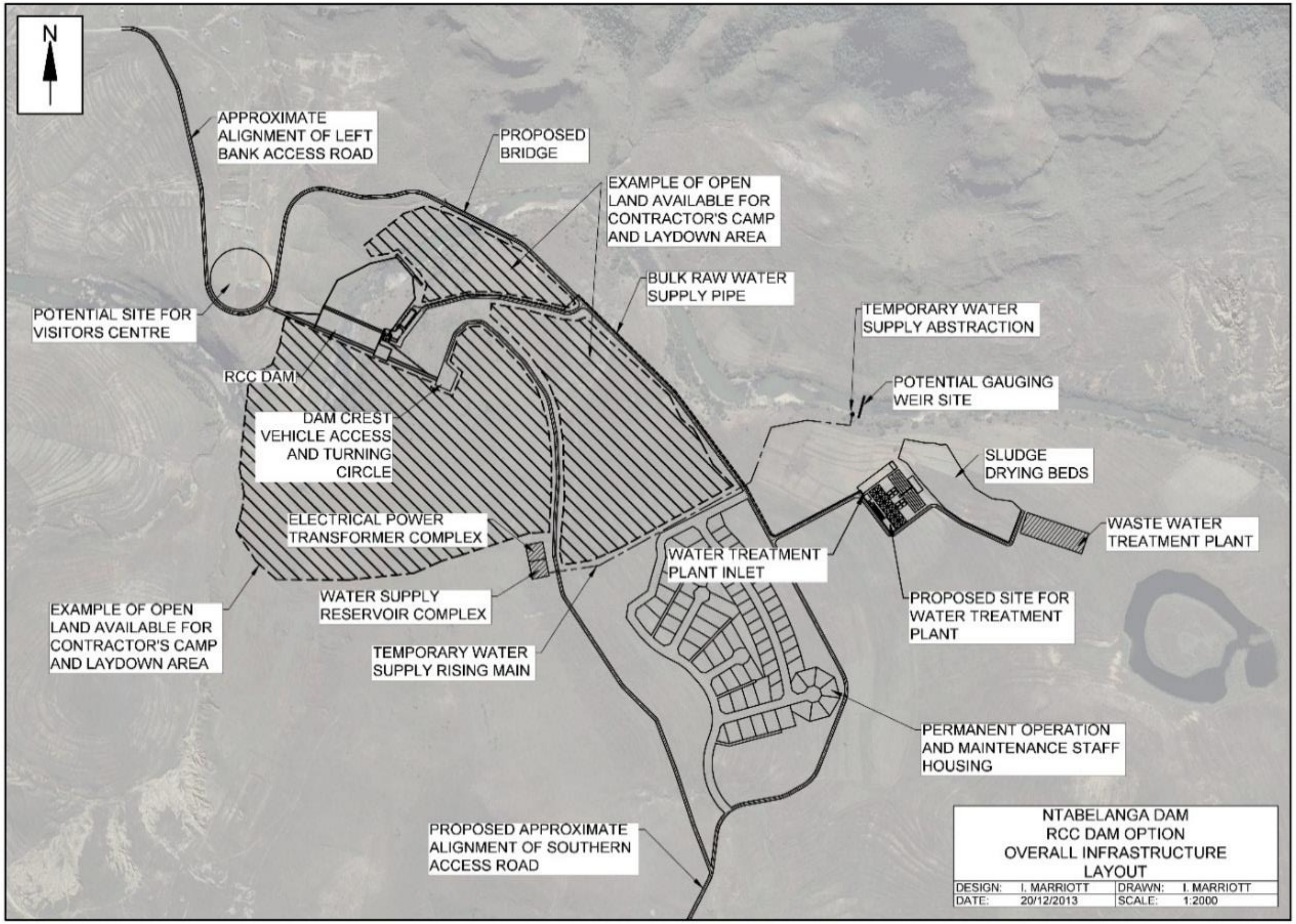


Figure 5-12: General Layout of the Proposed Dam and Associated Works

5.9.3 *Power Supplies*

The power requirements for the complete scheme are described in the Bulk Water Distribution Infrastructure Report No. P WMA 12/T30/00/5212/13. The total required is estimated as 12 572 kVA (circa 13 MW), with the majority of this centralized at the Ntabelanga Dam and WTW sites.

Table 5-5 summarises the duties and power requirements of the various energy consuming infrastructure components in the system.

ESKOM has 132 kV high voltage lines running parallel to the main road from Mount Frere to Mthatha and running through the project supply area from the above alignment to Maclear, passing between the Ntabelanga Dam and Tsolo. This is shown in green in Figure 5-13.

ESKOM are also implementing a programme of expansion of both high and medium voltage power supplies in the area, and information received from them indicates that this will eventually result in complete coverage of power services to all of the settlements in the area.

The Ntabelanga hydropower plant can only produce circa 2 000 kVA (2.0 MW) on average with a maximum of 5 000 kVA (5 MW), and there will therefore be a need to arrange for an ESKOM power supply to meet all of the project's needs in the Ntabelanga area, given that there will be times when the output of the hydropower plant will be very low or off-line.

Significant power will also be required in advance of the start of construction to supply contractor's camps, temporary water supply, site offices, accommodation, wastewater treatment, site lighting, dewatering, cranes and hoists, crushing and batching plants, etc. It is expected that such needs would also be in the order of 10 000 kVA (say 10 MW). The power supply connection from ESKOM to the Ntabelanga Dam site must therefore be implemented as an advance infrastructure component.

The conjunctive use hydropower scheme (i.e. Ntabelanga Dam in conjunction with the Lalini Dam and hydropower scheme), is expected to produce up to 37 500 kVA on a continuous basis, and this means that the conjunctive scheme will not only be "self-sufficient" in its energy usage for potable and irrigation water supply needs, but will also supply surplus energy into the local grid at the rate of up to 23 000 kVA (say 23 MW) continuously.

One option to be investigated is that the power produced by the conjunctive hydropower scheme can be "wheeled" into and out of the ESKOM grid system to the benefit of the long-term operating costs of the scheme, which is particularly important as regards the unit cost of raw water supplied to the irrigation scheme. This is discussed further in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15.

5.9.4 *Temporary Water Supply*

A temporary water supply will be required to supply potable water to the site during the construction period, and until the main WTW is commissioned. This will typically have a capacity of approximately 150 m^3/day, and it is usual for this facility to be a modular package plant.

The plant would be located such that water can be pumped from a river intake to the plant, and the treated water lifted into an elevated storage tank (24 hrs storage) serving the site by a gravity reticulation system. This elevated tank will later be used as the permanent treated water storage supplying the operations centre and housing, and its location has therefore been determined to meet this longer-term requirement. This water supply should also be installed as a part of the advance works.

Table 5-5 : Power Requirements for Scheme

2050 Power Requirements									
Treated Water	**Flow (l/s)**	**Head (m)**	**Duty Water Power (kW)**	**Pump Efficiency (%)**	**Maximum Electricity Demand (kW)**	**Maximum Electricity Demand (kVA)**	**Max hours per day**	**Usage - kWh per year**	**Power cost/year (Rand)**
Pump station PS1	935.27	246	2 257	75%	3 010	3 168	20	23 128 671	19 497 470
Pump station PS2	827.70	270	2 193	75%	2 924	3 077	20	22 465 459	18 938 382
Pump station PS3	476.66	279	1 305	75%	1 740	1 831	20	13 368 771	11 269 874
Pump station PS4	92.69	333	303	75%	404	425	20	3 102 814	2 615 672
Booster pump station Z3 PS1	170	94	157	75%	209	220	20	1 606 406	1 354 200
Booster pump station Z4 PS1	12.8	66	8	75%	11	12	20	84 924	71 591
Booster pump station Z4 PS2	3.53	195	7	75%	9	9	20	69 197	58 333
Water treatment plant processes	Estimated				500	526	varies	572 998	483 038
Waste water treatment works	Estimated				100	105	20	768 421	647 779
Housing	Estimated				250	263	12	1 152 632	971 668
Other, incl lighting etc	Estimated				250	263	12	1 152 632	971 668
TOTALS EXCL RAW WATER			**6 230**		**9 406**	**9 901**		**67 472 926**	**56 879 676**
Raw Water for Irrigation									
Main pumping station	1060	183	1 903	75%	2 538	2 671	20	19 500 041	16 438 535
Booster station P1	206	100	202	75%	269	284	20	2 070 836	1 745 715
Booster Station P2	223	165	361	75%	481	507	20	3 698 856	3 118 135
TOTALS INCL RAW WATER			**8 133**		**11 944**	**12 572**		**86 972 967**	**73 318 211**

