P WMA 12/T30/00/5212/19

water & sanitation

Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

DIRECTORATE: OPTIONS ANALYSIS

FEASIBILITY STUDY FOR THE MZIMVUBU WATER PROJECT

FEASIBILITY DESIGN:
LALINI DAM AND HYDROPOWER SCHEME

OCTOBER 2014

FEASIBILITY STUDY FOR THE MZIMVUBU WATER PROJECT

APPROVAL

Report title:	***Feasibility Design: Lalini Dam and Hydropower Scheme***
Authors:	*A Pepperell, D Ochan, T Speirs, C Brand and various other team members*
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CONSULTANTS: JEFFARES & GREEN
Approved for Consultants:

....	....
S Johnson	*A Pepperell*
Deputy Study Leader	*Study Leader*

DEPARTMENT OF WATER AND SANITATION (DWS)
Directorate: Options Analysis
Approved for DWS:

....	....
M Mugumo	*L S Mabuda*
Chief Engineer: Options Analysis (South)	*Chief Director: Integrated Water Resource Planning*

LIST OF REPORTS

REPORT TITLE	DWS REPORT NUMBER
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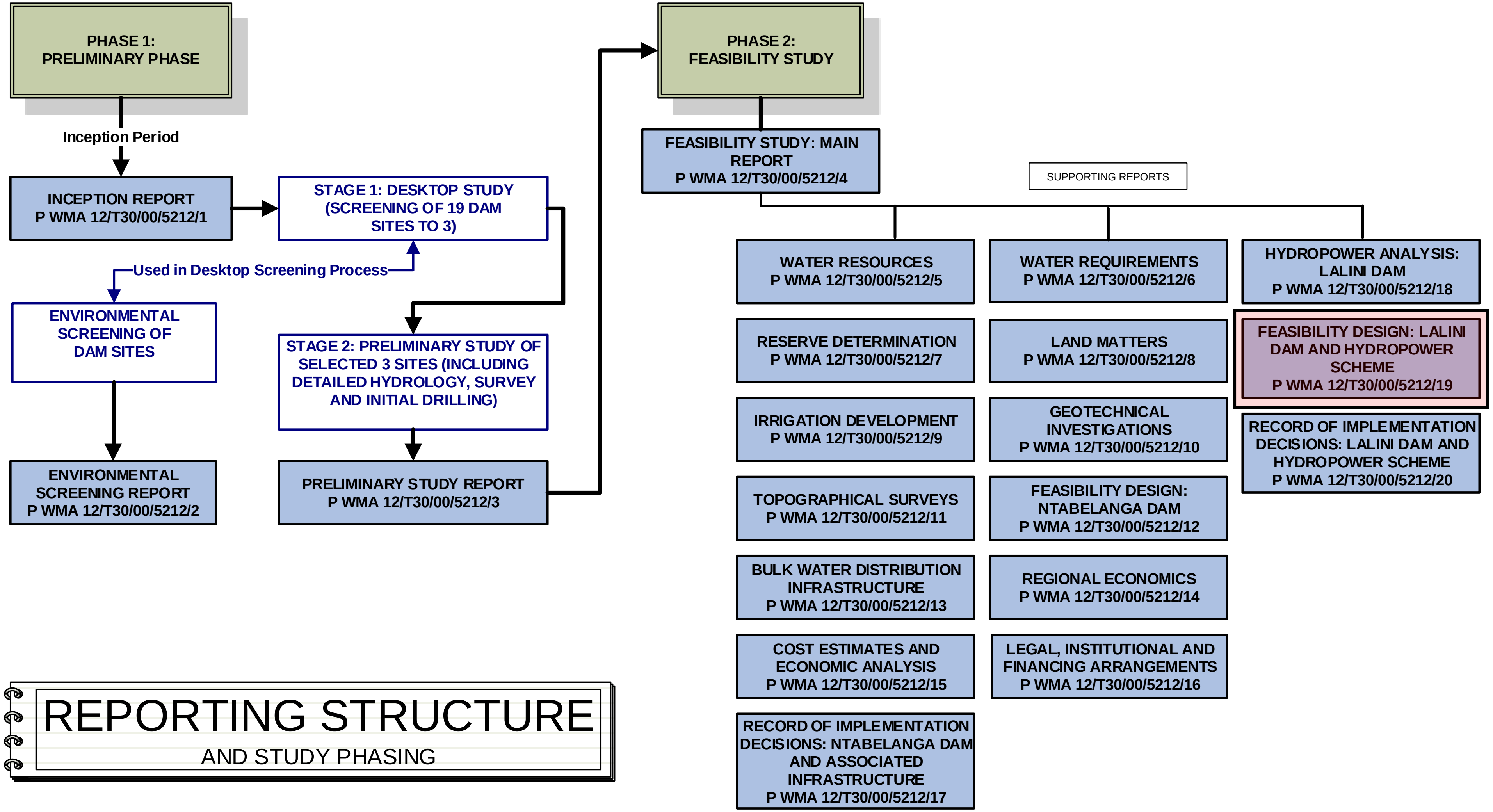


REPORTING STRUCTURE
AND STUDY PHASING

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: Lalini Dam and Hydropower Scheme***

DWS Report No: P WMA 12/T30/00/5212/19

Prepared for: Directorate – Options Analysis

Prepared by: Jeffares & Green (Pty) Ltd, P O Box 794, Hilton, 3245
Tel: 033 343 6700, Fax: 033 343 6701
Contact: Mr A Pepperell
Email: pepperella@jgi.co.za

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-Eastern 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 Feasibility Study 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 is to be completed by October 2014 in several stages as follows:

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

The purpose of this study was 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.

Report numbers P WMA 12/T30/00/5212/2 to 20 describe the feasibility study processes undertaken to prepare solutions that would be implemented to meet the development goals and social benefits.

Following the completion of the above feasibility study stages it was 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 some 726 616 people and other 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, smaller dam at Lalini (located on the Tsitsa River some 3.5 km above the Tsitsa Falls) to generate significant hydropower for supply into the national grid.

The basis of approach listed in item v) was that the generating of hydropower could be used to cross-subsidize the significant energy costs required for pumping water for the irrigation and domestic water supply schemes proposed to be supplied from the Ntabelanga Dam. The agricultural water requirements proposed for the Tsolo area would require lifting the water more than 150 m, which would normally render such a scheme non-viable in terms of the pumping cost component of water supplied, unless hydropower is developed to reduce the net unit cost of water.

The purpose of this second dam and hydropower scheme at Lalini would thus be to generate significant revenue by selling energy into the ESKOM grid, thus generating a net positive income stream which would be used to subsidise the energy and operating costs of the main Ntabelanga water supply and irrigation scheme, thus providing self-sustainability.

A more detailed hydropower analysis and feasibility design study was therefore undertaken to assess the output potential of the Lalini Dam hydropower scheme when used conjunctively with the Ntabelanga Dam. This analysis used the detailed hydrology developed for the catchment and the naturalised and historical flow series that was developed therefrom.

It was confirmed and agreed that the sizing and modus operandi of the Lalini Dam and its associated works would take into account its main role, namely:

i) To generate hydropower both locally at the dam wall and in the Tsitsa River gorge downstream of the Tsitsa Falls; and
ii) To provide sufficient flow of water downstream of the Lalini Dam and these hydroelectric plants (HEPs) to meet environmental water requirements for an ecological Class B/C.

In order to facilitate this analysis detailed investigations were undertaken of the Lalini Dam components of the scheme, inter alia:

- *Detailed topographical survey and positioning of the proposed Lalini Dam;*
- *Geotechnical investigations of the dam site, sources of construction materials, and tunnel alignments;*
- *Investigation of various Lalini hydropower scheme configuration options; and*
- *Hydropower modelling simulations of the Lalini hydropower plant and two mini-hydropower plants at Ntabelanga and Lalini dams for the conjunctive scheme.*

A reserve determination needed to be completed for the Lalini Dam and hydropower plant sites as the hydropower releases can have a significant impact upon the riverine ecology downstream of the proposed dam site and hydropower tunnel exit point.

This included the undertaking of a rapid determination of the EWR of the Tsitsa River downstream of the Tsitsa Falls, which indicated an ecological class of B/C. This EWR value and its recommended rules of operation were included into a new hydropower simulation model to improve the accuracy of estimation of the potential hydropower outputs of the scheme.

This was undertaken as a part of the independent EIA contract and results are given in that suite of reports. Based upon these findings, Lalini hydropower scheme operating rules were developed to ensure that environmental water requirement (EWR) recommendations were complied with, and these rules were discussed and agreed with the DWS Resource Directed Measures (RDM) Directorate.

LALINI DAM LOCATION

The location of a dam site at Lalini had been investigated in previous studies, including the 2004 ESKOM study of "Hydropower Potential in the Eastern Cape". This was further investigated during this feasibility study and confirmed following a site reconnaissance mission.

The preferred site is at a narrowing neck of the Tsitsa River approximately 3.5 km upstream of the Tsitsa Falls, co-ordinates: 31°15'44.76"S and 28°55'15.87"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.

This location also offered several different options for hydropower configurations which are described herein.

Location plans for the Ntabelanga and Lalini Dams are given on Figures 1 and 2.

PURPOSE OF 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 selected Lalini Dam site.

Also, included is the investigation of several hydropower options which includes water transfer conduits/penstocks and hydropower operational configurations (i.e. base load verses peaking options).

DAM TYPE ANALYSIS

The hydropower production modelling used theoretical dam storage capacities from 0.10 to 0.75 MAR_{PD} *(Mean Annual Runoff using Present Day flows), but it was noted that capacities below 0.25 x* MAR_{PD} *produced a poor hydropower yield, and those above 0.6* MAR_{PD} *overtopped watershed terrain, which would require significant additional saddle dams, as well as drowning some major existing infrastructure and settlements.*

The dam type analysis has therefore been undertaken for two alternative dam storage capacities, namely: 0.3 MAR_{PD} *and 0.6* MAR_{PD} *(1* MAR_{PD} *= 828 million* m^3*/a), but with the main focus on the most likely dam size of around 0.3* MAR_{PD}*.*

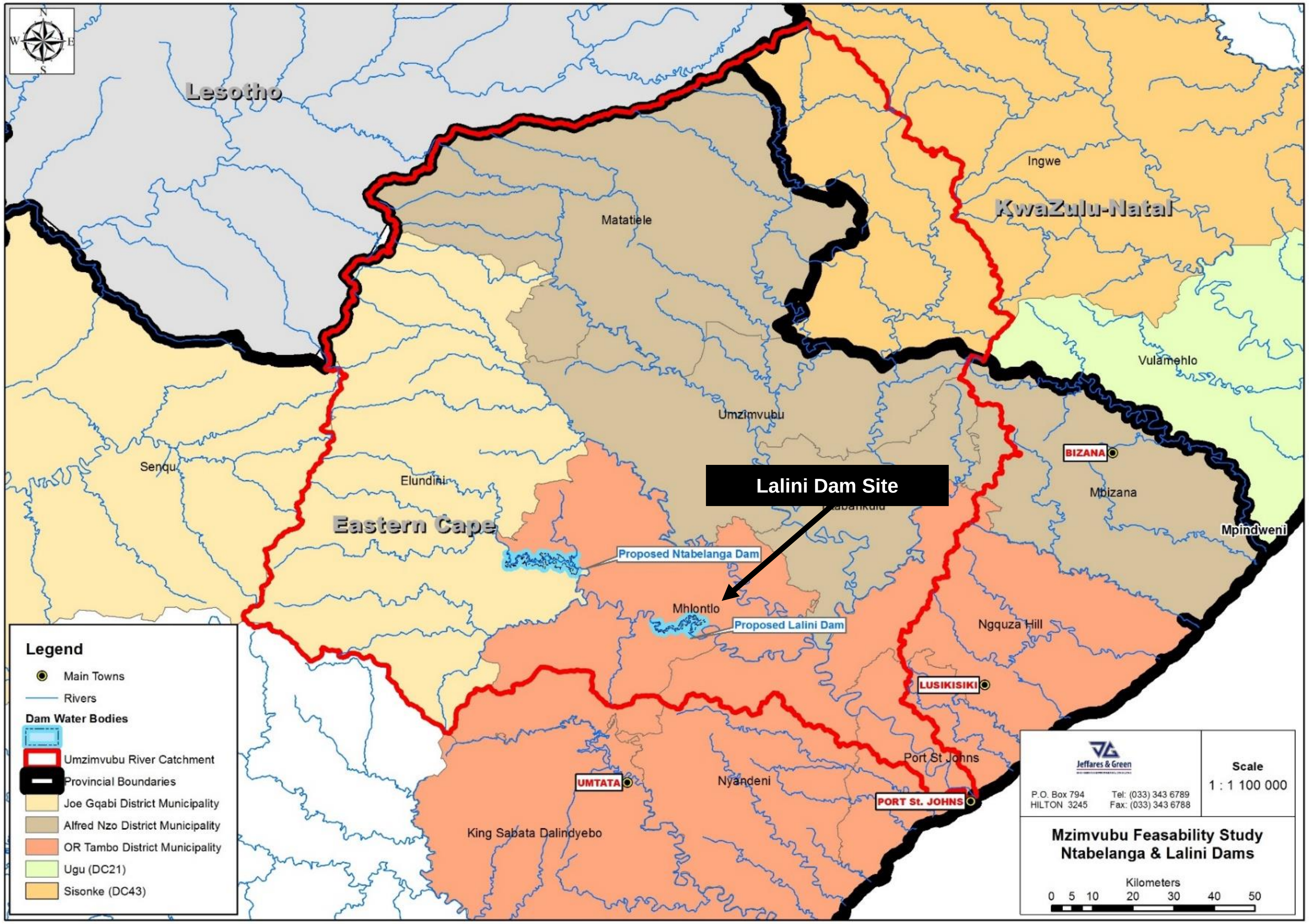


Figure 1: Location of Ntabelanga and Lalini Dams Relative to Overall Mzimvubu Catchment Area

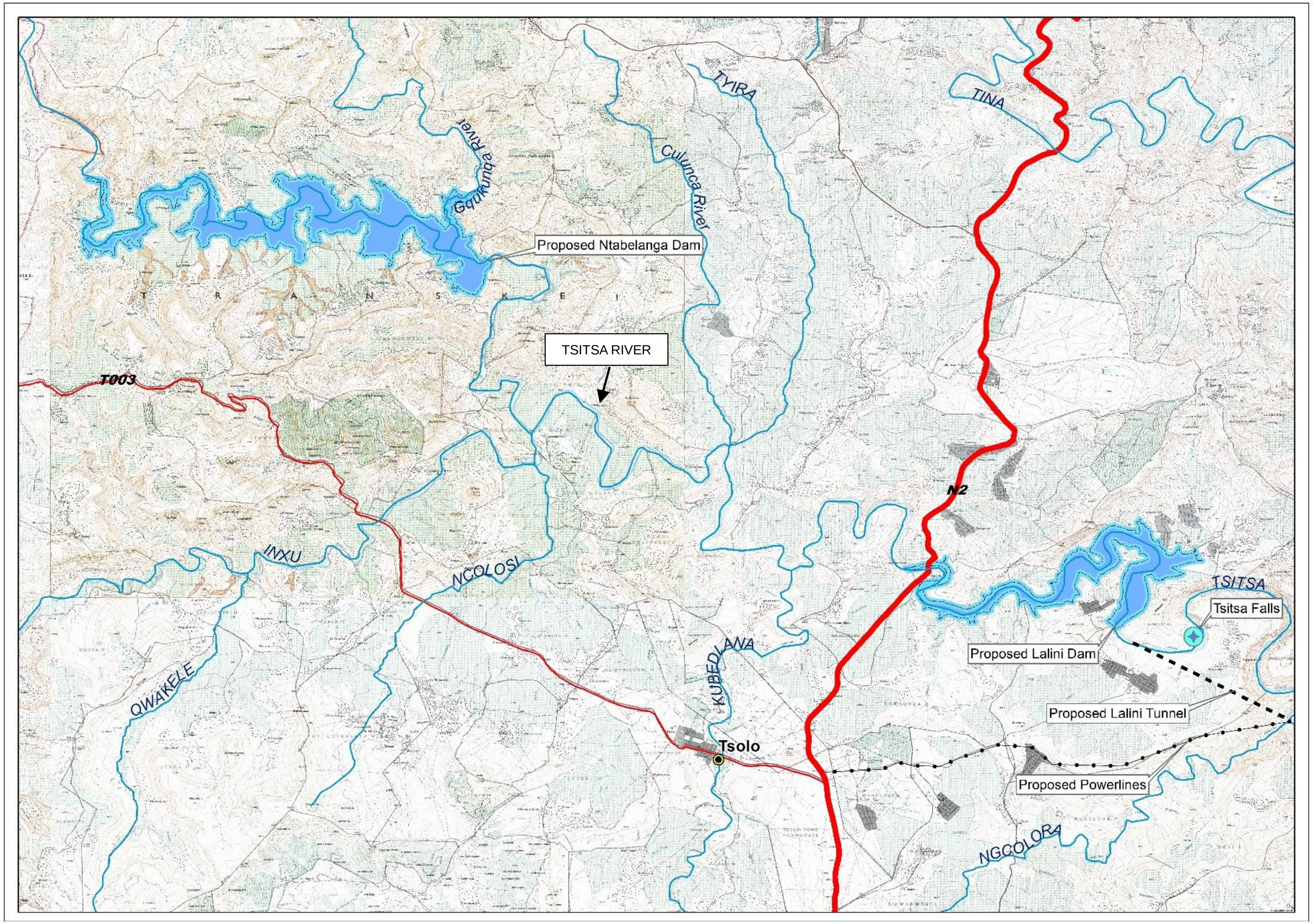


Figure 2: Locality of Lalini Dam Relative to the Ntabelanga Dam

It was deemed important to consider the range of possible dam type options before committing to further core drilling to be undertaken. 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.