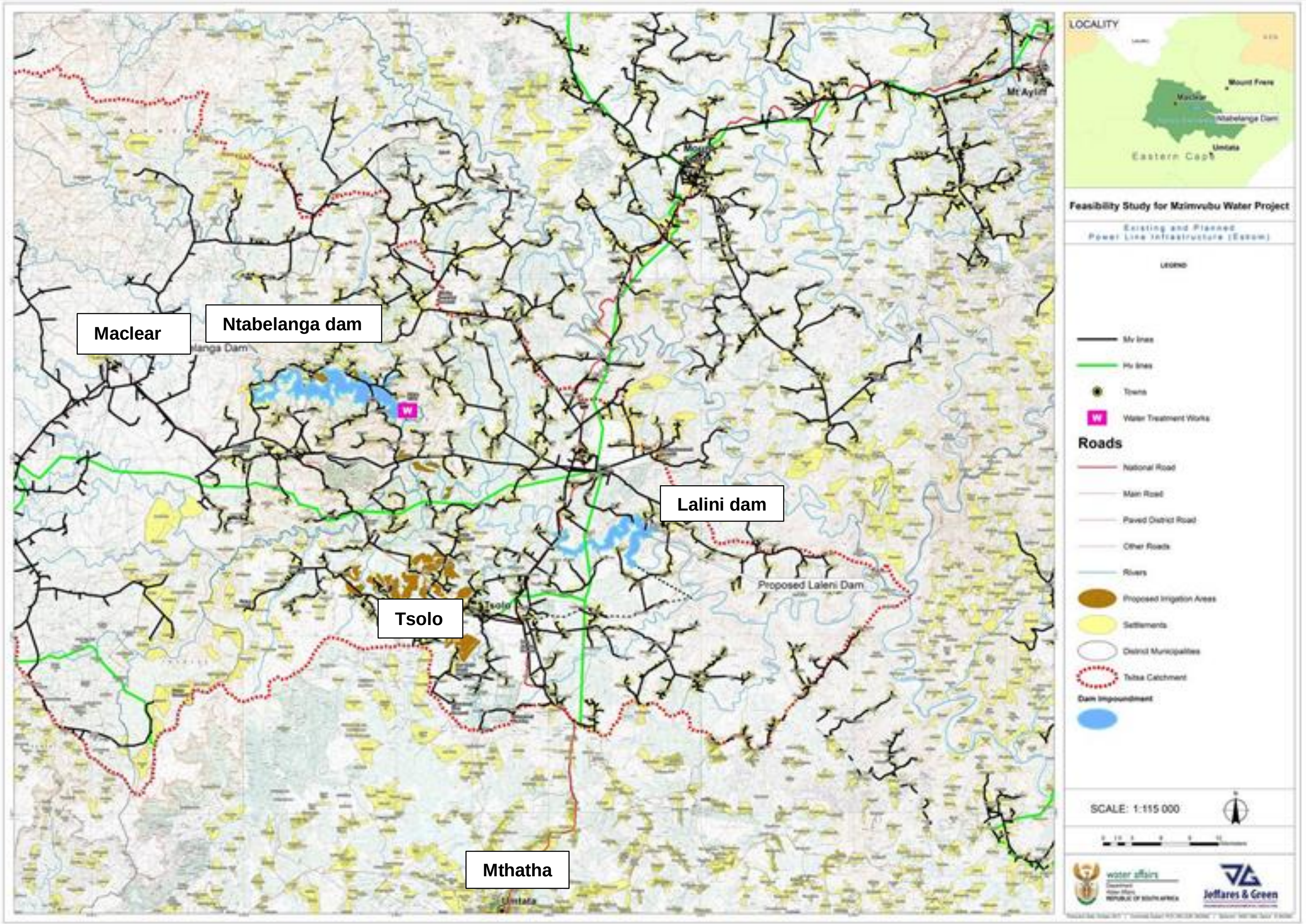


Figure 5-13: ESKOM Existing/Planned Power Distribution Network in the Project Supply Area

5.9.5 Flow Gauging Stations

Gauging stations will also be required as advance works in order to establish the ongoing monitoring of the river flows prior to and after construction of the dam. The hydrology section of the Department of Water and Sanitation has undertaken a reconnaissance of the scheme and their recommendations for flow measurement gauging stations on the project are given in Appendix G. The following sections summarize these recommendations.

a) Gauging Station Upstream of Ntabalenga Dam

Due to problematic access conditions to the Tsitsa River upstream of Ntabelanga, it is recommended that no gauging structure should be constructed to measure inflows into the dam. Inflows should rather be determined indirectly by means of a dam balance calculation. With a dam balance process the actual inflow into a dam can be calculated, using the rainfall and evaporation data collected at the dam in combination with the changes in reservoir level and releases or spills from Ntabelanga Dam.

b) Gauging Structure Downstream of Ntabelanga Dam

It is recommended that a dedicated gauging weir should be constructed approximately 1.5 km downstream of the dam (see Figure 5-14).

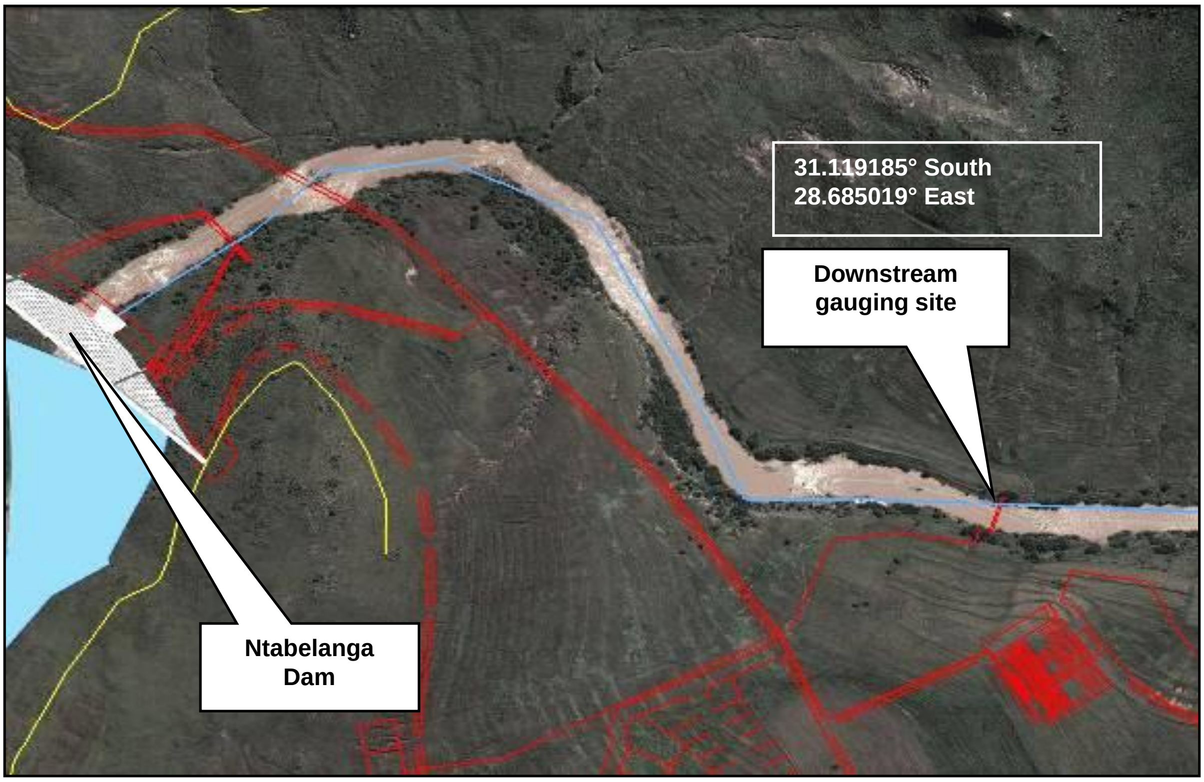


Figure 5-14: Recommended new gauging weir site downstream of Ntabelanga

This structure should be capable of measuring the total range of controlled releases from the dam into the river and also the first 300 mm to 500 mm of flow flowing over the spillway accurately. The structure should also measure the flows flowing through the hydropower turbines at Ntabelanga.The first gauging station immediately below the Ntabelanga Dam would be an ideal weir structure from which to abstract raw water for the temporary water supply, and it is proposed that the abstraction system be located at this gauging station.

c) *Gauging Structure in Tsitsa River Downstream of Inxu Confluence*

A gauging weir to measure the flow contribution of the Inxu River is recommended at a site approximately 12.7 km downstream of the confluence.

Two sites approximately 200 m apart have been identified (see Figure 5-15), and should be evaluated for construction during the detailed design stage.

Figure 5-15: Recommended new gauging weir sites downstream on Inxu confluence

d) *Tsitsa Upstream of Lalini*

If the preferred Lalini Dam scenario is to be implemented the existing DWS gauging structure T3H006 in the Tsitsa, just downstream of the N2 road bridge, will be inundated by the dam when full. In that case a new structure needs to be constructed to replace T3H006 immediately upstream of the influence sphere of the Laleni Dam, upstream of the N2 road (See Figure 5-16).

e) *Tsitsa Downstream of Lalini*

Two potential gauging sites downstream of Lalini have been identified approximately 1.3 km and 1.6 km downstream of the wall (See Figure 5-16). Site 2 is preferred as conditions appear more favourable, however it is proposed that both the sites should be included in the environmental process, as they are only 300 m apart. If foundation conditions at site 2 are poorer than expected, it might be necessary to utilise site 1, but constructing a higher than normal gauging structure to overcome the complex flow conditions expected at this site.

f) *Tsitsa downstream of the Lalini Dam hydropower turbines*

A gauging structure capable of measuring the maximum flow through the turbines accurately, located before any water is discharged back into the Tsitsa River is recommended. The structure should be located and designed in such a manner that flows coming down the Tsitsa will not impact on the gauging accuracy of the turbine measurements (See Figure 5-15).

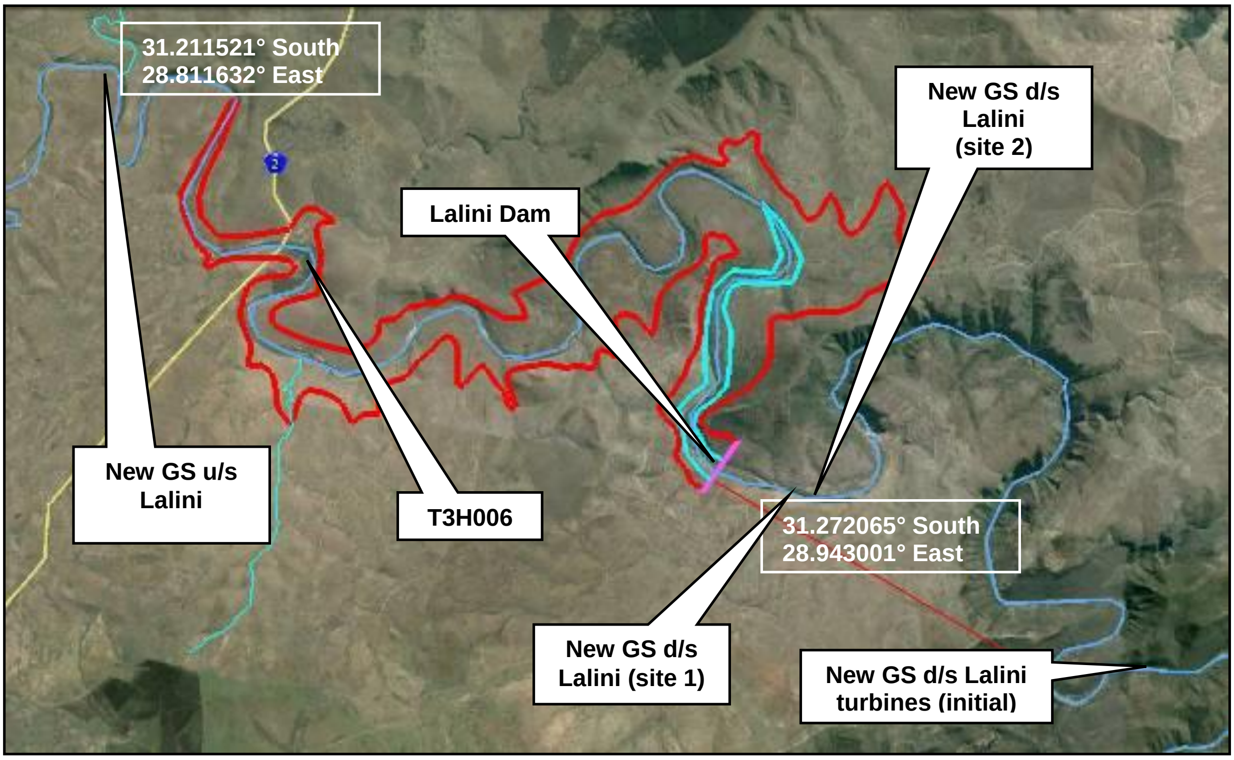


Figure 5-16: Recommended new gauging weir sites upstream and downstream of Lalini Dam

5.9.6 *Wastewater Treatment Plant*

A wastewater treatment plant will be required to treat effluents produced by the Ntabelanga Dam operations centre and housing. This will be appropriately sized for this purpose and it is probable that this requirement could be met by using a screening and pre-treatment process followed by a reed bed system.

It is not recommended that such a wastewater treatment plant be designed or used to treat the effluent from the construction activities, as this would be oversized and would have to deal with industrial pollutants as well as domestic effluents. The contractors themselves must be made responsible for the safe and environmentally sensitive disposal of all of their effluents and waste products, leaving only domestic effluents for the permanent wastewater treatment plant to deal with.

5.9.7 *Telecommunications*

Whilst the Vodacom network in the region has good coverage, telecommunications in the particular area of the Ntabelanga Dam works are limited, and improved communication systems will be required before the construction activities commence. This should include increasing the reliability and coverage of the cellular network system, as well as providing land lines, and data lines with sufficient transmission speeds for modern communications equipment.

This is normally dealt with by requesting quotations from the nationally-based telecommunications service providers, and this is also considered to be an important advance infrastructure requirement.

5.9.8 Visitor's Centre

The Ntabelanga Dam and its body of water will provide opportunities for tourism and recreation, which in turn can lead to job creation. Many large dams take up such opportunities and offer visitor facilities to encourage tourism and thus promote economic development.

A visitor's centre can form the focus of such an initiative and provide visitors with a view of the works, and information on the project and water related aspects, including the cultural and tourism activities in the area. A location for this centre is suggested above on Figure 5-9. It is recommended that such a building be of interesting architecture in keeping with the local culture and terrain.

5.9.9 Priority Infrastructure

The following are considered to be other works components that should be constructed as a priority:

- Main access roads, including roads at Ntabelanga Dam site shown on Figure 5-11;
- Bridge across the river downstream of the dam;
- Power supplies to the site;
- Temporary water supply and gauging station;
- Other gauging stations; and
- Telecommunications.

Also optional:

- Staff accommodation – if to be used by DWS during construction – do not allow contractor to use; and
- Wastewater treatment plant – if staff accommodation is built.

Most of the above works will require an EIA authorization, and it is therefore essential that the urgency of such priority works is not overlooked during the EIA authorization process.

Phase 1 of the Feasibility Study also identified the needs and benefits of a concerted catchment rehabilitation and management programme. This has been handed over to the Eastern Cape Provincial Department of Environmental Affairs, who are in the process of developing this programme, which has commenced in April 2014.

5.9.10 Compensation and Mitigation Works

The EIA PSP might identify other mitigations and compensation works that will require engineering inputs and construction activities. These will then become part of the project implementation and might include, *inter alia*:

- Relocation of homesteads affected by the scheme;
- Additional feeder roads, footbridges, etc;.
- Improvements to local water supplies not included in the proposed scheme;
- A sanitation programme; and
- Improvements to clinics, schools and police stations in the areas affected by the dam.

Budgets have been allowed in the cost estimates for these other potential works, the implementation of which should be carried forward into the detailed design stage.

6. COST ESTIMATE

6.1 Capital Costs

The cost estimate for the Ntabelanga Dam and its associated infrastructure, water supply and irrigation schemes, land care programme, and in-field development of irrigated farming units, is given in Table 6-1.

This does not include any of the Lalini Dam and hydropower scheme infrastructure which is dealt with in a separate Report No P WMA 12/T30/00/5212/19. This dam is, however, sized to provide adequate flow releases downstream when operating conjunctively with the Lalini Hydropower scheme component.

Table 6-1: Capital Cost Estimates

COMPONENT	R'million
Ntabelanga dam and associated works	1 075
Ntabelanga dam hydropower works	88
Ntabelanga land compensation/mitigation costs	18
Ntabelanga power transmission	29
Sub-Total Ntabelanga Dam and Associated Works	*1 209*
Engineering and EMP Costs (12%)	*145*
Sub-Total Ntabelanga Dam and Associated Works incl Eng & EMP	*1 354*
Escalation in Each Year @ 5.5% p.a.	*265*
Sub-Total Ntabelanga Dam and Associated Works incl Eng, EMP & ESC	*1 619*
VAT (14%)	227
Add in R22 million per year for catchment management (no esc)	220
Allowance for other offset activities (50% of R 100 million)	50
Total Ntabelanga Dam and Associated Works (incl Esc + VAT)	**2 116**
COMPONENT	**R'million**
Ntabelanga water treatment works	643
Ntabelanga primary & secondary bulk treated water distribution system	1 234
Ntabelanga tertiary bulk treated water distribution system (DM's)	1 425
Ntabelanga bulk irrigation water supply system	497
Sub-Total Ntabelanga WTW and Bulk Water Systems	*3 799*
Engineering and EMP Costs (12%)	*456*
Sub-Total Ntabelanga WTW and Bulk Water Systems incl Eng & EMP	*4 255*
Escalation in Each Year @ 5.5% p.a.	*1 067*
Sub-Total Ntabelanga WTW and Bulk Water Systems incl Eng, EMP & ESC	*5 322*
VAT (14%)	745
Total Ntabelanga WTW and Bulk Water Systems (incl Esc + VAT)	**6 068**

…. (cont.)