Taking cognizance of the approach taken for the Ntabelanga Dam, as well as the observations of the dam site during the reconnaissance mission, the following dam types were investigated

- *Roller compacted concrete (RCC) dam;*
- *Concrete faced rockfill dam (CFRD);*
- *Earth core rockfill dam (ECRD); and*
- *Earthfill embankment dam with earth core (EF).*

Options regarding spillway alternatives of a left bank side channel, cut-through spillway, and in-wall ogee spillway were also investigated.

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

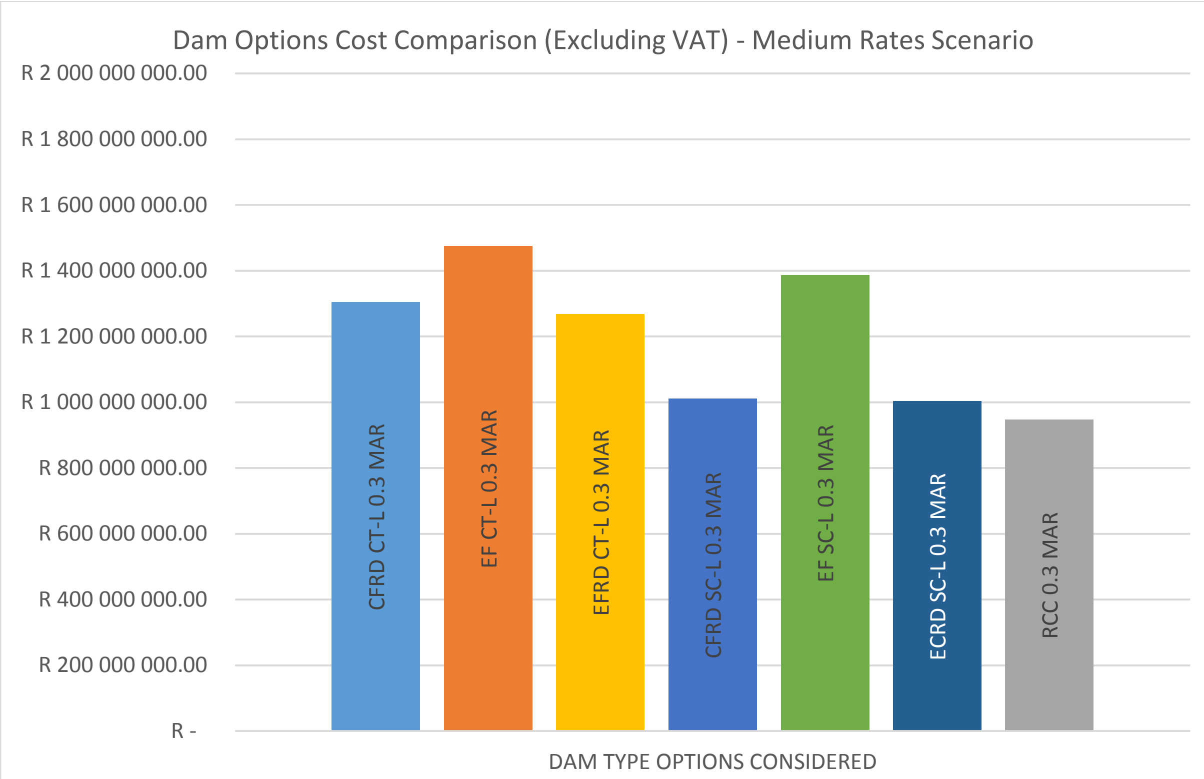
- *Availability of sufficient quantities and quality of construction materials in the vicinity of the dam wall;*
- *Constructability issues, especially relating to dealing with river flow during construction;*
- *Spillway location and capacity requirements;*
- *Operational requirements and outlet works 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 & Spillway Options

Option No.	Dam Wall Type	Spillway Type	Option Nomenclature	Estimated Capital Cost (R'million)		
				Low	Medium	High
1	Concrete Faced Rockfill Dam (CFRD)	Cut-Through on Left Flank (CT-L)	CFRD CT-L 0.3 MAR	1206	1304	1402
2	Concrete Faced Rockfill Dam (CFRD)	Side Channel on Left Flank (SC-L)	CFRD SC-L 0.3 MAR	924	1010	1095
3	Earth Core Rockfill Dam (ECRD)	Cut-Through on Left Flank (CT-L)	ECRD CT-L 0.3 MAR	1178	1268	1358
4	**Earth Core Rockfill Dam (ECRD)**	**Side Channel on Left Flank (SC-L)**	**ECRD SC-L 0.3 MAR**	**923**	**1002**	**1081**
5	Earthfill Dam with Earth Core (EF)	Cut-Through on Left Flank (CT-L)	EF CT-L 0.3 MAR	1385	1475	1564
6	Earthfill Dam with Earth Core (EF)	Side Channel on Left Flank (SC-L)	EF SC-L 0.3 MAR	1296	1386	1475
7	**Roller Compacted Concrete**	**Central Ogee**	**RCC 0.3 MAR**	**826**	**947**	**1069**
				Lowest		
				Second Lowest		

The green highlighted cells show the lowest cost option, which is, for all rate ranges of major quantity unit rates - Option No. 7 – an RCC dam, with Option No.4, the ECRD dam with a Side Channel Spillway cut through the Left-hand Flank, coming second lowest. Figure 3 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	138%	156%	134%	107%	146%	106%	100%
Cost Excluding VAT R'million	1 304	1 475	1 268	1 010	1 386	1 002	947
Total rock excavation used in embankment	1 350 000	17 000	1 100 000	1 350 000	23 000	1 100 000	N/A
Total rock excavation to spoil	3 534 000	5 090 000	3 785 000	1 436 000	2 952 000	1 779 000	N/A
Total all materials to spoil	3 644 000	5 090 000	3 796 400	1 436 000	2 952 000	1 779 000	N/A

Figure 3: 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**, **CFRD (with left hand side channel spillway)** and **ECRD (with left 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;*
- *Simplified infrastructure layout and access;*
- *Low maintenance inputs;*
- *Less risk when dealing with floods during construction; and*
- *Environmental impacts including the aesthetics.*

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, and better aesthetics and less environmental impact than the CFRD or ECRD dam with a side channel spillway options, and would offer the better opportunity for implementation in a shorter time period.

A general arrangement and elevations of the proposed RCC dam solution is given in Figures 4 to 6.

DAM CHARACTERISTICS

The proposed Lalini Dam has the following characteristics:

Full Supply Level (FSL):	*765.58 m.a.s.l.*
Non-Overspill Crest Level[1] – Left flank (NOCL):	*770.41 m.a.s.l.*
Minimum bed level in river at dam:	*717.00 m.a.s.l.*
Crest width:	*6 m*
Minimum operating level (MOL):	*740.14 m.a.s.l.*
Emergency drawdown minimum outlet level:	*735.00 m.a.s.l.*
Maximum dam wall height to NOC:	*53.41 m*
Wall crest length (incl spillway):	*371 m*
Spillway crest length:	*320 m*
Gross stored volume at FSL (0.28 x MAR_{PD}):	*232 million m^3*
Mean Annual Runoff (Present Day) at dam:	*828 million m^3*
Storage below MOL (V_{50} sedimentation):	*31.2 million m^3*
Surface area of lake behind dam:	*14.5 km^2*
Backwater reach upstream of dam:	*22.5 km*
Hydropower transfer conduit/tunnel length	*7.85 km*
HEP location elevation	*445 m.a.s.l.*

The dam wall height, impoundment volume, and downstream risk factors for the Lalini Dam put this structure into a Category III dam under the Gazetted Dam Safety Guidelines. The flood criteria for design of this dam are as follows:

1 in 200 year return period Design Flood:	*3 500 m^3/s*
Safety Evaluation Flood (SEF):	*7 100 m^3/s*

The above dam will provide enough water and effective head required to generate an average of 20 MW and a peak output of 37.5 MW of hydropower as well as providing regulated flow releases in the river below the dam to meet the EWR.