Table 6-1: Capital Cost Estimates (cont.)

COMPONENT	R'million
In-farm irrigation investment costs	**105**
Engineering and EMP Costs (12%)	*13*
Sub-Total in-farm irrigation investment costs incl Eng & EMP	*118*
Escalation in Each Year @ 5.5% p.a.	*40*
Sub-Total in-farm irrigation investment costs incl Eng, EMP & ESC	*158*
VAT (14%)	22
Total in-farm irrigation investment costs (incl Esc + VAT)	**180**
GRAND TOTAL NTABELANGA (R'MILLION INCL ESC AND VAT)	**8 364**

More detailed costing breakdowns and cashflow projections for each individual project component are given in Report No. P WMA 12/T30/00/5212/15. It should be noted that there are several risks involved in the accuracy of the above cost estimate:

- Estimating at feasibility level at best has a confidence level of ± 10%
- Escalation rates could increase or decrease, especially given the volatile nature of the economy at the moment
- Rand foreign exchange rates are also volatile and this will affect the cost of all Imported materials, services and equipment.
- The timing of the various components implementation may change which, if later, would increase the escalation cost.
- The amount of non-grant finance is unknown, and if significant will increase costs, depending on the terms of such loans, interest rates and foreign exchange rates.

One example of the impact of the above risks is that every month's delay in fully implementing a R8.4 billion project increases escalation cost by R38.5 million (at 5.5% p.a.)

6.2 Estimated Operation and Maintenance Costs

Operation and maintenance costs will to some extent depend upon the institutional arrangements set up to operate the scheme, and the structures and management costs of the one or more entities involved. Economies of scale can be lost if the management and operation of the works is split between several different organisations.

An estimate has been made of the likely management, maintenance and operational costs of these works based upon current costs and salary scales. Maintenance costs per annum are based upon the percentages of capital cost recommended in DWS's Water Supply Planning and Design Guidelines.

Operational staffing costs have been sourced from those currently applied to similar works operated by Amatola Water.

Energy costs (pumping, etc.) are based upon an average tariff per kWh using ESKOM's Ruraflex tariff, and assuming that pumping would be restricted to non-peak hours (i.e. up to 19 hours pumping per day). This is the current tariff used for pumping by Amatola Water in this region. Table 6-2 summarizes these annual operating and maintenance costs, but these should be treated with caution pending decisions being made on the eventual institutional arrangements.

Table 6-2: Annual Management, Operation, and Maintenance Cost Estimate (2014 Price Levels)

COMPONENT	ANNUAL MAINTENANCE COSTS (R'MILLION)	ANNUAL OPERATIONS STAFFING COSTS (R'MILLION)	POWER COSTS/ANNUM (R'MILLION)		TREATMENT COSTS/ANNUM (R'MILLION)
			ON COMMISSIONING	BY 2050	
NTABELANGA DAM + MINI HYDRO + ASSOCIATED INFRASTRUCTURE	8	4.2	3	3	
NTABELANGA WTW AND POTABLE BULK WATER SYSTEM (PRIMARY ONLY)	20.1	12.3	36	48.9	7.7
NTABELANGA POTABLE BULK WATER SYSTEM (SECONDARY)	9	4.1	2.5	3	
NTABELANGA POTABLE BULK WATER SYSTEM (TERTIARY)	12	11.6	1.5	2	
NTABELANGA IRRIGATION SYSTEM (DELIVERY TO EDGE OF FIELDS)	5.3	2.5	18.6	18.6	
LALINI DAM AND HYDROPOWER SCHEME	29.9	6.8	3	3	
TOTALS R'MILLION/ANNUM	**84.3**	**41.5**	**64.6**	**78.5**	**7.7**

7. IMPLEMENTATION PROGRAMME

A draft implementation programme has been developed and is included in Annexure 1.

This is under review by the DWS and should continue to be regularly updated.

8. REFERENCES

1. Martin Wieland and R. Peter Brenner (2004), paper no. 3399, Earthquake Aspects of Roller Compacted Concrete and Concrete-Face Rock fill Dams, World Conference on Earthquake Engineering, Vancouver.

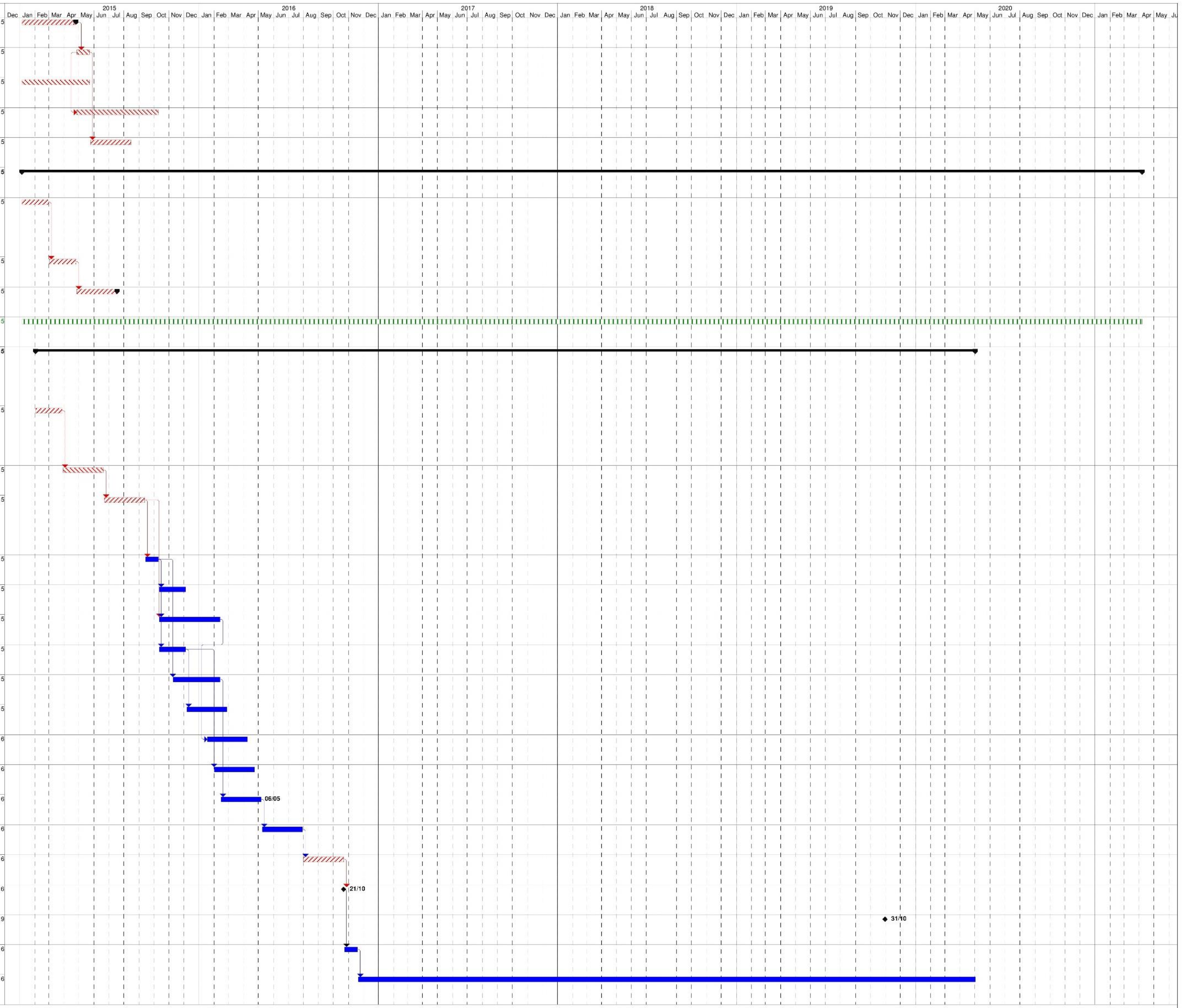
2. Van den Berg, M and Parrock, A L, (2009), Major Dam Foundation Design and Validation. Sustainable Development of Dams in Southern Africa, SANCOLD Conference, 4 – 6 November 2009.

3. Van Schalkwyk, A, Mouton, D and van der Merwe, A, (2009), Pitfalls in the Interpretation of Information from Rotary Core Drilling for the Prediction of Foundation Conditions for Dams. Sustainable Development of Dams in Southern Africa, SANCOLD Conference, 4 – 6 November 2009.

ANNEXURE 1

DRAFT IMPLEMENTATION PROGRAMME

ID	Task Name	Duration	Start
1	Project DEA Authorization	80 days	Mon 05/01/15
2	Ministerial Review and Approval to Proceed	20 days	Mon 27/04/15
3	Institutional Arrangements Memorandum	100 days	Mon 05/01/15
4	Project Funding Arrangements	120 days	Mon 27/04/15
5	Gazetting of Project and Receipt of Comments	60 days	Mon 25/05/15
6	**PROJECT MANAGEMENT UNIT**	**1632 days**	**Mon 05/01/15**
7	Prepare Terms of Reference, Request for Proposals, Adverts, and Invite Tenders	40 days	Mon 05/01/15
8	Tender Period	40 days	Mon 02/03/15
9	Evaluation of Tenders and Appointment of Project Management Team	60 days	Mon 27/04/15
10	**CATCHMENT REHABILITATION & ONGOING MANAGEMENT**	1632 days	Mon 05/01/15
11	**NTABELANGA DAM : DETAILED DESIGN/TENDER/SUPERVISION**	**1370 days**	**Mon 02/02/15**
12	Prepare PSP ToR, Request for Proposals, Adverts, and Invite Tenders	40 days	Mon 02/02/15
13	Tender Period	60 days	Mon 30/03/15
14	Evaluation of Tenders and Appointment of Design Tendering and Supervision PSP	60 days	Mon 22/06/15
15	Information Gathering and Review Period	20 days	Mon 14/09/15
16	Supplementary Survey of Dam Site and Associated Works	40 days	Mon 12/10/15
17	Detailed geotechnical and materials investigations	90 days	Mon 12/10/15
18	Review Feasibility Design and Optimisation of Works	40 days	Mon 12/10/15
19	Spillway Laboratory Modelling & Optimisation	70 days	Mon 09/11/15
20	Detailed design of dam & intake/outlet works	60 days	Mon 07/12/15
21	Final Cost Estimates and Implementation cashflows	60 days	Mon 18/01/16
22	Preparation of Tender Drawings	60 days	Mon 01/02/16
23	Preparation of Bidding Documents	60 days	Mon 15/02/16
24	Invitations to Tender and Tender Period	60 days	Mon 09/05/16
25	Evaluate Tenders Received	60 days	Mon 01/08/16
26	Award of Construction Contract	0 days	Fri 21/10/16
27	Targeted First Filling of Dam	0 days	Thu 31/10/19
28	Contractor Mobilisation	20 days	Mon 24/10/16
29	Dam Construction Period	900 days	Mon 21/11/16

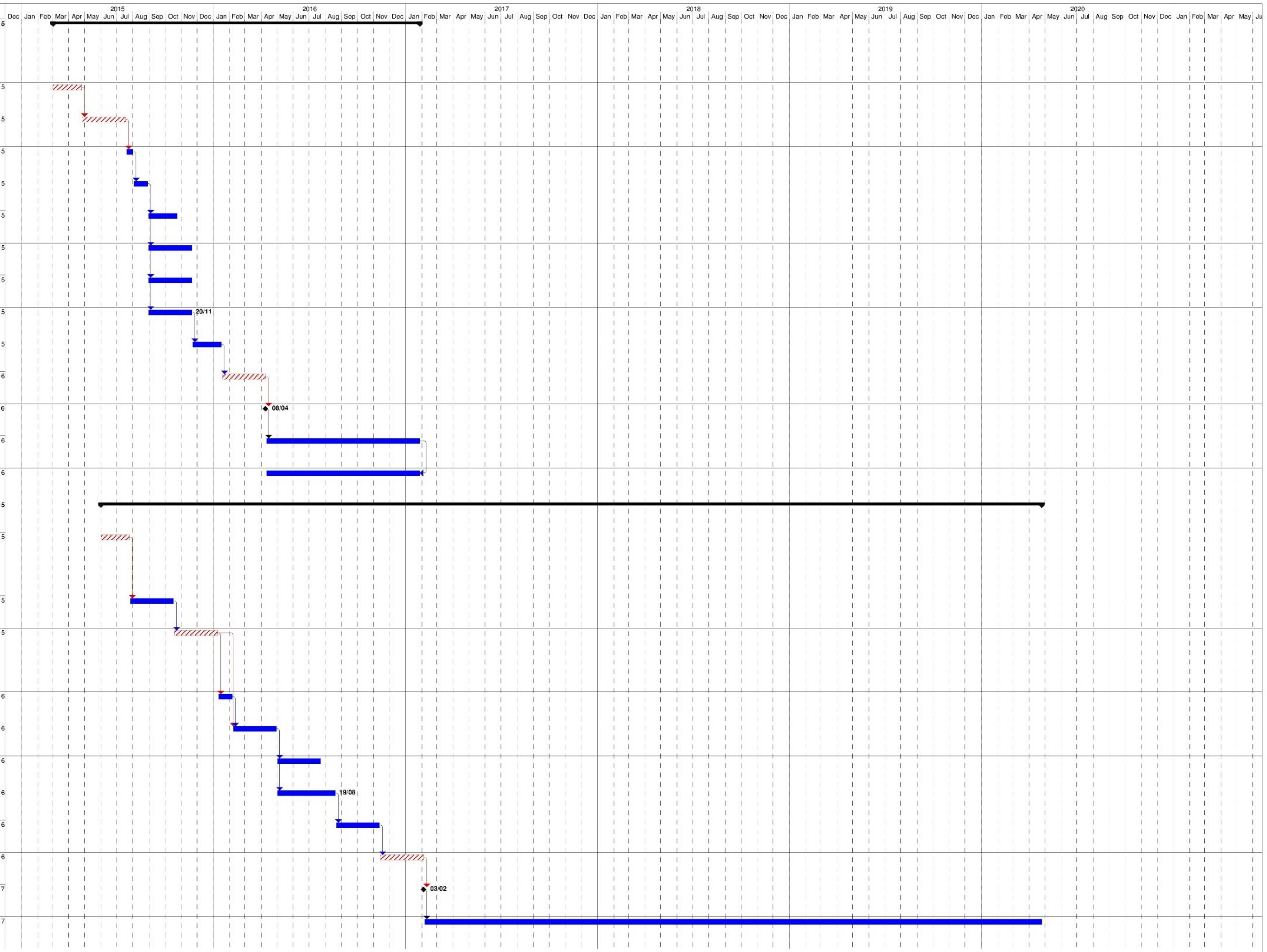


Task, Split, Milestone, Summary, Project Summary, External Tasks, External Milestone, Inactive Milestone, Inactive Summary, Manual Task, Duration-only, Manual Summary Rollup, Manual Summary, Start-only, Finish-only, External Tasks, External Milestone, Progress, Deadline

DRAFT PROGRAMME FOR MZIMVUBU WATER PROJECT - DETAILED DESIGN AND CONSTRUCTION STAGES - OCTOBER 2014 VERSION

1 OF 6

ID	Task Name	Duration	Start
30	**NTABELANGA ADVANCE ACCESS ROADS, BRIDGE AND POWER SUPPLIES**	**500 days**	**Mon 02/03/15**
31	Procurement of Detailed Design, Tendering & Supervision PSP	40 days	Mon 02/03/15
32	Adjudicate and Award of Contract and Mobilize PSP	60 days	Mon 27/04/15
33	Information Gathering and Review Period	10 days	Mon 20/07/15
34	Supplementary Survey of Road and Power Lines	20 days	Mon 03/08/15
35	Detailed geotechnical and materials investigations	40 days	Mon 31/08/15
36	Detailed design of roads, bridges and power lines	60 days	Mon 31/08/15
37	Preparation of Tender Drawings	60 days	Mon 31/08/15
38	Preparation of Bidding Documents	60 days	Mon 31/08/15
39	Invitations to Tender and Tender Period	40 days	Mon 23/11/15
40	Evaluate Tenders Received	60 days	Mon 18/01/16
41	Award of Construction Contract	0 days	Fri 08/04/16
42	Construction of Access Roads and Bridge	210 days	Mon 11/04/16
43	Construction of Power Lines	210 days	Mon 11/04/16
44	**NTABELANGA WTW : DESIGN AND BUILD CONTRACT**	**1280 days**	**Mon 01/06/15**
45	Prepare Terms of Reference, Request for Proposals, Adverts, and Invite Tenders	40 days	Mon 01/06/15
46	Tender Period	60 days	Mon 27/07/15
47	Evaluation of Tenders and Appointment of Design Manager and Supervision Team	60 days	Mon 19/10/15
48	Information Gathering and Review Period	20 days	Mon 11/01/16
49	Detailed geotechnical and materials investigations	60 days	Mon 08/02/16
50	Final Cost Estimates and Implementation cashflows	60 days	Mon 02/05/16
51	Preparation of Bidding Documents	80 days	Mon 02/05/16
52	Invitations to Tender and Tender Period	60 days	Mon 22/08/16
53	Evaluate Tenders Received	60 days	Mon 14/11/16
54	Award of Design and Build Contract	0 days	Fri 03/02/17
55	WTW Design, Construction and Commissioning Period	840 days	Mon 06/02/17