[1] *Right-hand flank NOCL is 1 m higher than this flank*

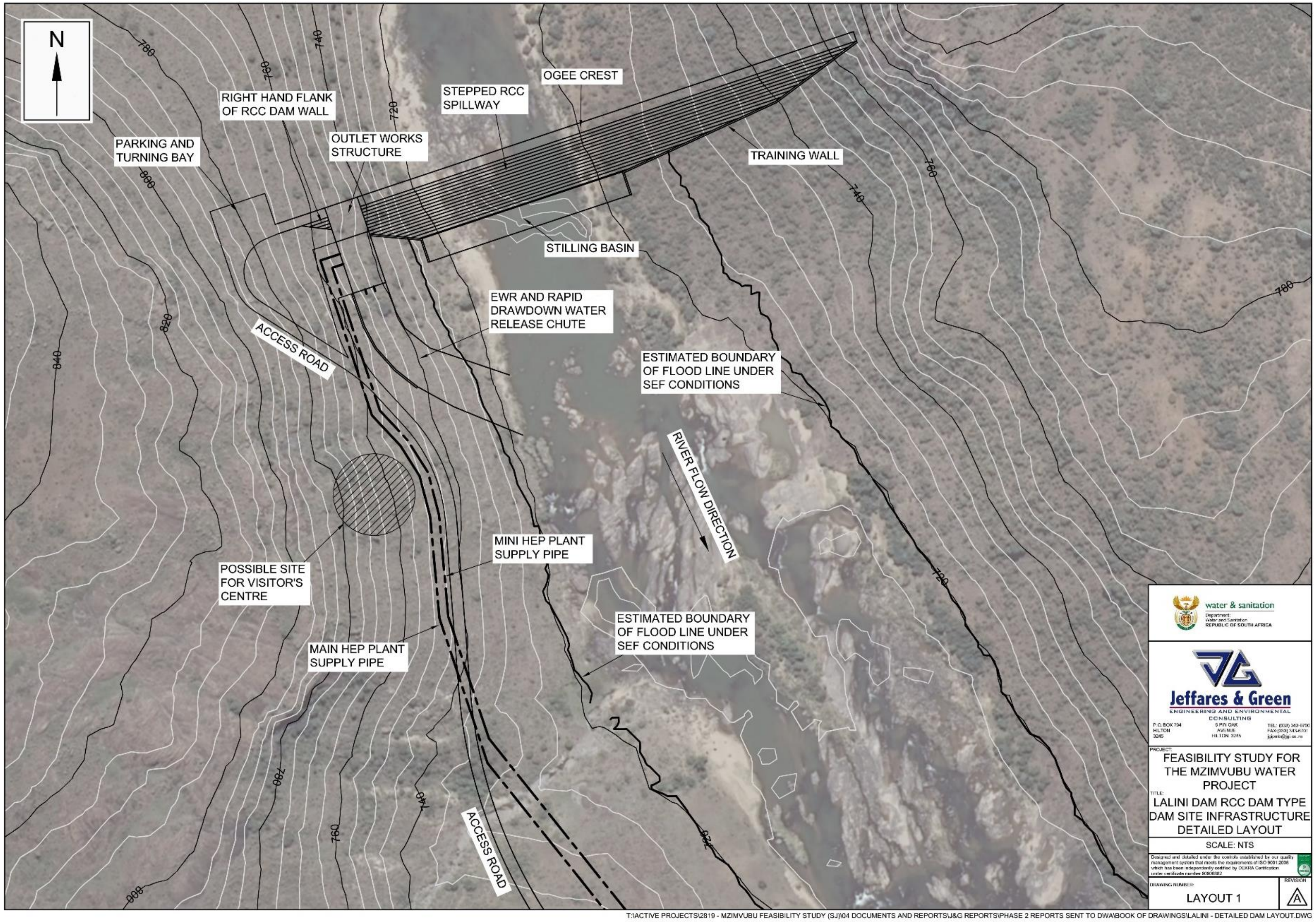


Figure 4: Proposed RCC Dam Layout Plan

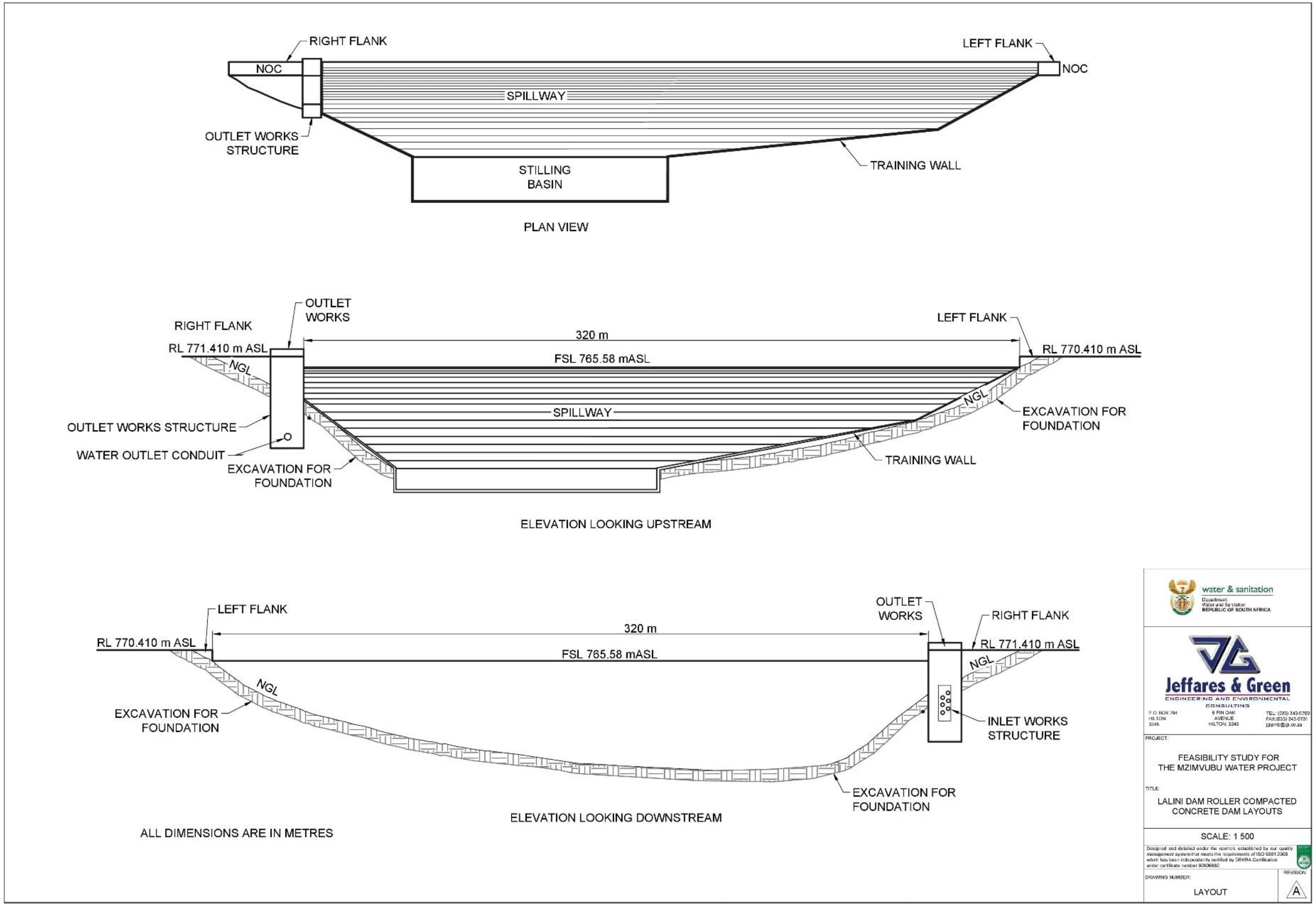


Figure 5: Proposed RCC Dam Elevations

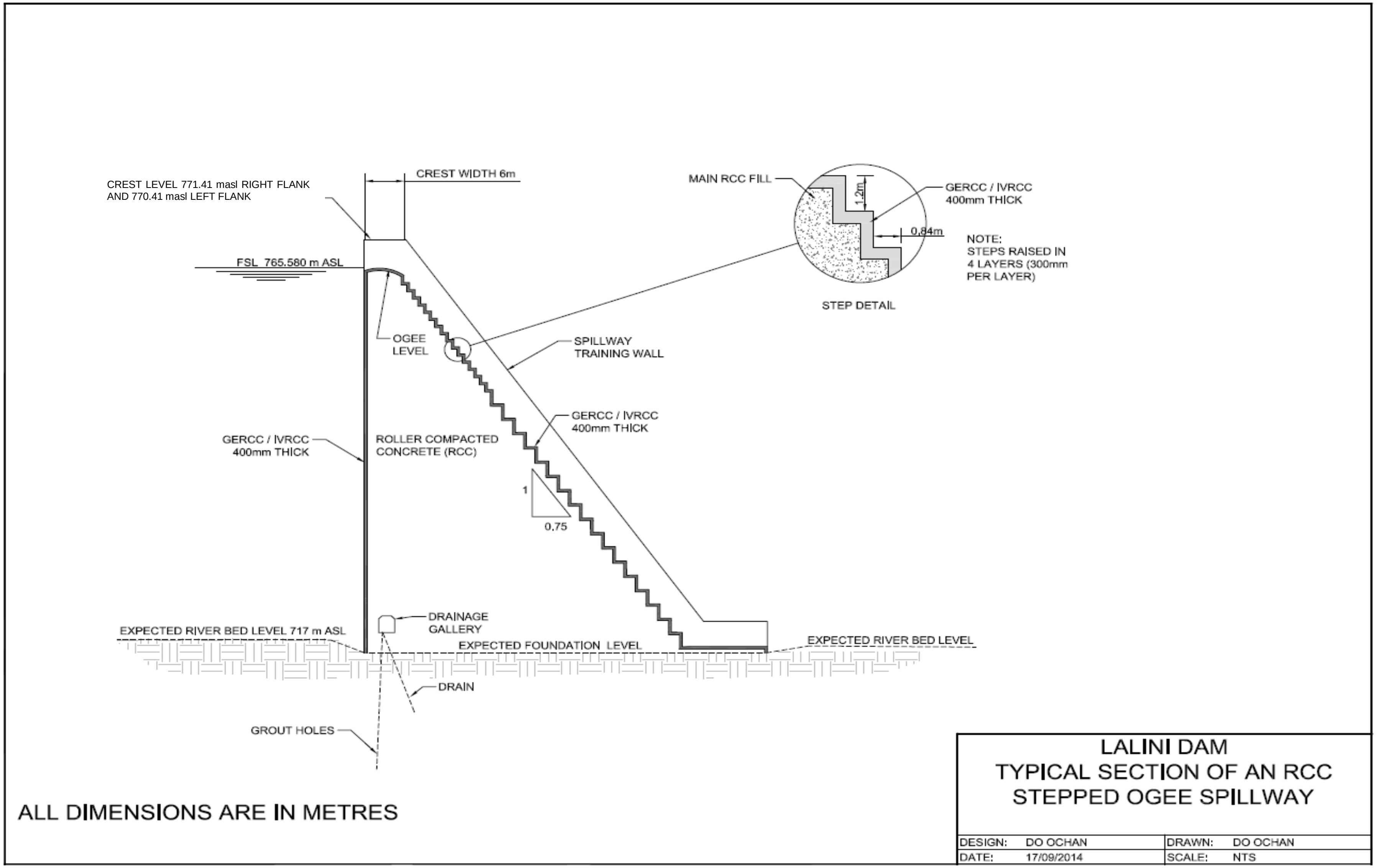


Figure 6: Proposed RCC Dam typical section

FEASIBILITY DESIGN

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

- *Generation of hydropower to be supplied to the national grid; and*
- *Maintaining Environmental Water Requirements (EWR) downstream of the dam;*

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.

EWR RELEASES

The Reserve Determination Report No. P WMA 12/T30/00/5212/7 determines the Environmental Water Requirements (EWR) to be released downstream of the Lalini Dam. This included a basic assessment of the expected EWR at the Tsitsa Falls site.

It was based upon running Water Resources Yield Model (WRYM) hydrological simulations and took into account the expected spills during the same period of simulation.
Additional Reserve Determination investigations were undertaken downstream of the Tsitsa Falls by the EIA PSP, and operational rules were developed for the Lalini Dam to comply with the updated EWR thus developed.

The recommended total releases at Lalini Dam are those required to maintain an intermediate ecological Class B/C of 287.1 million m^3 per annum (i.e. some 33% of MAR_{NAT}), which equates to an average of some 23.9 million m^3 per month.

The EWR is required to be released according to a seasonal pattern and this also depends on whether the river is in a state of flood or drought. EWR release rules are proposed in the reserve determination report, and release criteria are based upon preceding inflows. These operating rules are described in more detail in the Record of Implementation Decisions: Lalini Dam and Hydropower Scheme P WMA 12/T30/00/5212/20.

Given that water released for EWR can also be passed through a mini-hydroelectric plant just downstream of the dam wall, it was decided to consider both EWR and hydropower releases when determining outlet conduit capacity.

HYDROPOWER SCHEME

Typically, the main scheme components would comprise:

- *The Lalini Dam, with inflow supplied by natural runoff from the upstream catchment, as well as both the spillage and the controlled release of water from the Ntabelanga Dam;*
- *Lalini dam outlet works for the conveyance of raw water to a mini-hydroelectric plant (HEP);*
- *Lalini dam outlet works to release water downstream to supply Environmental Water Requirements (EWR), and to rapidly draw down the reservoir in an emergency situation;*
- *A gravity flow raw water conveyance conduit and penstock from the Lalini Dam to the main HEP;*
- *An HEP plant, control and switchgear, and output transformer station; and*
- *Inter-connecting power lines to evacuate the energy into the ESKOM grid.*

The power lines must be constructed as advance works and configured so that they will also supply power from the national grid to the works during the construction period.

Other associated infrastructure to be developed would be:

- *temporary and permanent access roads and servitudes for the construction and operation of the scheme;*
- *new, replacement or realigned roads, power lines, services, buildings, and other infrastructure impacted by the dam and its impoundment;*
- *water supply, power supply and telecommunications to the dam, tunnel, and HEP sites for the construction period and operational stage;*
- *administration and operations buildings;*
- *operations staff housing;*
- *wastewater treatment works for the above; and*
- *solid waste disposal facilities.*

As with the Ntabelanga Dam, the release of water for EWR purposes provides an opportunity for additional generation of power at a "mini"-HEP which could be constructed just downstream of the dam, and this is also considered as an option herein to increase the energy produced by the conjunctive scheme.

A visitor's information centre can encourage tourism and promote economic development by providing visitors with a view of the works and information on the project, including the cultural and tourism activities in the area.

SCHEME OPTIONS

Based upon the hydropower analysis undertaken in Lalini Dam Hydropower Analysis Report No. P WMA 12/T30/00/5212/18, the feasibility design focussed on three Lalini main hydropower options:

- *Base load station: installed capacity 37.5 MW*
- *Base load station: installed capacity 50.0 MW*
- *Peaking station: installed capacity 150 MW*

HYDROPOWER PLANT SIZING

The Hydropower Analysis Report No. P WMA 12/T30/00/5212/18 describes the findings of the modelled hydropower outputs of the Ntabelanga and Lalini Dams when used conjunctively, and recommended an optimum HEP configuration. This analysis was undertaken for the "base load" case of 24 hours/day operations.

The monthly hydropower generating regime is affected by the seasonal variations in river flow, the availability of water in each dam, the operational rules that determine minimum EWR releases at both dams, as well as maximum flow releases at Ntabelanga Dam in the dry season months.

Peaking options have also been considered to determine the cost benefits of operating the scheme to maximize income from energy sales by supplying higher power for fewer hours per day (using the same available daily water allowance) and targeting peak tariff periods.

The recommendations of the cost benefit analysis was to operate the scheme as a base load plant, but to be able to utilize the fully installed capacity for peaking during winter months when prevailing circumstances allow, and if environmentally acceptable.

The result of this was that, for the preferred 0.28 MAR_{PD} Lalini Dam, the HEP plant should have an installed generating capacity of 37.5 MW in the form of 3 x 12.5 MW Pelton wheel turbine generator sets. The resulting hydropower production outputs are as shown on Table 2 and Figure 7.

Table 2: Lalini Main Hydropower Scheme Average Monthly Energy Production

Month	Minimum Target (MW)	Avg HP Output (MW)	Avg Energy Supplied (KWh)
Oct	12.50	18.76	13 959 044
Nov	12.50	23.67	17 043 420
Dec	25.00	22.99	17 102 324
Jan	25.00	21.89	16 283 250
Feb	25.00	23.54	15 963 055
Mar	37.50	24.55	18 268 136
Apr	25.00	22.27	16 035 946
May	12.50	15.69	11 672 893
Jun	12.50	15.83	11 399 591
Jul	12.50	15.95	11 866 003
Aug	12.50	16.04	11 931 220
Sep	12.50	16.46	11 849 343
Total Energy Per Year (kWh)			173 374 226
Average Power (MW)		19.77	

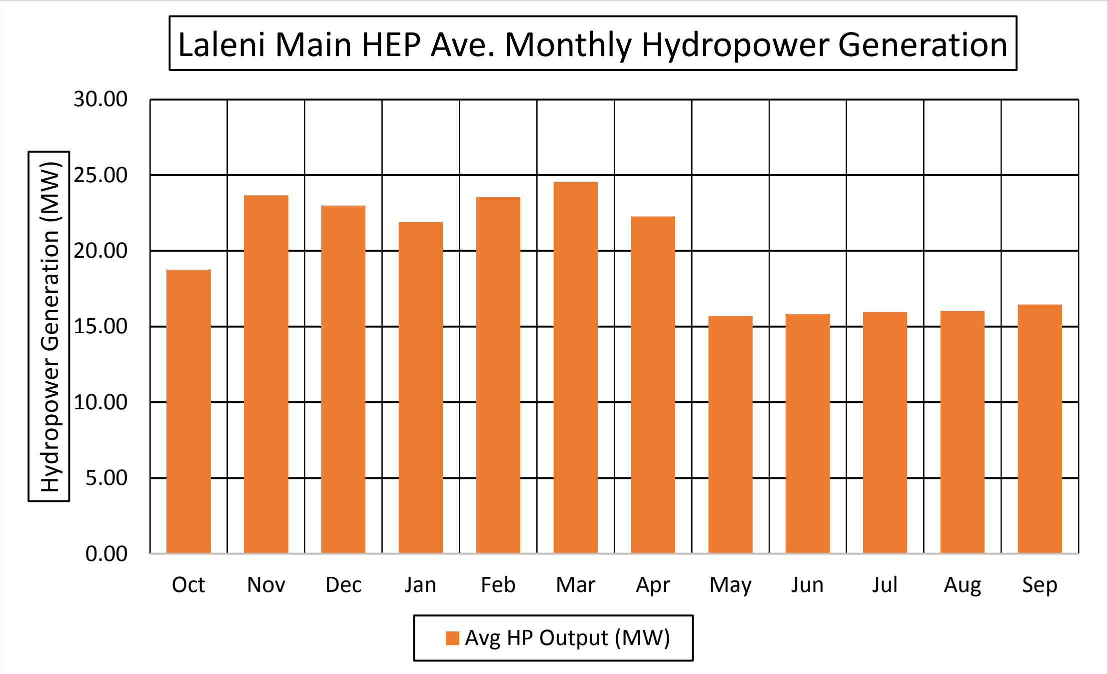


Figure 7: Lalini Main HEP Average Monthly Hydropower Generation

WATER TRANSFER CONDUIT

Following a reconnaissance mission, three hydropower conduit route options and HEP configurations were investigated as shown in Figure 8. After consideration of the advantages and disadvantages of these options, the longer route (Option 3) was selected which had the least environmental and aesthetic impact, an accessible site for the hydroelectric plant (HEP) and the highest generating head which maximises the potential revenue through energy sales.

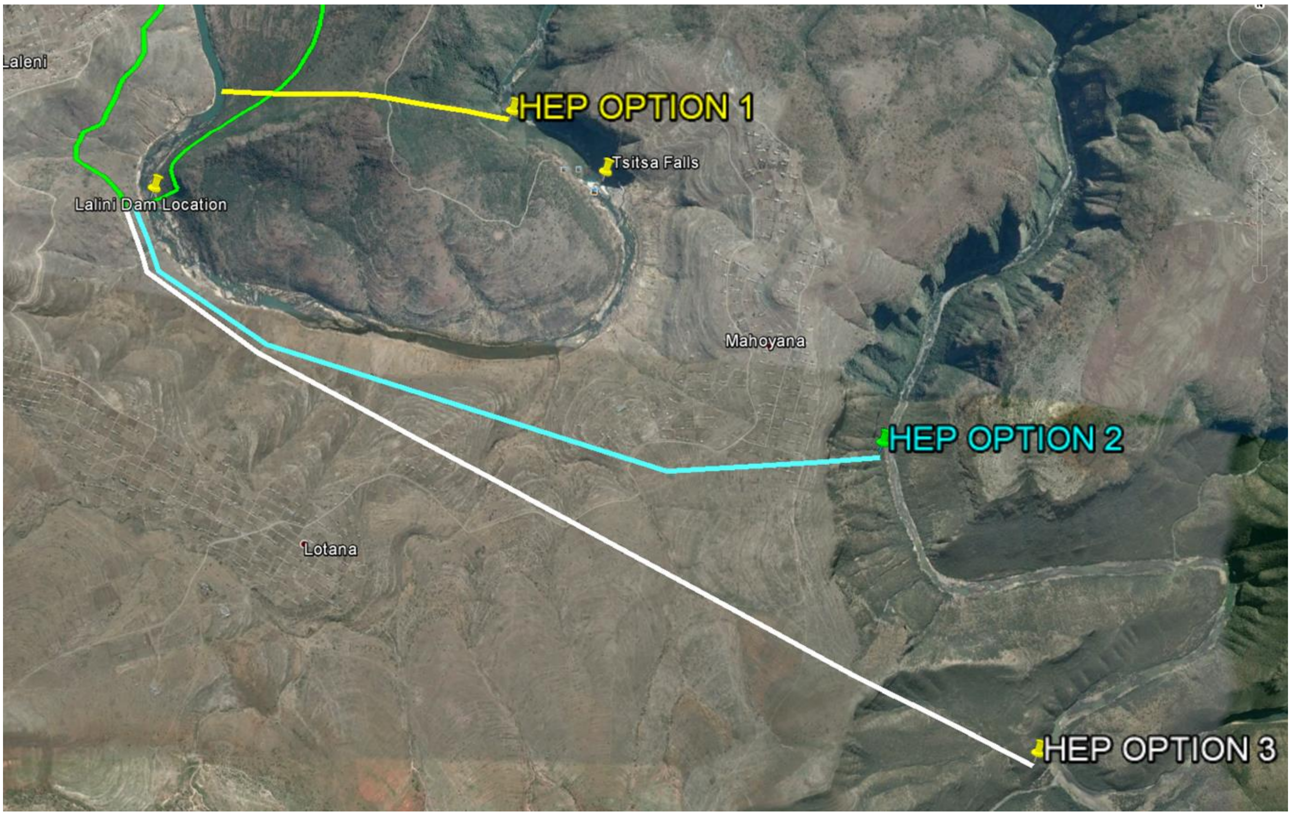


Figure 8: Hydropower Water Transfer Conduit Options

The 7.9 km long conduit routing for Option 3 was optimised once the final Lalini Dam configuration had been confirmed, and was based upon ensuring that gravity flow is maintained at all dam water levels, and pressures are contained within an acceptable working envelope under all operational conditions, which required a surge analysis to be undertaken.

The optimum route required that the conduit pass through an intervening ridge to maintain gravity flow, and this required tunnelling through competent sandstone and dolerite, which was investigated by the core drilling of several boreholes along the planned conduit route.

The eventual solution was to build the first 3.6 km long section of the conduit from the dam outlet to the inlet portal of the tunnel in pipeline laid below ground, and the remainder in tunnel.

The final route and long-section of this solution is shown in Figure 9 (selected solution was the "long-tunnel" solution).