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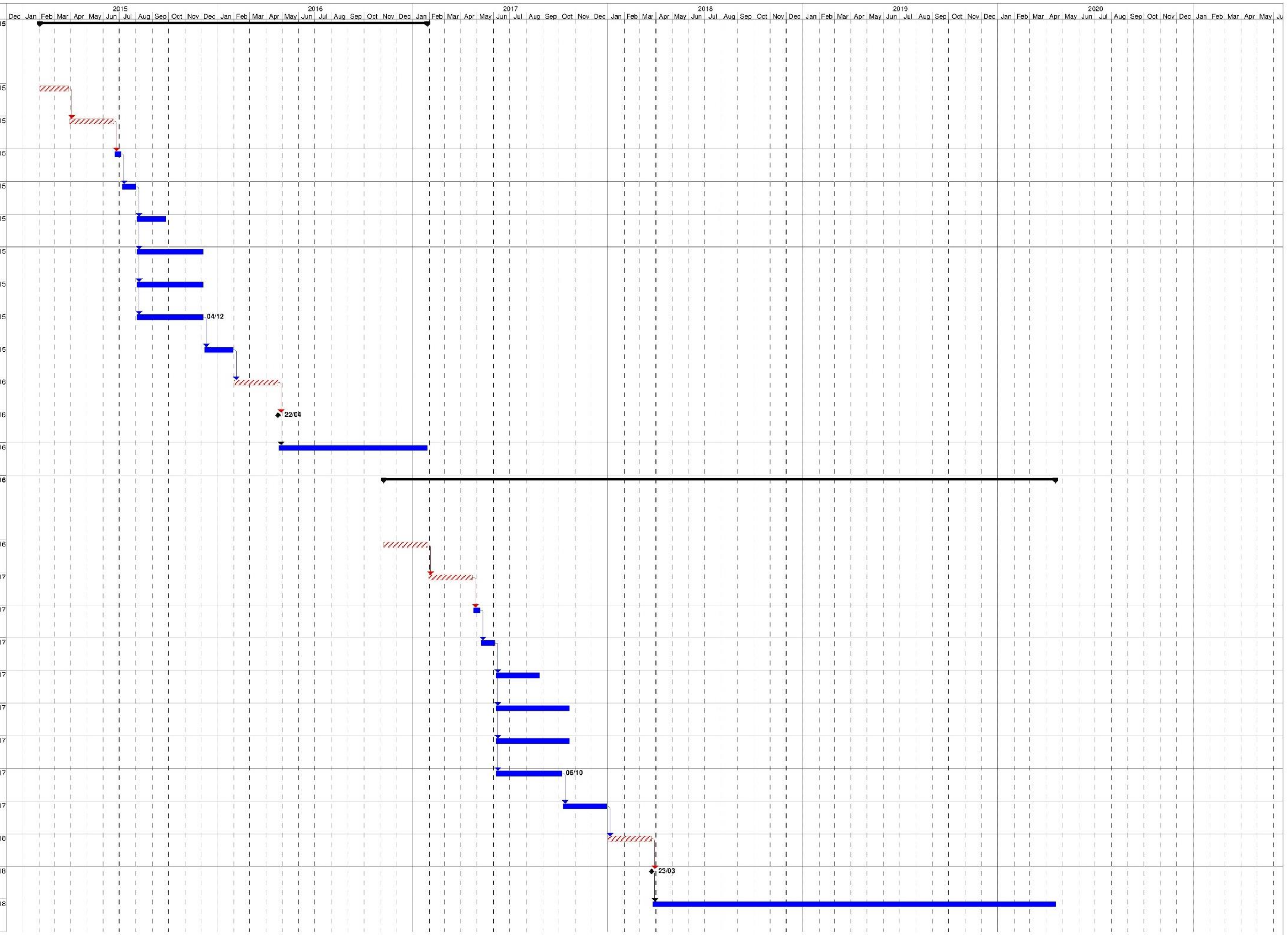
DRAFT PROGRAMME FOR MZIMVUBU WATER PROJECT - DETAILED DESIGN AND CONSTRUCTION STAGES - OCTOBER 2014 VERSION — 2 OF 6

ID	Task Name	Duration	Start
56	**NTABELANGA OPERATIONS ACCOMMODATION VILLAGE/VISITORS CENTRE**	**520 days**	**Mon 02/02/15**
57	Procurement of Detailed Design, Tendering & Supervision PSP	40 days	Mon 02/02/15
58	Award of Contract and Mobilize PSP	60 days	Mon 30/03/15
59	Information Gathering and Review Period	10 days	Mon 22/06/15
60	Supplementary Survey of Works Areas	20 days	Mon 06/07/15
61	Detailed geotechnical and materials investigations	40 days	Mon 03/08/15
62	Detailed design of works	90 days	Mon 03/08/15
63	Preparation of Tender Drawings	90 days	Mon 03/08/15
64	Preparation of Bidding Documents	90 days	Mon 03/08/15
65	Invitations to Tender and Tender Period	40 days	Mon 07/12/15
66	Evaluate Tenders Received	60 days	Mon 01/02/16
67	Award of Construction Contract	0 days	Fri 22/04/16
68	Construction of Works	200 days	Mon 25/04/16
69	**NTABELANGA DAM PRIMARY AND SECONDARY BULK POTABLE WATER SYSTEM**	**900 days**	**Mon 07/11/16**
70	Procurement of Detailed Design, Tendering & Supervision PSP	60 days	Mon 07/11/16
71	Adjudicate and Award of Contract and Mobilize PSP	60 days	Mon 30/01/17
72	Information Gathering and Review Period	10 days	Mon 24/04/17
73	Supplementary Survey of Works Areas	20 days	Mon 08/05/17
74	Detailed geotechnical and materials investigations	60 days	Mon 05/06/17
75	Detailed design of works	100 days	Mon 05/06/17
76	Preparation of Tender Drawings	100 days	Mon 05/06/17
77	Preparation of Bidding Documents	90 days	Mon 05/06/17
78	Invitations to Tender and Tender Period	60 days	Mon 09/10/17
79	Evaluate Tenders Received	60 days	Mon 01/01/18
80	Award of Construction Contract	0 days	Fri 23/03/18
81	Construction of Works	540 days	Mon 26/03/18

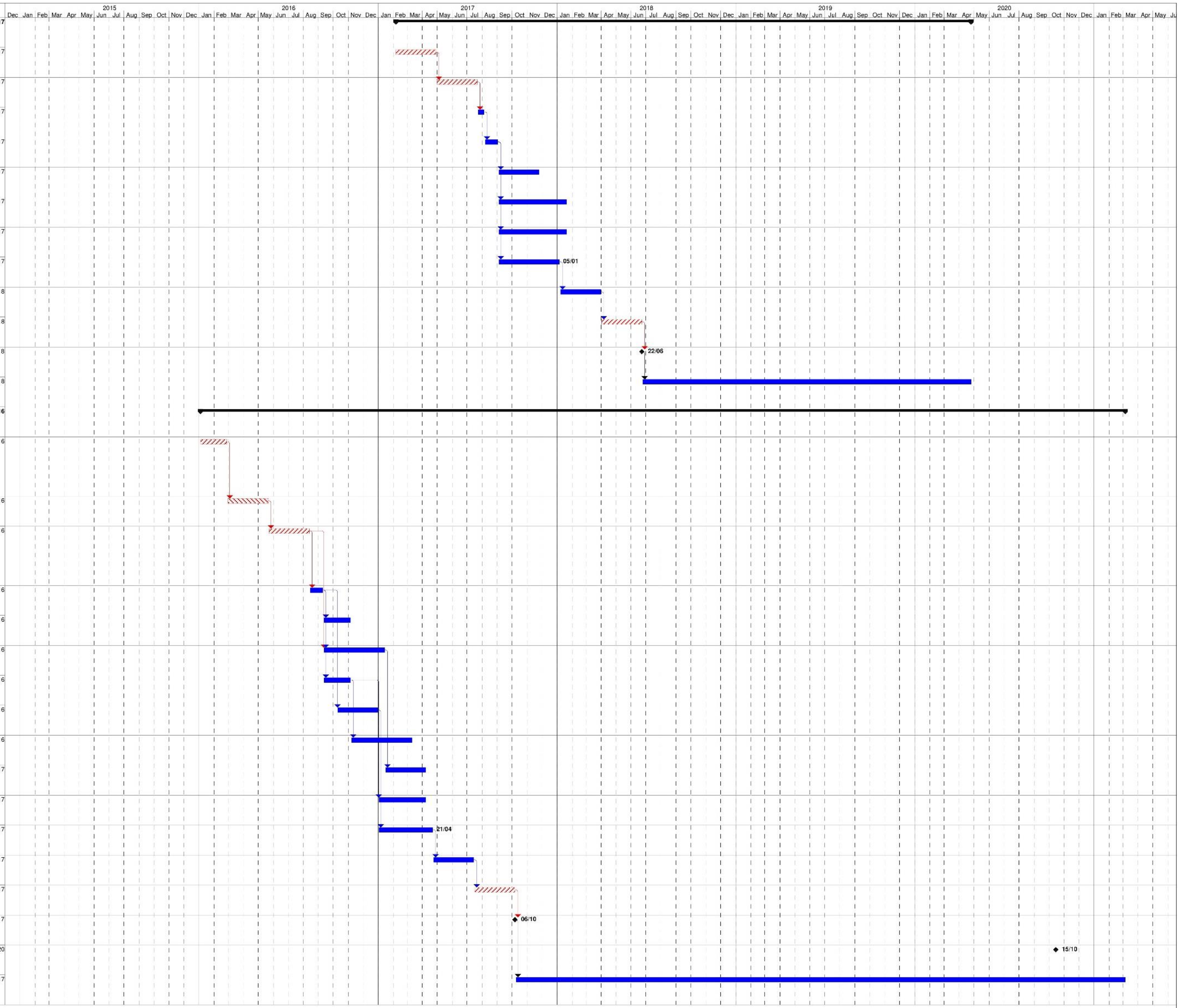


Task | Split | Milestone | Summary | Project Summary | External Tasks | External Milestone | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Progress | Deadline

DRAFT PROGRAMME FOR MZIMVUBU WATER PROJECT - DETAILED DESIGN AND CONSTRUCTION STAGES - OCTOBER 2014 VERSION

3 OF 6

ID	Task Name	Duration	Start
82	**NTABELANGA DAM BULK IRRIGATION WATER SYSTEM**	**840 days**	**Mon 06/02/17**
83	Procurement of Detailed Design, Tendering & Supervision PSP	60 days	Mon 06/02/17
84	Adjudicate and Award of Contract and Mobilize PSP	60 days	Mon 01/05/17
85	Information Gathering and Review Period	10 days	Mon 24/07/17
86	Supplementary Survey of Works Areas	20 days	Mon 07/08/17
87	Detailed geotechnical and materials investigations	60 days	Mon 04/09/17
88	Detailed design of works	100 days	Mon 04/09/17
89	Preparation of Tender Drawings	100 days	Mon 04/09/17
90	Preparation of Bidding Documents	90 days	Mon 04/09/17
91	Invitations to Tender and Tender Period	60 days	Mon 08/01/18
92	Evaluate Tenders Received	60 days	Mon 02/04/18
93	Award of Construction Contract	0 days	Fri 22/06/18
94	Construction of Works	480 days	Mon 25/06/18
95	**LALINI DAM : DETAILED DESIGN/TENDER/SUPERVISION**	**1350 days**	**Mon 04/01/16**
96	Prepare Terms of Reference, Request for Proposals, Adverts, and Invite Tenders	40 days	Mon 04/01/16
97	Tender Period	60 days	Mon 29/02/16
98	Evaluation of Tenders and Appointment of Design Tendering and Supervision Team	60 days	Mon 23/05/16
99	Information Gathering and Review Period	20 days	Mon 15/08/16
100	Supplementary Survey of Dam Site and Associated Works	40 days	Mon 12/09/16
101	Detailed geotechnical and materials investigations	90 days	Mon 12/09/16
102	Preliminary Design and Optimisation of Works	40 days	Mon 12/09/16
103	Spillway Laboratory Modelling & Optimisation	60 days	Mon 10/10/16
104	Detailed design of dam & associated infrastructure	90 days	Mon 07/11/16
105	Final Cost Estimates and Implementation cashflows	60 days	Mon 16/01/17
106	Preparation of Tender Drawings	70 days	Mon 02/01/17
107	Preparation of Bidding Documents	80 days	Mon 02/01/17
108	Invitations to Tender and Tender Period	60 days	Mon 24/04/17
109	Evaluate Tenders Received	60 days	Mon 17/07/17
110	Award of Construction Contract	0 days	Fri 06/10/17
111	Targeted First Filling of Dam	0 days	Thu 15/10/20
112	Dam Construction Period	890 days	Mon 09/10/17



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DRAFT PROGRAMME FOR MZIMVUBU WATER PROJECT - DETAILED DESIGN AND CONSTRUCTION STAGES - OCTOBER 2014 VERSION

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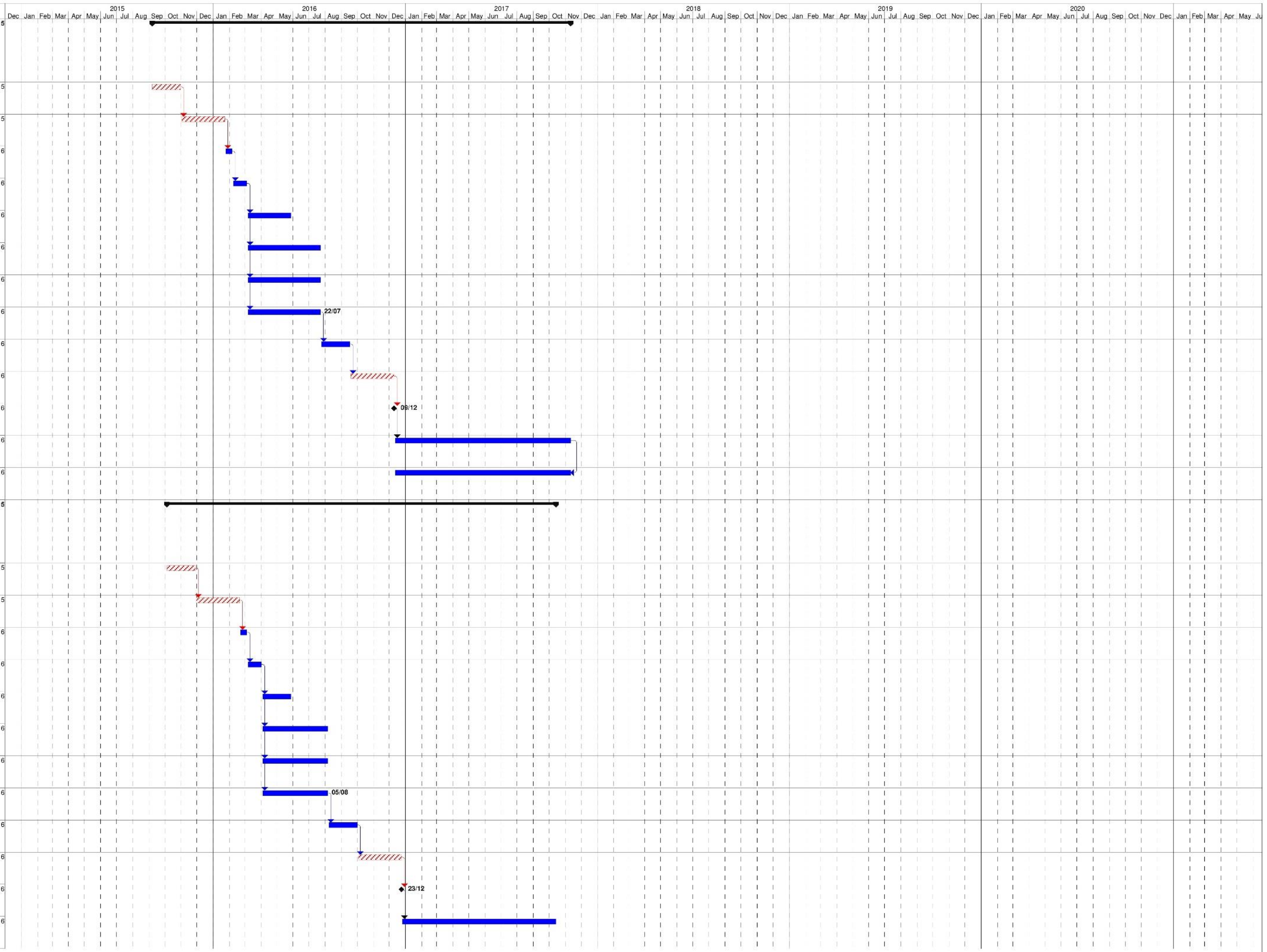
ID	Task Name	Duration	Start
113	**LALINI ADVANCE ACCESS ROADS, BRIDGES AND POWER SUPPLIES**	**570 days**	**Mon 07/09/15**
114	Procurement of Detailed Design, Tendering & Supervision PSP	40 days	Mon 07/09/15
115	Award of Contract and Mobilize PSP	60 days	Mon 02/11/15
116	Information Gathering and Review Period	10 days	Mon 25/01/16
117	Supplementary Survey of Road and Power Lines	20 days	Mon 08/02/16
118	Detailed geotechnical and materials investigations	60 days	Mon 07/03/16
119	Detailed design of roads, bridges and power lines	100 days	Mon 07/03/16
120	Preparation of Tender Drawings	100 days	Mon 07/03/16
121	Preparation of Bidding Documents	100 days	Mon 07/03/16
122	Invitations to Tender and Tender Period	40 days	Mon 25/07/16
123	Evaluate Tenders Received	60 days	Mon 19/09/16
124	Award of Construction Contract	0 days	Fri 09/12/16
125	Construction of Access Roads and Bridge	240 days	Mon 12/12/16
126	Construction of Power Lines	240 days	Mon 12/12/16
127	**LALINI OPERATIONS ACCOMMODATION VILLAGE/VISITORS CENTRE**	**530 days**	**Mon 05/10/15**
128	Procurement of Detailed Design, Tendering & Supervision PSP	40 days	Mon 05/10/15
129	Award of Contract and Mobilize PSP	60 days	Mon 30/11/15
130	Information Gathering and Review Period	10 days	Mon 22/02/16
131	Supplementary Survey of Works Areas	20 days	Mon 07/03/16
132	Detailed geotechnical and materials investigations	40 days	Mon 04/04/16
133	Detailed design of works	90 days	Mon 04/04/16
134	Preparation of Tender Drawings	90 days	Mon 04/04/16
135	Preparation of Bidding Documents	90 days	Mon 04/04/16
136	Invitations to Tender and Tender Period	40 days	Mon 08/08/16
137	Evaluate Tenders Received	60 days	Mon 03/10/16
138	Award of Construction Contract	0 days	Fri 23/12/16
139	Construction of Works	210 days	Mon 26/12/16