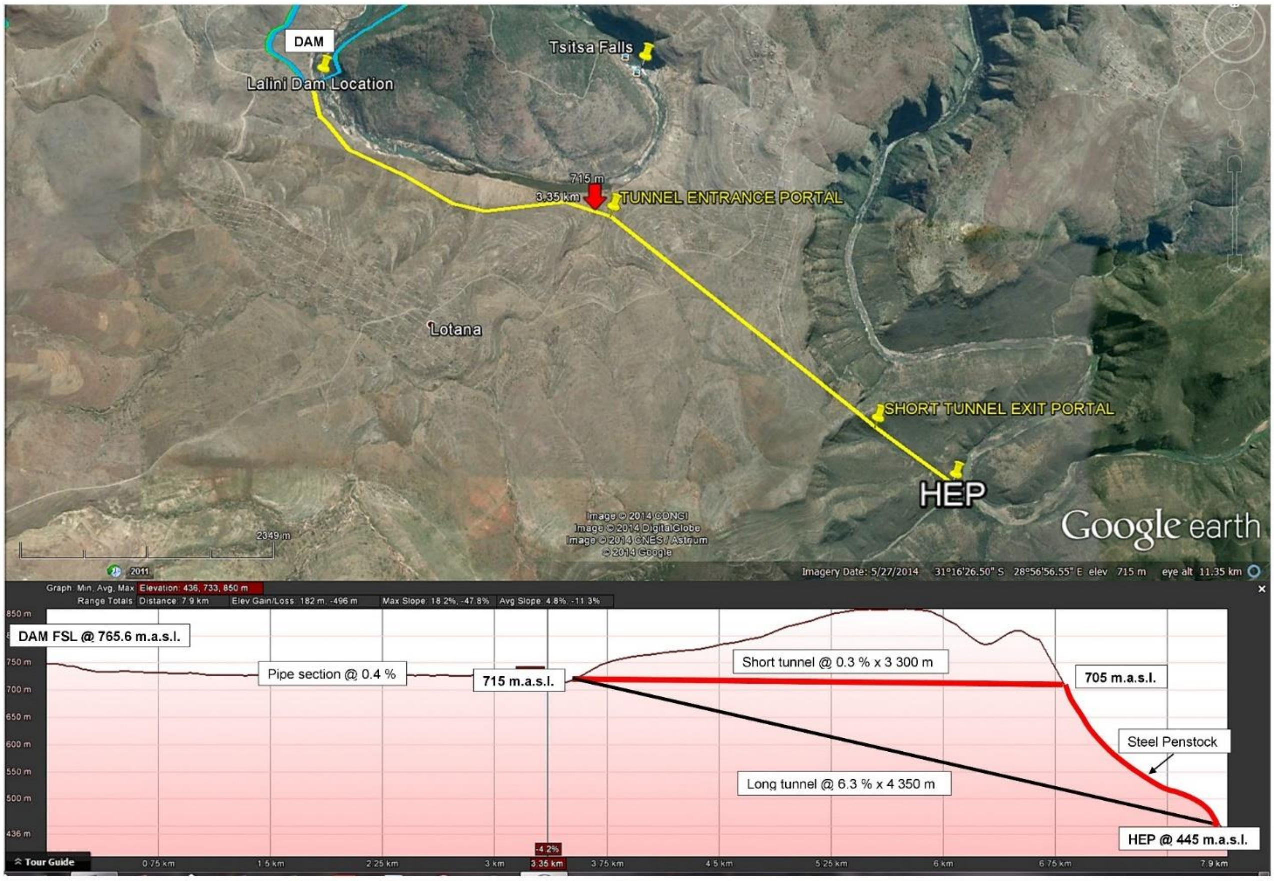


Figure 9: HEP Conduit Horizontal and Vertical Alignment Options

CONDUIT MATERIAL AND SIZING

The selection of conduit sizing was based upon:

- *Hydraulics: to ensure that head losses were minimized to maintain positive minimum pressures and contain maximum pressures under surge condition, and to maximize power production; and*
- *Cost benefits: to ensure that the conduit was economically sized based upon a discounted cash flow analysis for various diameters;*

Options were also investigated as to whether the tunnel section should be a lined pressure tunnel or a dry tunnel with a pipeline laid through it.

Various conduit materials were also considered based upon the expected range of diameters from 2.5 m to 4.5 m (dependent upon the installed hydropower capacity), and the working pressure which ranged from 70 m to 340 m head of water.

The recommended solution is to construct the conduit in welded steel from dam to HEP, with the first 3.6 km laid just below ground and parallel to the river, and the remainder laid on plinths within a dry drill and blast tunnel, which will allow for future inspection and maintenance of the pipeline.

Optimum pipeline sizes for the above three hydropower options are as follows:

- *Base load station: installed capacity 37.5 MW: 2.5 m dia.*
- *Base load station: installed capacity 50.0 MW: 3.0 m dia.*
- *Peaking station: installed capacity 150 MW: 4.5 m dia.*

SUMMARY OF HYDROPOWER PLANT SUPPLY CONDUIT CONFIGURATION

The HEP operational regime rules heavily influence the optimum plant and supply conduit configuration.

Given that the hydropower scheme comprises the conjunctive use of both Ntabelanga and Lalini Dams, the operating rules of both dams as determined by Environmental Water Requirements (EWR) must be considered.

a) Operating Rules – Ntabelanga Dam

This dam release flows down the Tsitsa River into the Lalini Dam and, together with the incremental inflow from the intervening catchment areas, thus supplementing the volume in Lalini Dam that can be utilized for hydropower generation and EWR purposes. In-stream losses are allowed for between the Ntabelanga and Lalini Dams.

The amount of water released downstream from the Ntabelanga Dam would be determined by operating rules which the dam operators will need to follow on a weekly basis. Based upon the recommendations of the EWR studies, the minimum amount released is determined by the monthly EWR with the same percentage occurrence as the measured inflow volume, as is given on the EWR flow duration curve for that particular calendar month. Thus the EWR releases will mimic the prevailing rainfall-runoff conditions in the catchment in any particular month, bearing in mind the historical flow patterns that occurred historically over the 90 year simulation period.

The maximum that can be released from the Ntabelanga Dam is generally limited to the simulated naturalized monthly flow with the same percentage of occurrence as the prevailing inflow as determined from the flow duration curves for that same calendar month. The exception to this is where the dam spills, and no constraints are applied.

During the hydropower generation model simulations it was noted that in extreme drought periods, the EWR volumes released did not always satisfy the hydropower generation needs to be sustained by the Lalini Dam balancing storage. In such cases it was agreed that up to 7 m3/s could be released from Ntabelanga Dam downstream to sustain a minimum hydropower generation output and the EWR requirements at Lalini Dam.

Hydropower generation is achieved at Ntabelanga Dam by using the available head of water in the dam and passing the EWR releases through the mini-HEP located just downstream of the dam wall before returning this flow back to the river. This HEP diversion is limited to 16 m^3/s as EWR flows above this have a low recurrence interval, and it was considered not worth sizing the HEP plant and its conduit for a larger flow rate than this.

b) Operating Rules – Lalini Dam

The monthly inflow balancing regime as described for Ntabelanga Dam was modelled in the same way at Lalini Dam. In this case however, there is no potable or irrigation water requirement, but water is instead diverted through a 7.85 km long conduit to the main HEP located in the river gorge downstream of the Tsitsa Falls, and at an elevation of some 300 m below the Lalini Dam site. This arrangement is shown in Figure 9. The figure shows two tunnel options of which the deeper, direct option is recommended.

The HEP operational regime options are discussed in detail in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15, and the Hydropower Analysis: Lalini Dam Report No. P WMA 12/T30/00/5212/18.

As with the Ntabelanga Dam, the amount of water released downstream from the Lalini Dam would again be determined by operating rules which the dam operators will need to follow on a weekly basis. Based upon the recommendations of the EWR studies, the minimum amount released is determined by the monthly EWR with the same percentage occurrence as the measured inflow volume, as is given on the EWR flow duration curve for that particular calendar month.

In this case the water released from the Ntabelanga Dam would alter the natural Lalini inflow regime, and this will need to be taken into consideration when determining the precedent streamflow conditions in the Lalini catchment when setting the percentage occurrence factor to apply to the monthly flow duration curve, and thus the volume of EWR to be released in any particular month.

Hydropower generation is achieved at the Lalini Dam itself by using the available head of water in the dam and passing the EWR releases through the mini-HEP located just downstream of the dam wall before returning this flow back to the river. This HEP diversion is again limited to 16 m^3/s as EWR flows above this have a low recurrence interval, and it was considered not worth sizing the HEP plant and its conduit for a larger flow rate than this.

The hydropower simulation model always allows for the EWR to be released downstream of the Lalini dam before allowing water to be passed through the main HEP system via the conduit shown in Figure 9.

In order to determine how much water is to be passed through the main HEP plant, a target hydropower output was set for each month of the year. The model allows this to be undertaken quickly and iteratively until the maximum average energy output per year is achieved.

From the results that this produced it was noted that for a base load (24/7 operations) main HEP there was no merit in installing plant of capacity greater than 50 MW and, furthermore, this maximum installed capacity was often only fully useable in the one wettest month of the year.

In addition, in the drier months of the year, it was shown that the maximum power output would drop to around 5 to 15 MW, due to the need to limit the flow rate of water returned back into the river when mimicking the naturalized flow regime, as well as times in drought cycles when both Ntabelanga and Lalini Dams would be at their lowest levels.

If the rule of not exceeding the simulated naturalized flow regime for all months and percentage occurrences is strictly adhered to, then the main Lalini HEP scheme would need to be shut down or operated at a very low output level in a significant number of months in the driest years of operation.

The flow rate required to operate a single 12.5 MW turbine unit continuously is some 6 m^3/s. The operational regime proposed was to therefore make use of the available balancing capacity in the dams to pass a minimum of 6 m^3/s through the main Lalini HEP turbines in the particularly low flow dry season months in order to ensure that a minimum of 12.5 MW can always be produced by the main HEP at all times.

However, when strictly limiting the main HEP flow throughput to the naturalized flow regime, it was evident that the power outputs in dry season months would be low for a significant proportion of the years of simulation, which significantly reduced the average power generated by the scheme.

Modelling was undertaken to determine the quantum of water that would be required to be released through the main HEP extra over the naturalized flow regime values, and the percentage occurrence of when this would be required (e.g. 80% actually means this would only be required 20% of the time).

It was shown that this additional release amount averaged less than 3 m^3/s, but in some drought years could be up to the maximum 6 m^3/s, albeit that this would be a rare occurrence.

Table 3 shows the additional release amounts required per month for various %age occurrence.

Table 3: Water Released Through HEP Extra Over Naturalized Flow to Maintain 12.5 MW

	% age Occurrence of Water Released Over Naturalized Flow (m³/s) to Maintain 12.5 MW Output										
MONTH	**0%**	**10%**	**20%**	**30%**	**40%**	**50%**	**60%**	**70%**	**80%**	**90%**	**99%**
Oct	0.00	0.00	0.00	0.00	0.30	2.25	2.52	2.81	3.33	4.19	4.70
Nov	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.62	3.90
Dec	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.53	1.49	6.00
Jan	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.62
Feb	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.56
Mar	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.39
Apr	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.53	5.27
May	0.00	0.00	0.00	0.00	1.55	2.54	3.39	3.46	3.58	3.76	4.24
Jun	0.00	0.00	0.00	1.04	2.35	3.65	3.54	3.85	3.83	4.19	4.78
Jul	0.00	0.00	0.00	0.87	3.11	3.75	3.77	3.79	4.03	4.18	4.96
Aug	0.00	0.00	0.00	1.37	2.48	3.21	3.77	3.82	4.22	4.43	4.58
Sep	0.00	0.00	0.00	3.78	4.82	5.91	6.00	5.78	5.15	5.08	5.40
AVE	**0.00**	**0.00**	**0.00**	**0.59**	**1.22**	**1.78**	**1.92**	**1.96**	**2.06**	**2.46**	**4.62**

The benefits of this additional release allowance within the EWR rules are that on average, some 10% more power can be generated by the same HEP configuration than if the additional release is not allowed.

This situation was presented to the team undertaking the Lalini EWR study and the consensus was that such releases would not significantly change the ecological regime of the river below the HEP outlet, and therefore could be allowed.

Following review and discussion of the EWR Report, the DWS RDM office has approved the operational regime whereby an additional 6 m³/s over naturalized flow can be passed through the HEP turbines and released back to the river as and when required in any month.

Table 4 shows the resulting average power generated by the man Lalini HEP with this agreed operational regime.

Table 4: Main HEP Power Output with Supplementary Release Through HEP (MW)

	%age Occurrence of HEP Output (MW) - With Supplementary Release										
MONTH	**0%**	**10%**	**20%**	**30%**	**40%**	**50%**	**60%**	**70%**	**80%**	**90%**	**99%**
Oct	37.5	37.5	37.5	19.3	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Nov	37.5	37.5	37.5	37.5	37.1	26.2	18.2	15.7	13.6	12.5	12.5
Dec	37.5	37.5	37.5	37.5	37.5	22.4	18.0	16.9	12.5	12.5	12.5
Jan	37.5	37.5	37.5	37.5	37.5	37.5	25.7	27.0	17.8	13.6	12.5
Feb	37.5	37.5	37.5	37.5	37.5	37.5	33.0	19.5	15.0	18.6	12.5
Mar	37.5	37.5	37.5	37.5	37.5	37.5	37.5	33.1	31.6	19.2	12.5
Apr	37.5	37.5	37.5	37.5	37.5	37.5	23.5	18.8	14.5	12.5	12.5
May	37.5	37.5	23.3	13.7	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Jun	37.5	31.3	18.2	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Jul	37.5	29.3	14.3	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Aug	37.5	37.5	16.1	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Sep	37.5	37.5	14.7	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
AVE	**37.52**	**36.32**	**29.11**	**25.67**	**24.97**	**22.81**	**19.24**	**17.16**	**14.99**	**13.66**	**12.50**

In addition to the 37.5 and 50 MW installed capacity options, a further scenario was also investigated whereby the scheme is operated as a peaking station only. In such a case, some 150 MW of power generation would be installed and operated for a limited number of hours per day to focus only on earning the highest tariff rates. In such a case, the conduit size would need to be 4 500 mm dia.

Costing and economic analysis have been undertaken for these scenarios, and the recommended solution is that of the 37.5 MW installed capacity and a 2 500 mm diameter conduit.

REGULATION OF FLOW BELOW HEP OUTLET

When operated as a base load (24/7) station, there would be no need to regulate the recombined EWR and HEP discharges downstream of the HEP plant outlet, as these would fall within the accepted operating rules determined following the Reserve Determination and EWR studies.

Should the base load (37.5 MW) station be operated as a peaking station in the winter/dry season months, then a typical scenario would be that the full installed capacity turbines were operated over (say) 8 peak hours per day instead of 12.5 MW over 24 hours, thus using the same daily volume of water available.

In order to ensure that the recombined flows are balanced, regulated, and normalized back to a 24 hour regime, a regulating dam and storage facility would need to be constructed in-stream with a minimum storage capacity of 16 hours of the daily HEP flow under the prevailing conditions. In this case, this would require a minimum balancing dam capacity of 375 000 m^3.

Should a full-time peaking station be installed (up to 150 MW), then this requirement increases significantly as the peaking operations would be concentrated to 3 to 5 hours per day, and the balancing storage requirement would rise to as high as 2 million m^3.

For the former option, this balancing storage would extend approximately 500 m downstream of the HEP discharge location, and for the latter peaking option this body of balancing storage could extend as far as 1 500 m downstream and require a dam wall height of 15 m or more.

Such in-stream balancing storage would have its own impact on the environment by drowning the river bed flora and fauna at that location and significantly changing its natural state.

It would also be very difficult to adequately regulate outflow rates from this storage.

The storage would also act as a sediment trap and would rapidly lose its capacity to regulate flow.

In conclusion, it is considered to be highly unlikely that such a balancing regime would be practical or environmentally acceptable, and this further supports the conclusion that the most likely solution is the 37.5 MW installed capacity and a 2 500 mm diameter conduit, operated as a base load station.

This would still allow for the HEP station to be operated as a peaking station in the winter months in years when the flow regime is not in a drought condition.

MAIN HYDROPOWER PLANT CONFIGURATION

ELECTRO-MECHANICAL EQUIPMENT

Internationally-renowned hydropower plant manufacturers from Europe were consulted to determine suitable hydropower generating plant types, design details, performance, costs, installation requirements and general arrangements.

For the 37.5 MW and 50 MW plant options, and the likely monthly generating regime, it was recommended that three or four (net) 12.5 MW units would be best suited to match the head verses flow regime. The basis of feasibility design presented herein is for the 37.5 MW solution.

The turbines proposed are of the vertical Pelton type with 6 jet nozzles. Depending upon the eventual procurement process and manufacturer selected, the number and configuration of jet nozzles could vary.

The proposed arrangement is overhung, i.e. the turbine runner is mounted directly onto an extended and reinforced generator shaft. All remaining (small) axial thrust and radial loadings on the turbine runner created by rotational speed, jet impact and weight are therefore taken by a suitably designed generator shaft/bearing system. The main cooling of the generator is by water cooling and therefore requires a two circle cooling system.

Typical arrangements and a photo of plants of a similar capacity are given in Figures 10 to 13. Please note these are generic examples and not specific to this project.

MAIN HYDROPOWER PLANT STRUCTURE

The structure to house the HEP is designed to meet the functionality requirements of the plant as well as the construction and installation sequencing required for this type of turbine.

A two-stage basement concrete placement is required, and cut-outs in the basement are required to allow operational valves and turbine jet volute casings to be accessed and maintained.

Channels are also included below the Pelton wheel runner to carry the water away from the plant once the jet energy has been absorbed.

Each of these channels must be able to carry a minimum of 6.5 m^3/s, and upon leaving the structure basement, the flow is discharged down the bank of the river via a stepped energy dissipating cascade system founded on good rock and constructed using reinforced concrete and gabion systems.