Task | Split | Milestone | Summary | Project Summary | External Tasks | External Milestone | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Progress | Deadline

DRAFT PROGRAMME FOR MZIMVUBU WATER PROJECT - DETAILED DESIGN AND CONSTRUCTION STAGES - OCTOBER 2014 VERSION

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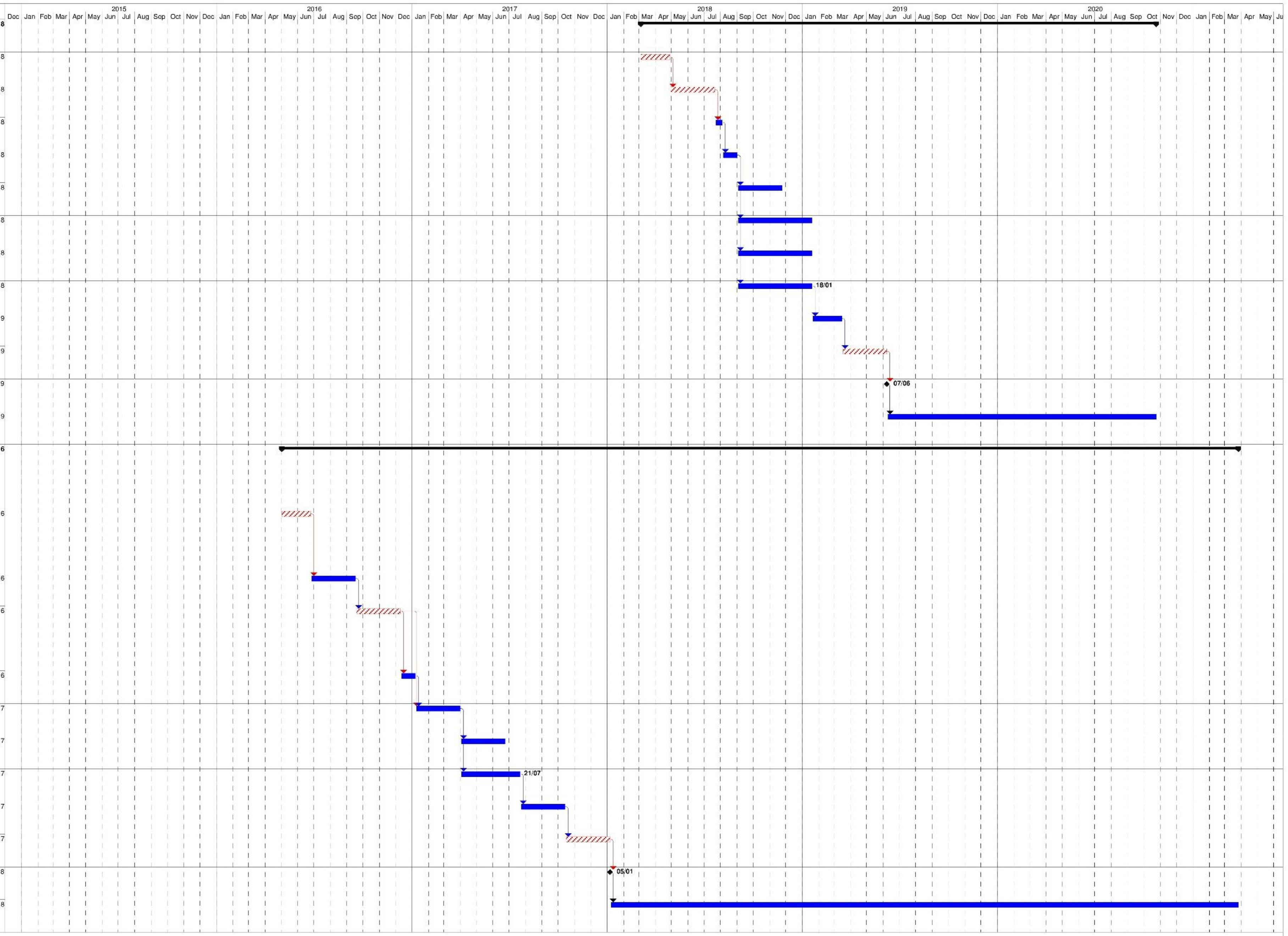
ID	Task Name	Duration	Start
140	**LALINI HYDROPOWER CONDUIT AND TUNNEL**	**690 days**	**Mon 05/03/18**
141	Procurement of Detailed Design, Tendering & Supervision PSP	40 days	Mon 05/03/18
142	Award of Contract and Mobilize PSP	60 days	Mon 30/04/18
143	Information Gathering and Review Period	10 days	Mon 23/07/18
144	Supplementary Survey of Road and Power Lines	20 days	Mon 06/08/18
145	Detailed geotechnical and materials investigations	60 days	Mon 03/09/18
146	Detailed design of conduit and tunnel	100 days	Mon 03/09/18
147	Preparation of Tender Drawings	100 days	Mon 03/09/18
148	Preparation of Bidding Documents	100 days	Mon 03/09/18
149	Invitations to Tender and Tender Period	40 days	Mon 21/01/19
150	Evaluate Tenders Received	60 days	Mon 18/03/19
151	Award of Construction Contract	0 days	Fri 07/06/19
152	Construction of Tunnel and Conduit	360 days	Mon 10/06/19
153	**HYDROELECTRIC PLANTS AND STRUCTURES : DESIGN AND BUILD CONTRACT**	**1280 days**	**Mon 02/05/16**
154	Prepare Terms of Reference, Request for Proposals, Adverts, and Invite Tenders	40 days	Mon 02/05/16
155	Tender Period	60 days	Mon 27/06/16
156	Evaluation of Tenders and Appointment of Design Manager and Supervision Team	60 days	Mon 19/09/16
157	Information Gathering and Review Period	20 days	Mon 12/12/16
158	Detailed geotechnical and materials investigations	60 days	Mon 09/01/17
159	Final Cost Estimates and Implementation cashflows	60 days	Mon 03/04/17
160	Preparation of Bidding Documents	80 days	Mon 03/04/17
161	Invitations to Tender and Tender Period	60 days	Mon 24/07/17
162	Evaluate Tenders Received	60 days	Mon 16/10/17
163	Award of Design and Build Contract	0 days	Fri 05/01/18
164	Design, Construction and Commissioning Period	840 days	Mon 08/01/18



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