Specific spacing of each generator is important to avoid interference with each other with respect to both vibration and high voltage current.

This results in a long and narrow building layout as shown in Figure 14. This figure is for a 3 x 12.5 MW turbine solution. If an additional turbine is to be installed, then the building would be proportionately longer.

This building would require adequate lighting, heating, and ventilation and will have a sound-proofed control room at one end.

The generator is the heaviest single component of the generating set, and each would have a weight of some 75 tonnes, with each turbine weighing some 35 tonnes.

The building would be equipped with a suitable overhead crane, and has access doors between each generator set so that transport vehicles can reverse into the building for delivery and replacement of these components.

The HEP building is positioned adjacent to the tunnel exit portal so that the pipeline penstock exiting the tunnel can be connected to the HEP inlet pipework below the hard-standing area.

This site layout and cross-section is shown on Figures 15 and 16.

This shows a diagram of the earthworks and hard-standing areas required between the tunnel and HEP building, as well as the discharge cascades returning hydropower flow back to the river.

This hard-standing platform and access road thereto would be required as a first priority so that the tunnel and HEP building construction can be undertaken.

This will also require a power supply and water supply to be brought to the location for construction and long-term usage.

The water supply would be developed by a package plant abstracting from the river, and the power supply could share the same powerline as would eventually be used to evacuate energy from the HEP into the grid. However, the means of implementing this power supply aspect would be at the discretion of ESKOM.

It is proposed that operators of the HEP would be housed in the same staff housing compound as is to be developed for the Lalini Dam, and would commute via the access road each day.

A small ablution and mess block should be provided at the HEP building.

As shown on the layout diagram, a separate transformer compound is located next to the control room end of the HEP building.

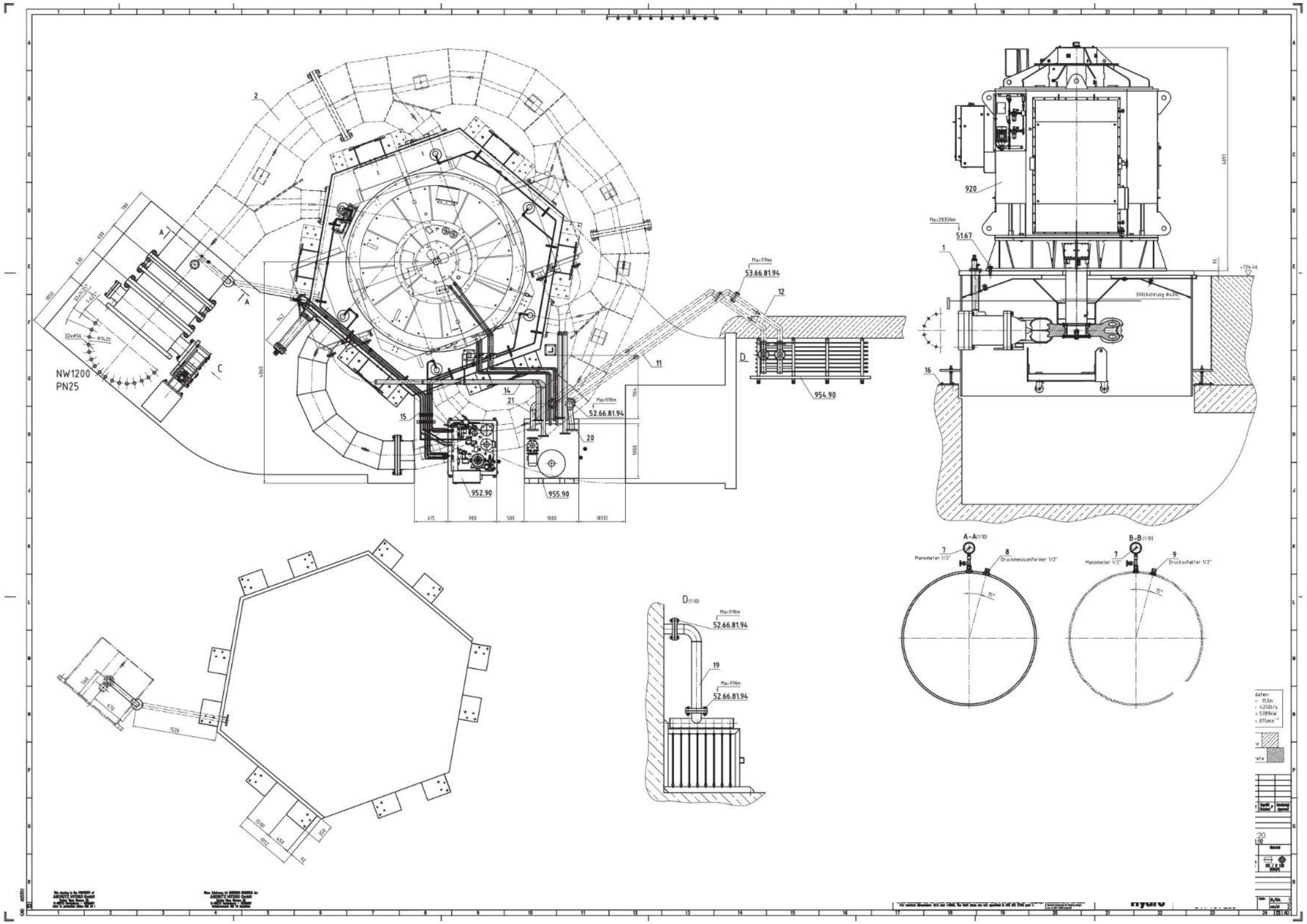


Figure 10: Installation Arrangement of a Similar Pelton Wheel Turbine

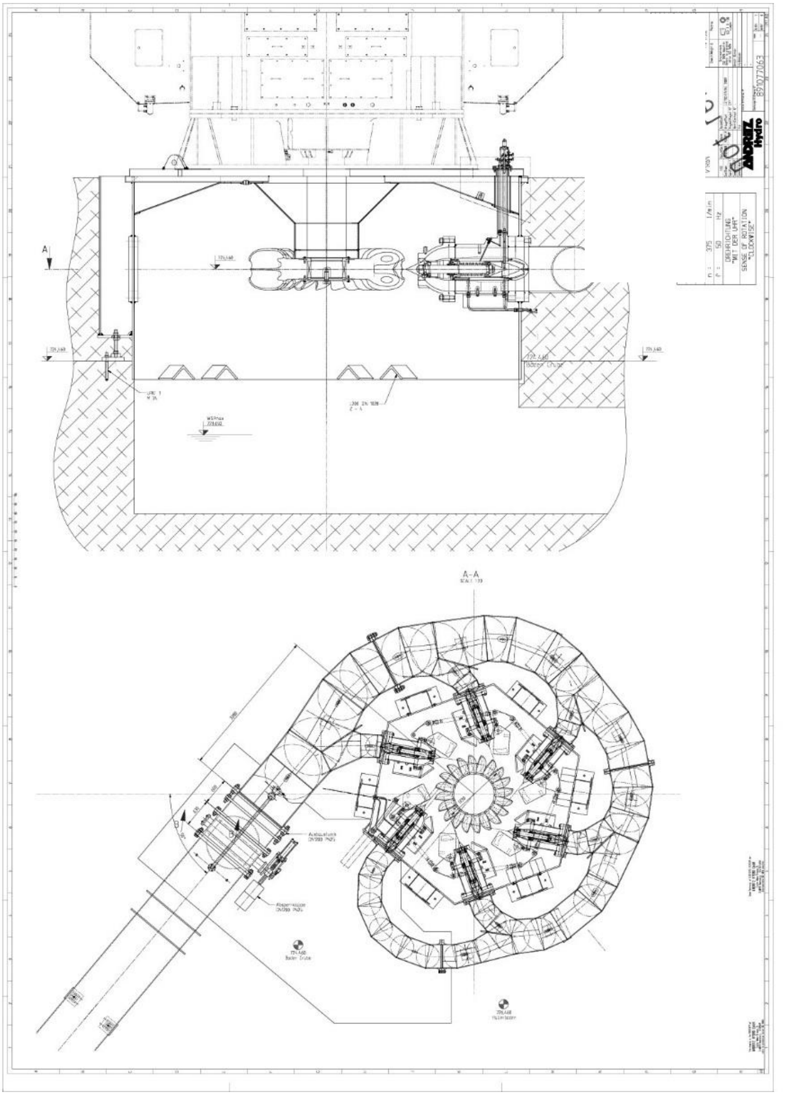


Figure 11: Detail of Pelton Runner and Jet Arrangement

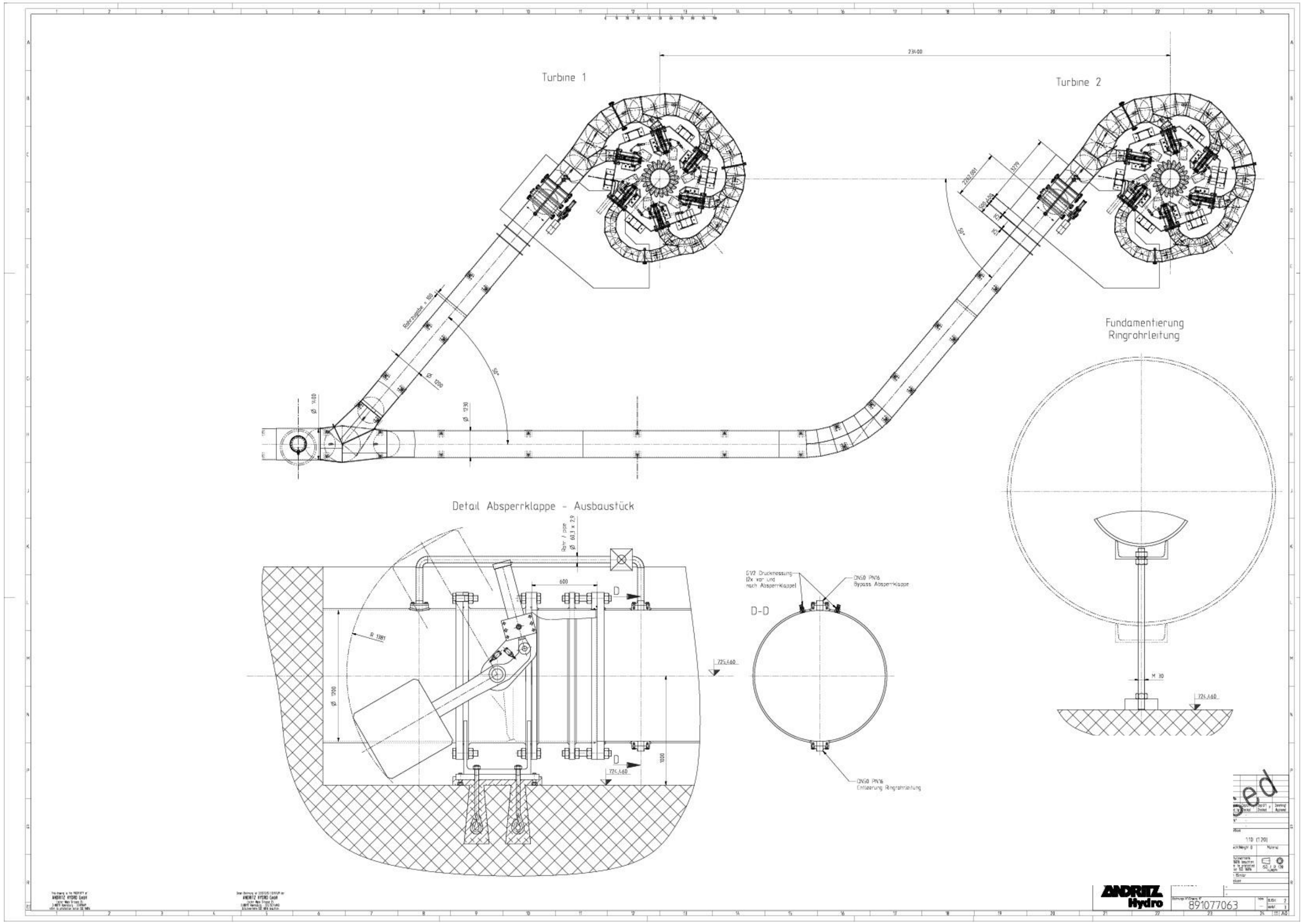


Figure 12: Typical Installation of Adjacent Turbines and Main Control Valve

Figure 13: Photo of Similar Sized Pelton Wheel Generator Installation

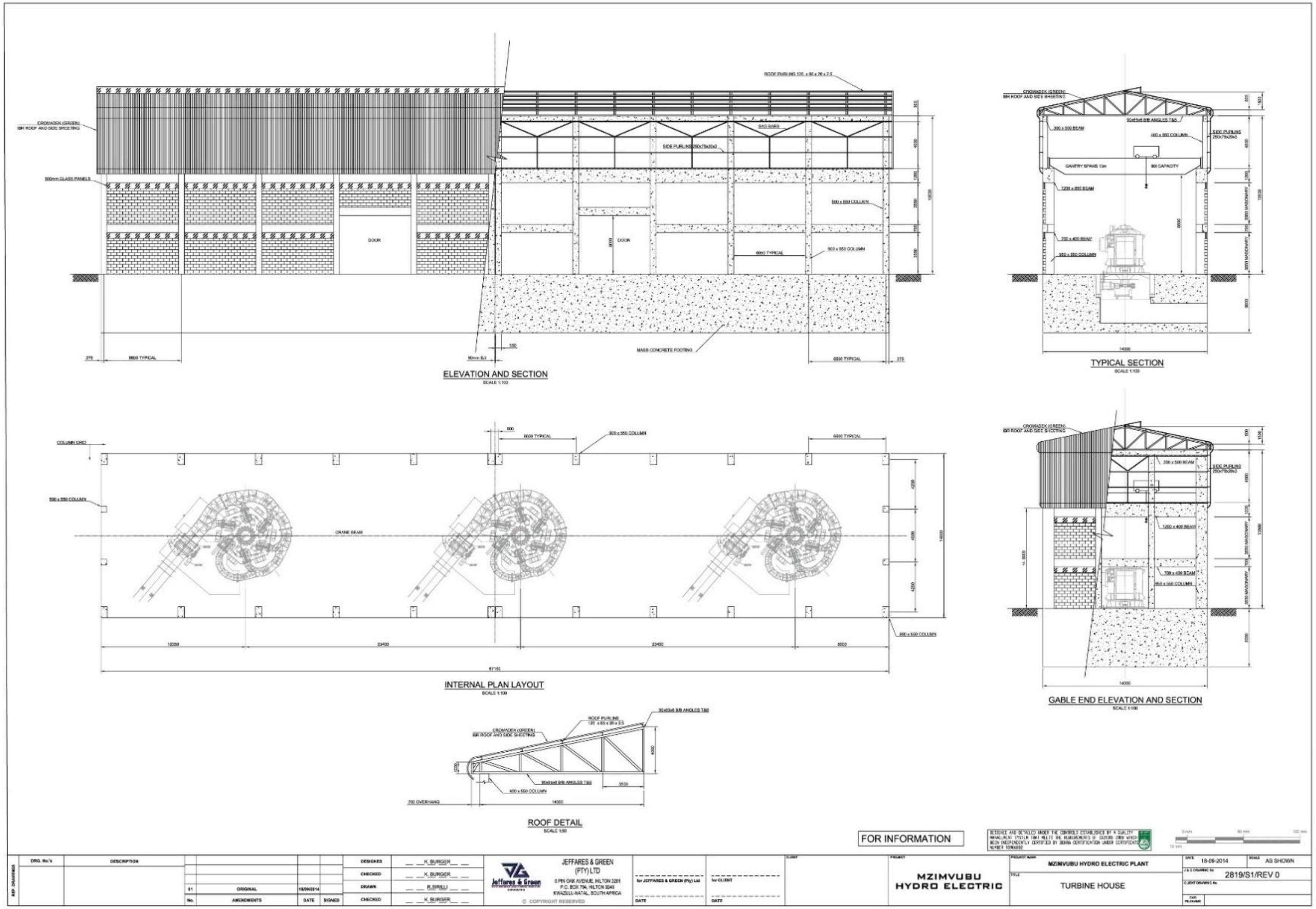


Figure 14: Hydroelectric Power Plant Building (3 Turbine Option)

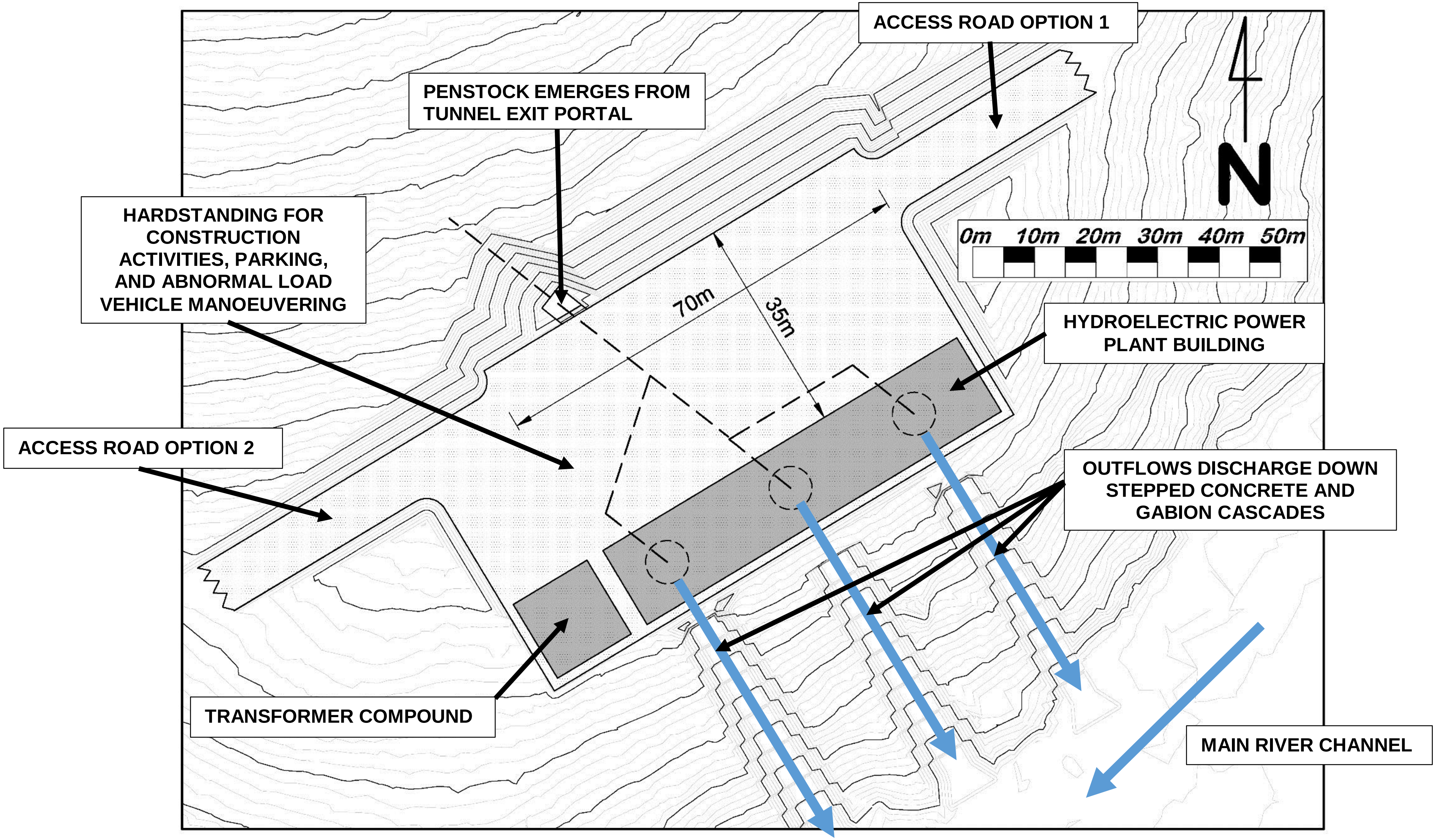


Figure 15: Lalini Main Hydropower Plant Site Layout

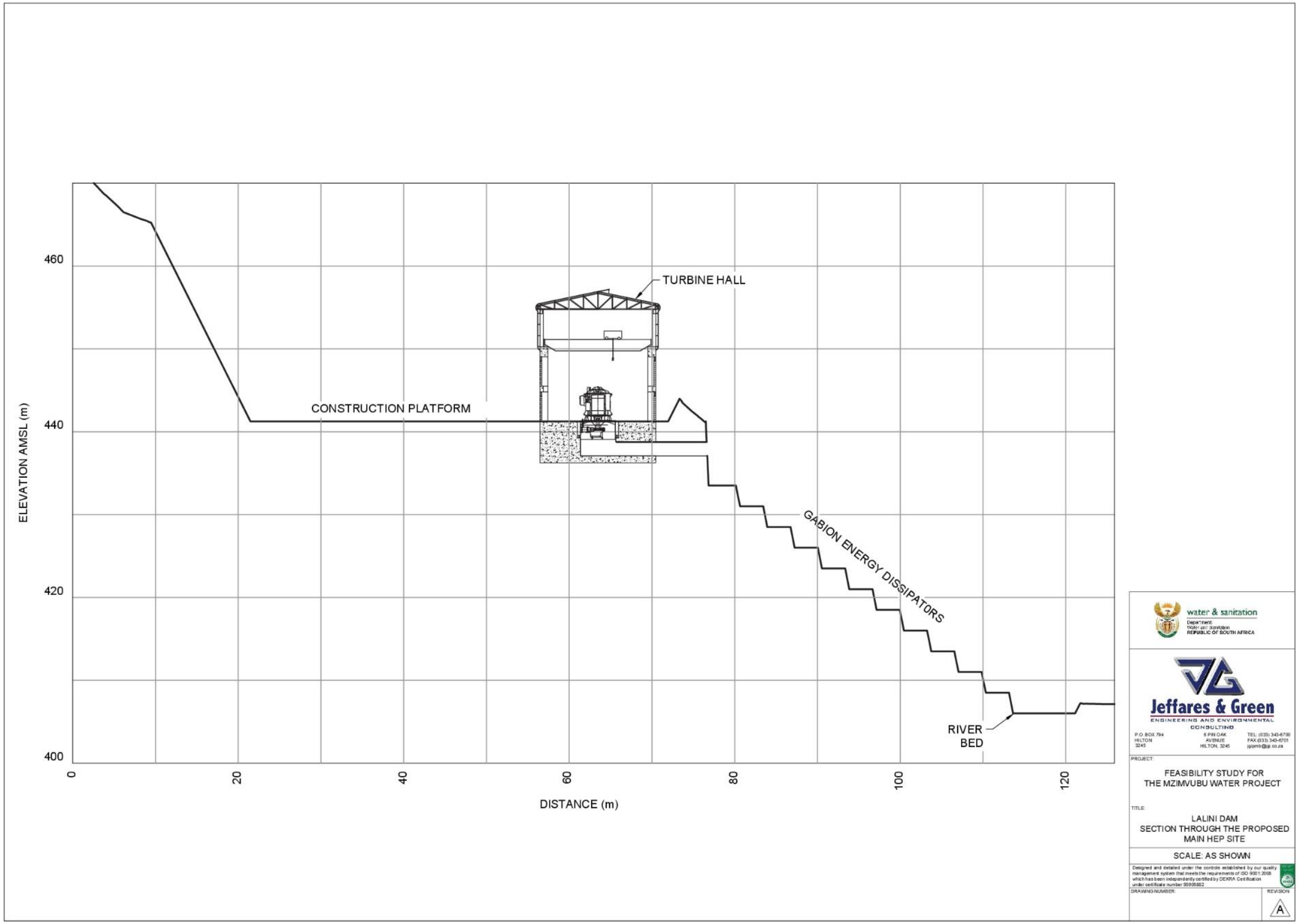


Figure 16: Turbine House and Outlet Works Cross-section

LALINI DAM MINI-HYDROPOWER PLANT

As with the Ntabelanga Dam, the environmental water requirements (EWR) released from the Lalini Dam into the river above Tsitsa Falls creates an opportunity for some additional hydropower to be generated at this location.

The Hydropower Analysis Report No. P WMA 12/T30/00/5212/18 describes the conjunctive scheme hydropower modelling simulations undertaken and indicates that up to 5 MW can be generated in the wetter months, with seasonal availability of EWR determining outputs that can be achieved in other seasons.

The results of the analysis for the 0.28 MAR$_{PD}$ Lalini Dam are as shown in Table 5 and Figure 17.

Table 5: Model Results: Lalini Dam HEP

Month	Minimum Target (MW)	Avg HP Output (MW)	Avg Energy Supplied (KWh)
Oct	2.00	1.41	1 047 895
Nov	3.00	1.74	1 251 338
Dec	3.00	2.34	1 742 819
Jan	4.00	3.10	2 303 120
Feb	5.00	3.90	2 644 895
Mar	5.00	3.91	2 910 565
Apr	5.00	1.74	1 249 716
May	4.00	1.22	905 288
Jun	3.00	0.66	476 106
Jul	1.00	0.59	440 637
Aug	1.00	0.54	401 078
Sep	1.00	0.81	585 678
Total Energy Per Year (kWh)			**15 959 136**
Average Power (MW)		**1.83**	

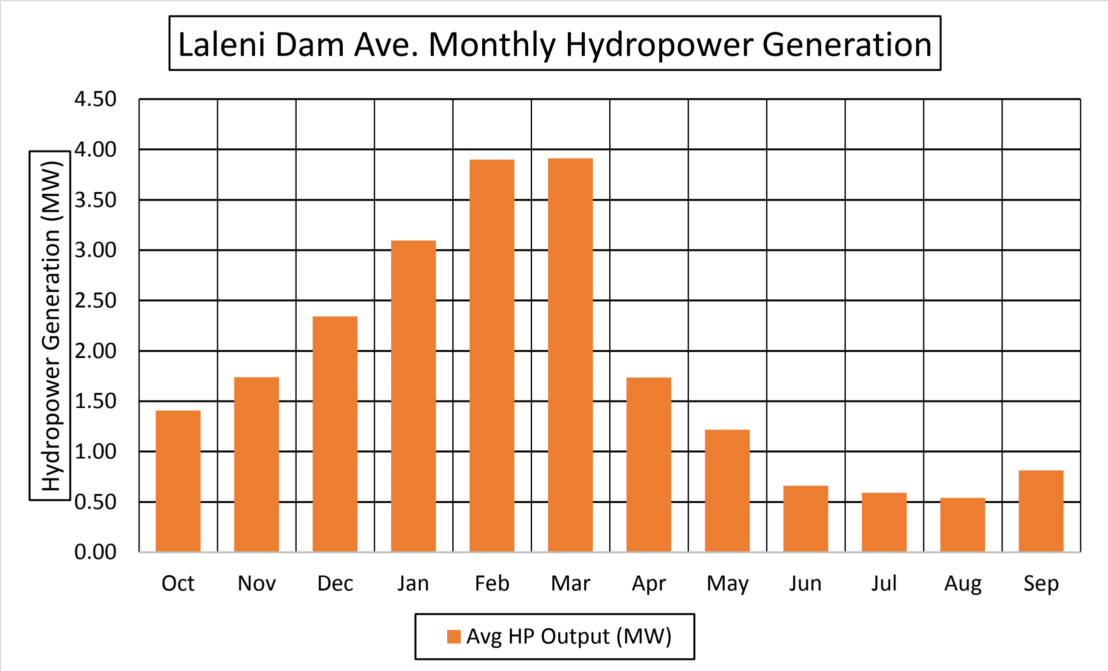


Figure 17: Lalini Dam HEP Average Monthly Hydropower Generation

Thus the hydropower plant configuration has been based upon a target operating range of between 1 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 (blackstart 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, 2 PT100 sensors (1 per bearing) to check bearing temperature and also 2 vibration sensors (1 per bearing).

Typical "Andritz" pump-turbine units suggested were:

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

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

The final decision of which supplier of turbines would be made following a competitive tendering process, and these quoted turbines are only by way of an example.

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

Table 6: Lalini Mini-Hydropower Plant Output Performance

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

Figure 18 shows a proposed layout of the hydropower turbine house together with the inlet and outlet pipework arrangements.

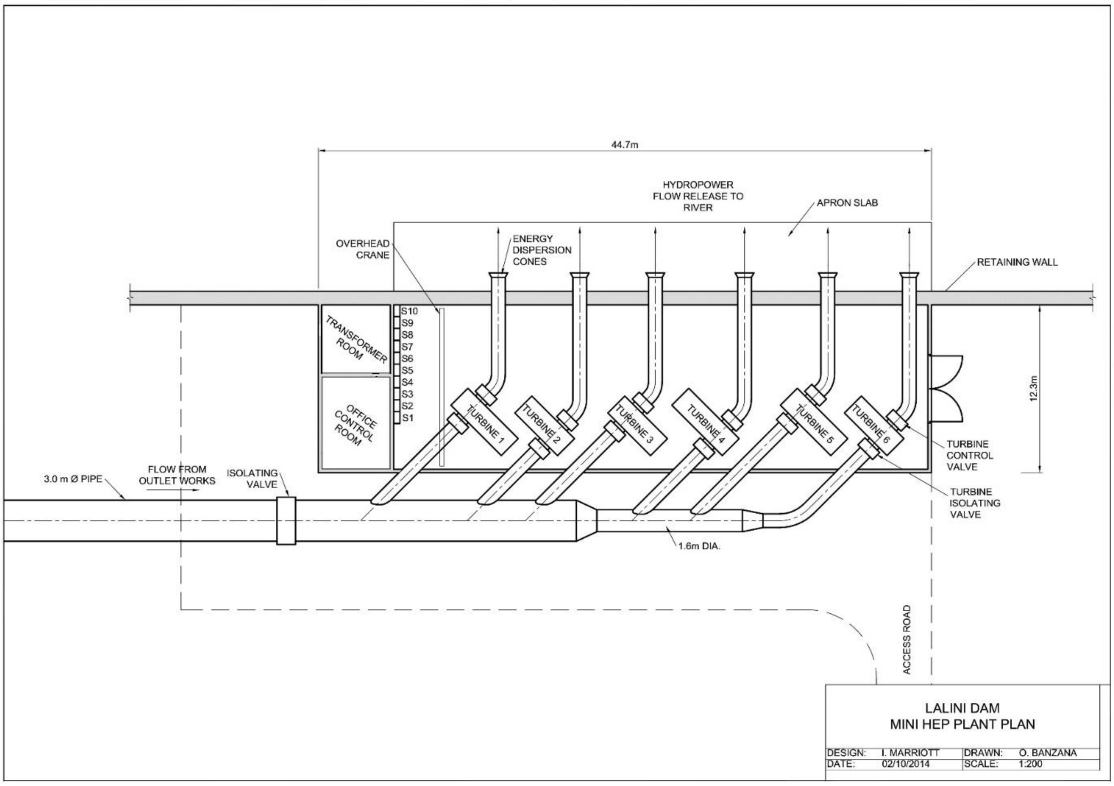


Figure 18: Lalini Dam Mini-HEP Layout

When the hydropower plant is not in use, release of water for EWR purposes can still be made via a sleeve valve in the main dam outlet 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 Lalini Dam are further discussed in detail in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15.

ASSOCIATED INFRASTRUCTURE

CONSTRUCTION AND PERMANENT ACCESS ROADS

Some major road works will be required for the construction and long-term operation of the scheme.

In general, road designs, realignments and upgrades have been designed in accordance with the South African Technical Recommendation for Highways (TRH) standards for such work as detailed in the following documents;

1. *TRH 4 : Structural design of Flexible Pavements*
2. *TRH 17: Geometric Design of Rural Roads*
3. *TRH 20: The Structural Design Construction and Rehabilitation of Unpaved Roads*

MAIN ACCESS ROAD

Figure 19 shows the existing District Road DR 08170 linking the N2 national road near to the Tsolo to Maclear road junction with the villages of Lotana and Lalini in the vicinity of the dam and hydropower infrastructure locations.

This existing gravel road also services the settlements of Madadeni, Gwali, Upper Lotana, Cingcosdwadeni, Ngcolorha, Manzimabi, Mahoyana, and Mbutho.

Figure 19: Main Access Road to Infrastructure Construction Locations

This 17.4 km "Main Access Road" provides the best access to the dam and tunnel construction sites from the main road and does not have any major bridge crossings to contend with. Some donga crossing would need to be widened and upgraded to carry heavy loads.

In addition to construction traffic, this road would be the main route used for the delivery of the heavy electromechanical components of the HEP, which will require abnormal load vehicles able to transport loads of up to 100 tonnes.

Thus it is proposed that this road be upgraded geometrically and structurally to cater for heavy construction traffic and abnormal vehicles that is anticipated to be used in the construction activities. This district road would, however, remain a gravel surfaced road. Provision has been made in the costing to refurbish the upper base courses to a high standard gravel road once construction has been completed in order to ensure that the road is "handed over" to the Provincial Roads Department in an acceptable state.

From this main access road, several new roads will need to be constructed for both construction and permanent access purposes. These are shown on Figure 20.

DAM AND PIPELINE ACCESS ROADS

The 4.2 km roads shown in blue (Figure 20) will be new roads. These roads will be initially established as gravel haul roads for use by normal construction vehicles. However as this will be the permanent main access route to the Lalini Dam and mini-hydropower plant, the road would be upgraded to a double sealed surface, once main construction activities have ceased.

TUNNEL ENTRANCE PORTAL ACCESS ROAD

This 1.3 km road shown in dark green (Figure 20) will be a new road. The road would be constructed as a gravel haul road for use by normal construction vehicles. It will mainly be used during the construction of the tunnel portal section, and during the delivery and installation of the pipeline section within the tunnel. As frequent access to the tunnel in the future would not be required, this could remain a gravel road. However, as this section of road is relatively short it is recommended that this also be upgraded to a double sealed surface, once main construction activities have ceased.

ACCESS TO THE MAIN HEP AND TUNNEL EXIT PORTAL

The access road to the main HEP building and outlet portal of the tunnel is the highest priority road, has exacting requirements in terms of gradients and load carrying, and yet has to traverse the most difficult terrain on the whole project.

This road will be used as the main construction haul link for the tunnel and HEP building construction, and will also be the route along which the abnormal loads are carried when delivering the hydropower electromechanical and transformer components, and for servicing and replacement of such plant in the future. Two options were investigated, and these are shown as HEP Access Road Option 1 (red) and HEP Access Road Option 2 (light green) in Figure 20.

It is recommended that a high specification asphalt road be designed and constructed at the start of the project to provide reliable access to the HEP during construction and operation. For the purposes of this feasibility study, the design is based upon a high grade specification road. This specification and difficult terrain results in an expensive road, but one which would require only low maintenance inputs during construction and in the longer term.

The abnormal load vehicles that will be required to transport the electromechanical plant components would be a multiple axle flatbed horse and trailer arrangement, which has specific requirements in terms of maximum gradients and turning circles.

These requirements determine the geometric standards that must be applied in determining the road design.

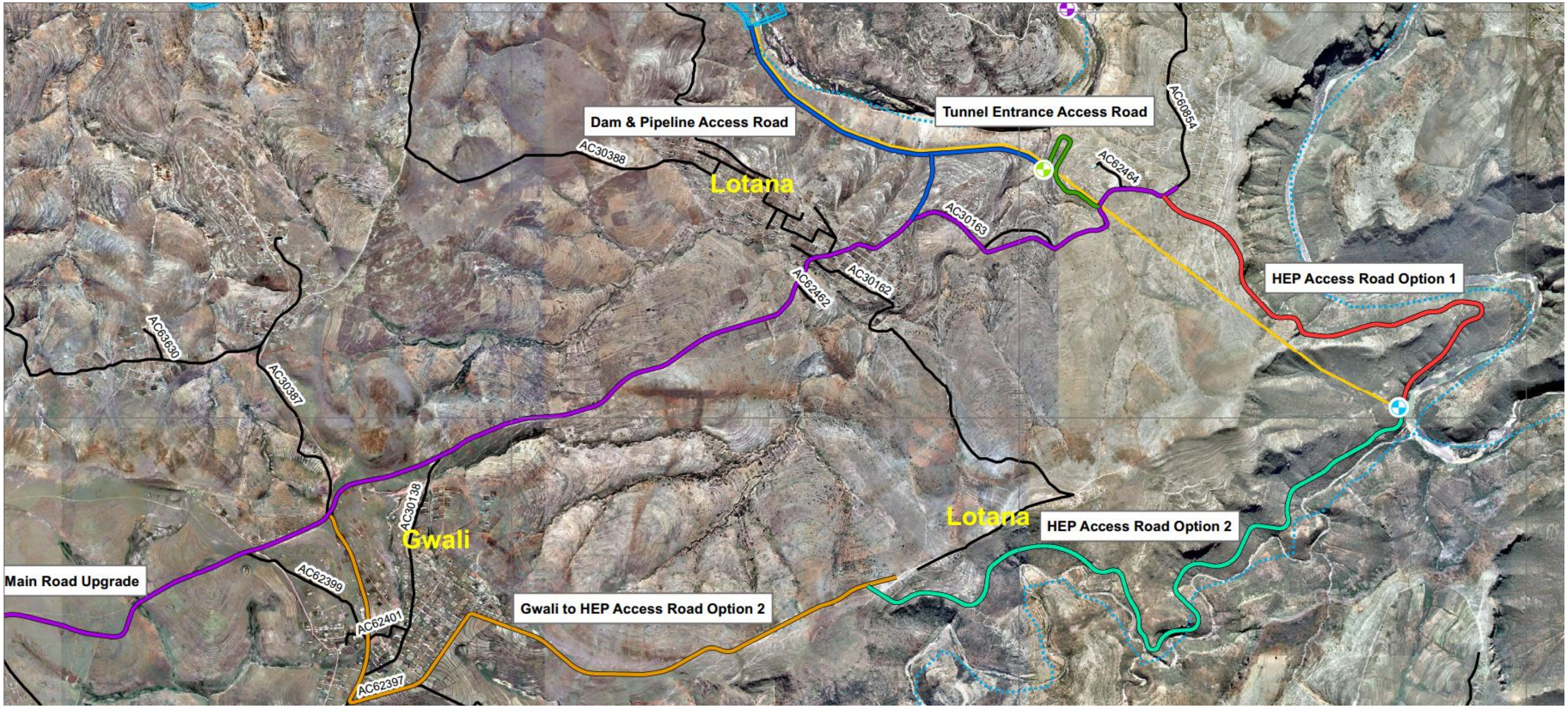


Figure 20: Main Access Road and Other Roads to Construction Sites

GWALI TO HEP OPTION 2 EXISTING ROAD UPGRADE

This 8.2 km long section of road would need to be upgraded if Option 2 were to be adopted. The geometric standards and layer works would be the same as for the Main Access Road.

RECOMMENDATION

At this feasibility design level of study, Option 1 has been adopted as being the cheaper option, but it is recommended that further detailed investigation and optimisation of the HEP Access Road route be undertaken at the detailed design stage.

ROADS AND BRIDGES UPGRADES AND REALIGNMENT

Other major road works will be required to undertake the realignment of infrastructure that will become inundated once the Lalini Dam has been commissioned. The layouts of these are shown on Figure 21.

MTSHAZI MAIN ROAD

The Lalini Dam basin will inundate some existing roads as well as drowning an existing river crossing vehicular bridge which connects the village of Lalini with the settlements of Mtshazi, Shawbury, and the main N2 national road to Qumbu and Mthatha.

District Road DR08167 shown in pink is a tarred road, is the main access from these villages to the N2, and is also a main tourist route for visitors to the Thina and Tsitsa Falls.

This 10.4 km road is in a pot-holed state, and some 40% of the existing alignment will need to be realigned to ensure that it passes outside of the future inundated area.

LALINI BRIDGE RELOCATION

The existing link road from the above Mtshazi road to Lalini village crosses the Tsitsa River via a low level single track vehicular bridge, which was constructed by SANRAL. This carries both vehicular and pedestrian traffic and is the main route for Lalini residents to travel to Mtshazi and Shawbury as well as accessing the main N2 national road. This existing bridge and road will be permanently drowned by the impoundment of Lalini Dam.

Alternative routes were sought to replace this route, which included a new road from Lalini along the south bank of the river and connecting to the N2. Unfortunately this would increase the travelling distance for journeys from Lalini to Mtshazi and Shawbury by some 15 km. This would be highly unacceptable for pedestrians which include children going to school. If this option were adopted, then a high level footbridge would also be required to cater for the pedestrian users, but this would still not be a very acceptable solution as far as additional travel distance and time required by the vehicular road users.

The EIA study team were consulted and it was suggested that in such circumstances the solution should follow the principles of a “like-for-like” replacement (maintaining the status quo). In this case, the nearest and shortest bridge crossing is as shown in yellow on Figure 21. In order to meet the SANRAL standards, the bridge deck soffit would need to be at an elevation such that 1.4 m freeboard is allowed under the 1 in 100 year flood condition. This results in a bridge deck length of some 450 m. The alignment of the new link road and bridge is shown in yellow on Figure 21. A general arrangement of the proposed bridge is given in Figure 22.

A multi-purpose bridge was therefore designed which has a single track vehicular way and a barrier-protected pedestrian walkway. Given the long length of the bridge, the vehicular carriageway has two widened waiting bays for vehicles to pass each other. The bridge must meet SANRAL design standards.

The 4.4 km new link road connecting the bridge to the existing Mtshazi road and to the existing main road into Lalini, would be designed to the same standards and have the same layer works as the district road DR 08167, and would therefore be a tarred surface road.

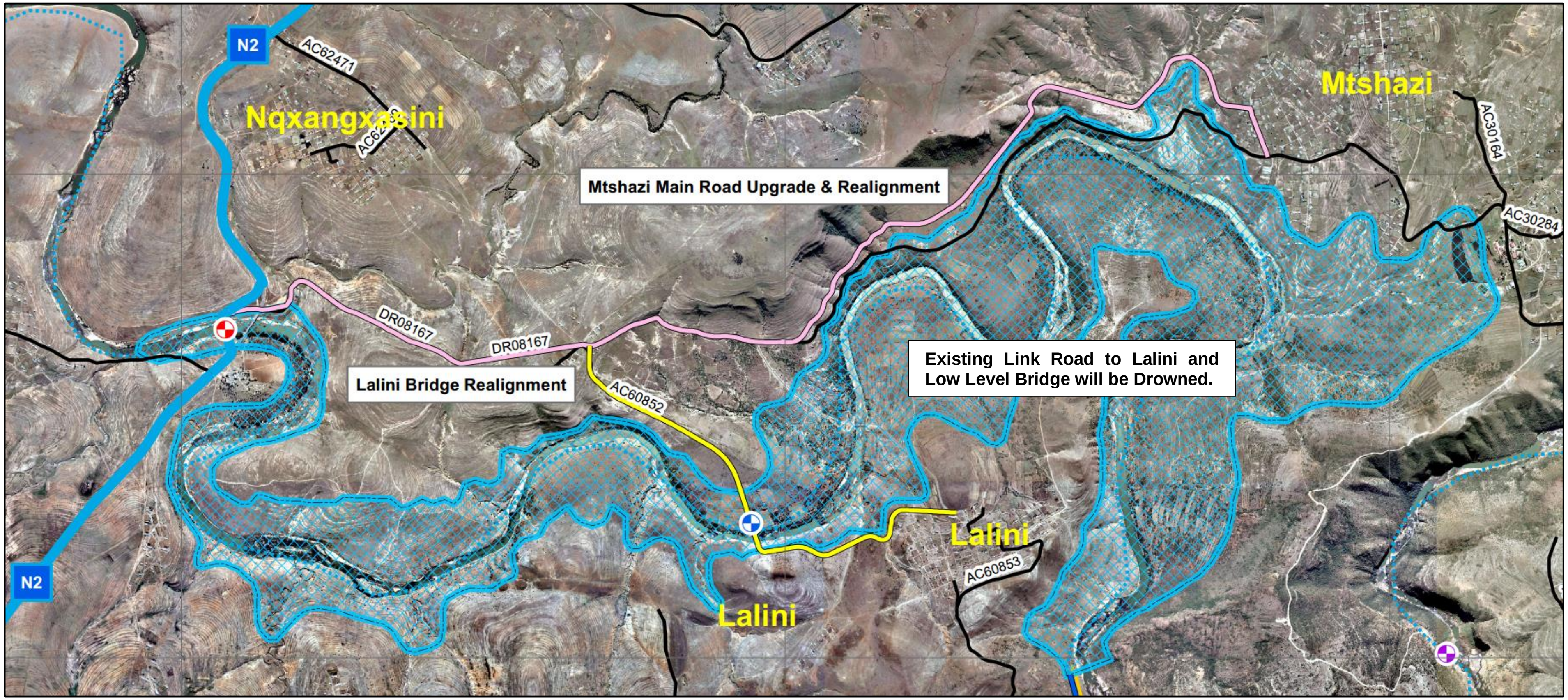


Figure 21: Roads and Bridges to be Permanently Upgraded and Realignment Before and During Construction

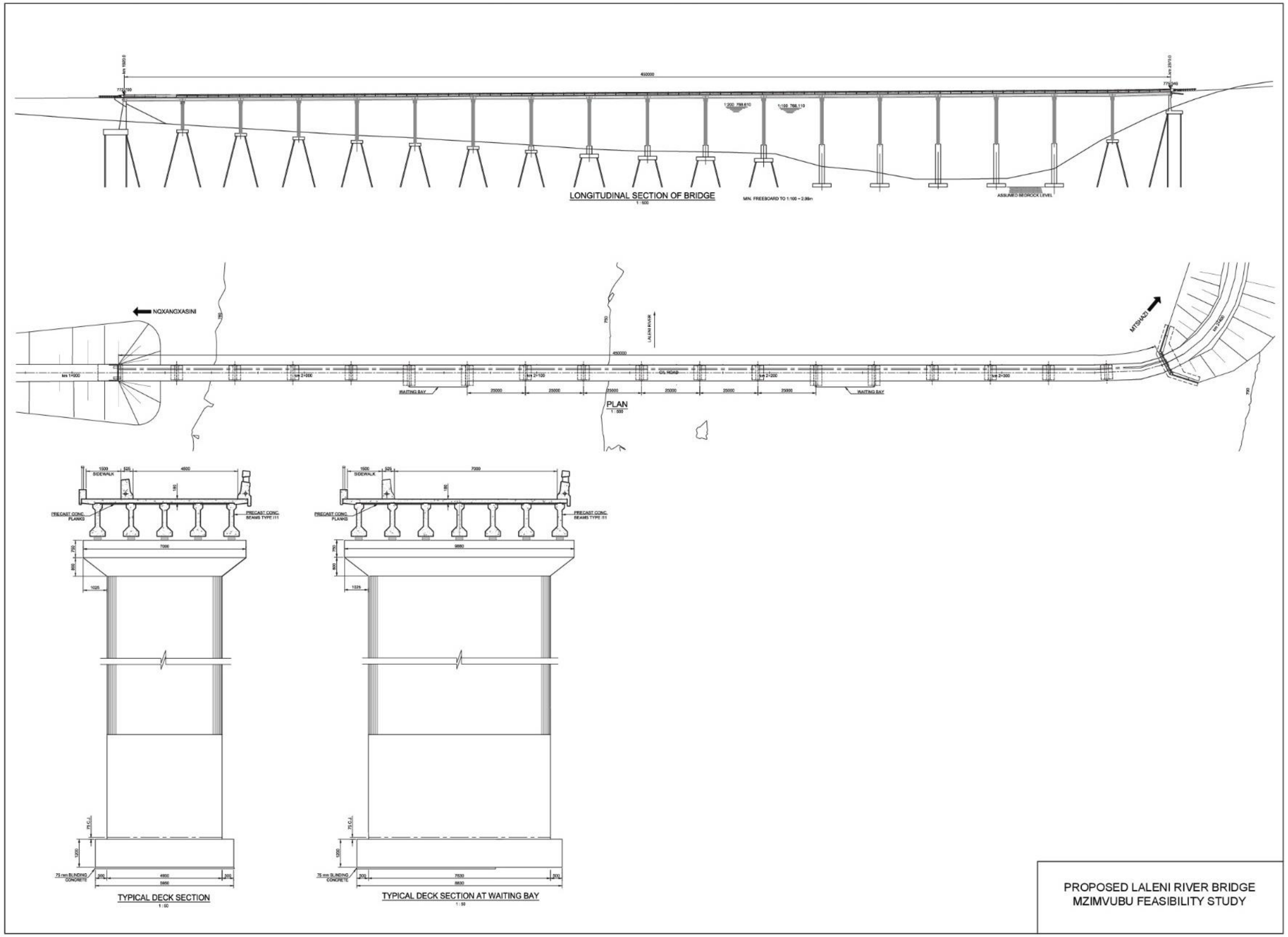


Figure 22: Proposed Lalini Bridge over Inundated River Section

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 1 000 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 Qumbu, Maclear, Tsolo and Mthatha, and 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.

The estimated operational staff levels of the Lalini Dam and HEP are as given in Table 7.

These are considered to be the maximum number required, and these numbers may reduce depending upon who operates the dam and HEP and the calibre of staff assigned to these operations.

Table 7: Estimated Staff Requirements at Lalini Dam and Hydropower Plant

Lalini Dam				
Position	**Haygrade[2]**	**Day Shift**	**Night Shift**	**Total Shifts/Day**
Senior Water Control Officer	G	1	1	2
General Worker	A	4	2	6
Totals		**5**	**3**	**8**
Lalini Hydropower Plants (Both)				
Position	**Haygrade**	**Day Shift**	**Night Shift**	**Total Shifts/Day**
Certified Engineer (also covers dam)	L	1		1
Senior Plant Superintendent	J	1		1
Artisan Electrician	H	1	1	2
Artisan Millwright / Fitter & Turner	H	1		1
Artisan Aid	C	4	2	6
Totals		**8**	**3**	**11**

A proposed site infrastructure layout is given on Figure 23.

Given the permanent road network that will be established to access all of the Lalini infrastructure components, it is proposed that a staff accommodation housing estate is constructed as shown at a suitable location within short commuting distance to both the dam and HEP.

Allowance will also be made to additionally accommodate official visitors such as head office management, and the occasional VIP.

[2] *The Hay system of job evaluation is a point factor method of job evaluation that measures three factors common to all jobs – know-how, problem solving and accountability. The classification system focuses on internal job relationships and maintaining internal equity.*

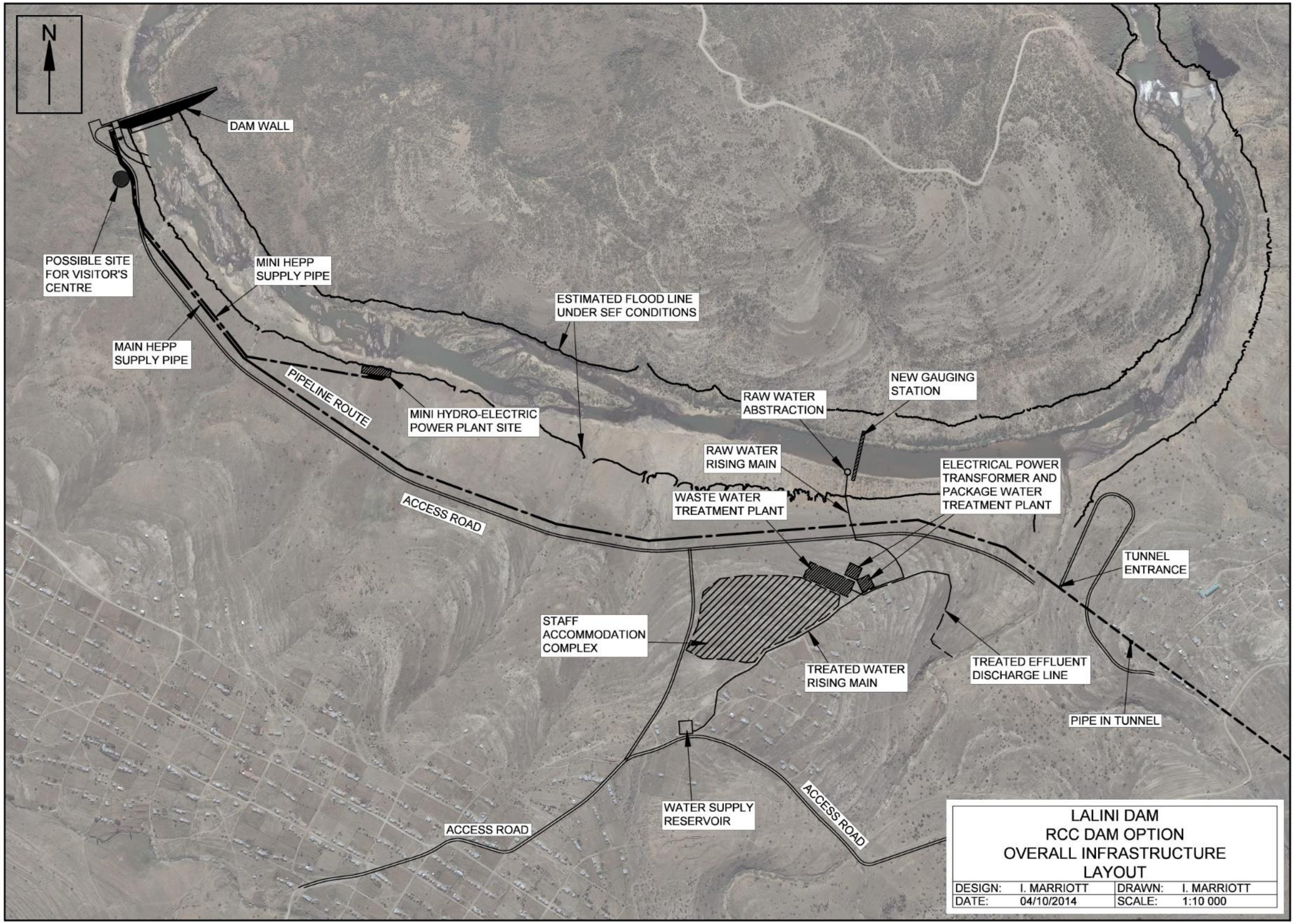


Figure 23: Proposed Lalini Site Infrastructure Layout

Provision has therefore been made for a housing estate containing some 16 stands on which one, two- and three-bedroom staff houses can be built. These will also have fitted kitchens, bathrooms, lounge and dining rooms, and will have mains electricity, water, and waterborne sanitation. If more housing is eventually required, there is sufficient land available for this purpose within the boundary shown.

Allowance has been made in the project budget for construction of 4 x one bedroom, 10 x two bedroom, and 2 x three bedroom houses. These requirements would be reviewed during the design stage.

Electricity will be via ESKOM connection, water supply from a small package plant drawing from the river downstream of the dam (using the proposed new flow gauging station as an abstraction weir), and a wastewater treatment facility will also be built, with its discharge of treated effluent either directly to the river or via a tributary which flows into the river. The housing complex will also have street lighting, tarred roads and surface water drainage.

POWER SUPPLIES AND GRID CONNECTIONS

Table 8 summarizes the expected power load requirements during the construction and operation of the scheme as well as the grid access connection capacities required to deliver the generated hydropower into the local grid system

The connections required for loads 1 and 2 would be used both for the works construction and longer term to operate the works. This would also include the supply of power to the housing, offices, water supply and wastewater treatment plant.

Discussions with ESKOM have resulted in suggestions that the main grid connection to the Lalini scheme would be via a 132 kV line to the existing 132 kV grid system. This is as indicated on Figure 24.

This line should be constructed as advance works under the project to ESKOM's approved standards rather than ESKOM themselves undertaking the construction. The reason for this is that the construction power supply is required to be in place before any construction can start and ESKOM stated that they would need up to three years to implement if they were tasked with this component of the scheme.

This 132 kV line would therefore initially provide a power supply to the Lalini scheme, but would later be switched and synchronized so that the net surplus power generated by the Lalini HEPs could be fed back into national grid to facilitate revenue generation.

Within the Lalini scheme itself, a further 22 kV power line will need to be constructed from the Lalini main HEP transformer/switching compound to provide power to the dam, tunnel and infrastructure works, which later can be used to evacuate the surplus power generated at the Lalini mini-HEP back into the national grid. This 22 kV line should also be expediently constructed under the advance works rather than be assigned to ESKOM to implement.

The proposed alignments of the 132 kV and 22 kV lines are as indicated in Figure 23, and these maximize the usage of existing and proposed road corridors which can serve as joint servitudes, thus minimizing the land requirements. These alignments must be optimized during the detailed design stage. An amendment to the environmental authorisation or a new EIA will be required if these routes need to be revised from those included in the EIA study.

Table 8: Power Requirements for Scheme

						Load Locations	
Ref. No.	**Use description**	**Eskom infrastructure required from:**	**Capacity**	**Required for construction**	**Required for permanent use**	**Latitude**	**Longitude**
New Loads Required on ESKOM grid							
1	Power supply for Lalini tunnel and HEP	Year 2018	5 MW	Yes	Yes*	31°17'53.54"S	28°59'10.76"E
2	Power supply for Lalini dam and associated works	Year 2018	10 MW	Yes	Yes*	31°15'54.61"S	28°55'05.82"E
Hydropower Plants to Feed into ESKOM grid						**HEP Plant Locations**	
3	Lalini mini-hydropower plant	Year 2021	Seasonal output of 1 MW to 5 MW	No	Yes	31°15'58.25"S	28°55'08.37"E
4	Lalini Main Hydro Power Plant	Year 2021	Seasonal output of 12.5 MW to 37.5 MW.	No	Yes	31°17'55.04"S	28°59'10.67"E

** Permanent use would be at a much lower power requirement for operations, housing, water supply, wastewater treatment, HEP black-start, lighting, valves, and control systems, etc.*

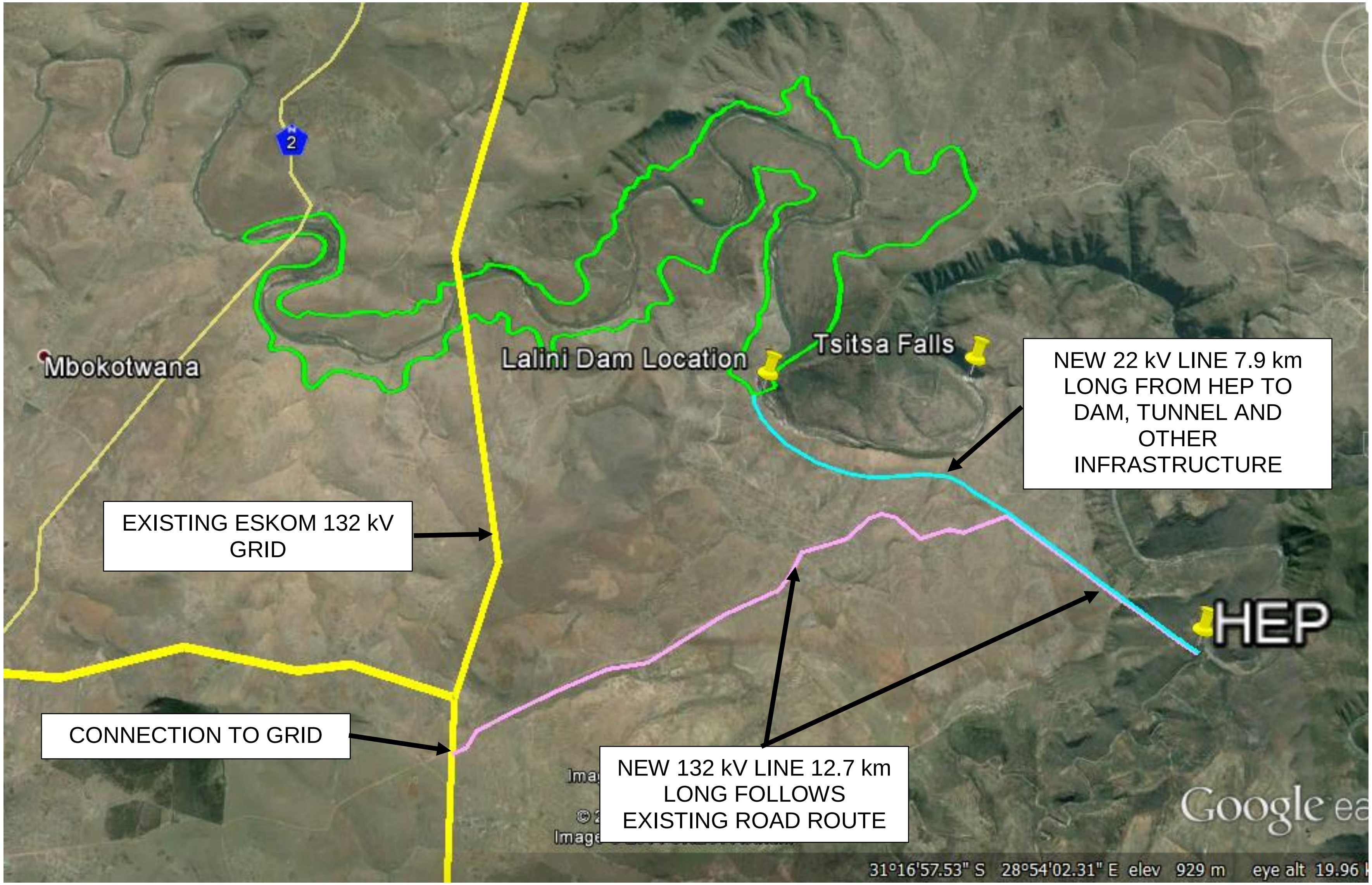


Figure 24: Proposed 132 kV and 22 kV Power Line Alignments

WATER SUPPLY

The villages of Lalini and Lotana both have existing water supplies but it is not certain that these would have sufficient capacity to meet the short and longer-term requirements at the Lalini Dam and staff accommodation complex.

A separate water supply should therefore be developed to supply potable water to the offices and temporary accommodation during the construction period, and for the permanent accommodation village and administration offices in the longer term. 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.

It is recommended that this plant not be sized any larger than this to cater for the dam, tunnel, and other works construction as this would normally be the contractor's responsibility.

WASTEWATER TREATMENT PLANT

A wastewater treatment plant will be required to treat effluents produced by the Lalini Dam operations centre and housing complex. This would 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, before discharging treated effluents back to the river to approved quality standards.

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.

At the main HEP site, the ablution facilities could discharge to a septic tank system as usage will be of low volume.

TELECOMMUNICATIONS

Whilst the cellular network in the region has reasonably good coverage, adequate communication systems will need to be assured 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.

VISITOR'S INFORMATION CENTRE

The Lalini Dam and its body of water, and the hydropower plants, 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 information centre can form the focus of such an initiative by providing visitors with a view of the works and information on the project, including the cultural and tourism activities in the area. A location for this centre is suggested above on Figure 23. It is recommended that such a building be of interesting architecture in keeping with the local culture and terrain.

Consideration could also be given to combining this building for both visitors and as the administration and operations centre. If this building could be completed early enough as a part of the advance infrastructure, then it could be used as the Client and Resident Engineers offices during construction as was the case at Katse Dam.

COMPENSATION AND MITIGATION WORKS

The EIA PSP has identified other mitigations, offsets, and compensation works that could require engineering inputs and construction activities.

These include, inter alia:

- *relocation of homesteads affected by the scheme;*
- *lost livelihood compensation;*
- *a water and sanitation health (WASH) awareness programme;*
- *land acquisition and offsets;*
- *wetland offsets;*
- *flora and fauna relocation and rescue;*
- *fish/eel ladders etc.; and*
- *other mitigations, such as improvement of schools, clinics and police stations.*

Preliminary budgets have been provided in the cost estimates for these other potential works, the final requirements and implementation of which should be further considered in the detailed design stage.

CAPITAL COST

The cost estimate for the Lalini Dam and its associated infrastructure, and the two hydropower plants and associated infrastructure, is given in Table 9. Full details of these cost build-ups, cashflow projections and escalation calculations are given in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15.

Costs have been presented for the two base load options described above as well as the peaking station option. As described in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15, the levelized cost of power produced by the two base load options are identical. It is very much higher for the peaking option, which also has significant increased cost and environmental impact implications. The recommended scheme is therefore the 37.5 MW base load installation.

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. More details are given in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15. Maintenance costs per annum are based upon the percentages of capital cost recommended in DWS's Water Supply Planning and Design Guidelines. Operational staff costs have been sourced from those currently applied to similar works operated by Amatola Water.

The following are estimates of these annual operating and maintenance costs, but these should be treated with caution pending decisions being made on the eventual institutional arrangements:

Operation and Maintenance Costs:	*R20.83 million/a*
Staff costs:	*R 6.80 million/a*
Power costs:	*R 3.00 million/a*

These costs are taken into account in the financing options detailed in the Legal, Institutional and Financing Arrangements Report No. P WMA 12/T30/00/5212/16.

Table 9: Capital Cost Estimates

Main HEP Installed Capacity Option:>	***37.5 MW***	***50 MW***	***150 MW***
Component	***Capital Cost R'million***		
Lalini Dam (0.28 x MAR Capacity)	*601.64*	*601.64*	*601.64*
Associated Works	*127.01*	*127.01*	*127.01*
Mini-Hydropower Plant			
Building Structure incl O/H Crane	*11.55*	*11.55*	*11.55*
Turbines & Generators Electro-Mech	*37.00*	*37.00*	*37.00*
Transformer Station	*2.00*	*2.00*	*2.00*
Power lines (22 kV) to Grid (say 8 km)	*6.00*	*6.00*	*6.00*
Access Roads			
Lalini Main Road Upgrade	*52.31*	*52.31*	*52.31*
Tunnel Entrance Access Road	*11.20*	*11.20*	*11.20*
Dam & Pipeline Access Road	*15.43*	*15.43*	*15.43*
HEP Access Road Option 1	*173.02*	*173.02*	*173.02*
Roads and Bridges Realignments			
Mtshazi Main Road Upgrade & Realignment	*87.36*	*87.36*	*87.36*
Lalini Bridge Realignment	*103.70*	*103.70*	*103.70*
Hydropower Water Delivery Conduit	*2 500 mm dia.*	*3 000 mm dia.*	*4 500 mm dia.*
Longer tunnel option	*687.07*	*860.88*	*1 320.68*
Main Hydropower Plant			
Building Structure incl O/H Crane	*28.80*	*38.40*	*42.24*
Turbines & Generators Electro-Mech	*119.59*	*163.27*	*907.50*
Switching and Transformer Station	*3.00*	*5.00*	*incl*
Earthworks	*7.50*	*10.00*	*10.00*
Power Lines to Grid 12.7 km (132 kV)	*17.50*	*17.50*	*17.50*
Sub-Total Cost Estimates	***2 091.69***	***2 323.28***	***3 526.14***
Contingencies (10%)	*209.17*	*232.33*	*352.61*
Engineering and EIA Mitigations (12%)	*276.10*	*306.67*	*465.45*
Escalation (averages 18%)	*463.85*	*515.21*	*781.96*
VAT (14%)	*425.71*	*472.85*	*717.66*
Grand Total (R'million)	***3 466.53***	***3 850.34***	***5 843.83***

PROJECT IMPLEMENTATION PROGRAMMING

Given that the implementation of the Lalini hydropower scheme is a key component of the conjunctive scheme which generates significant revenue such that all of the power costs and more on the Ntabelanga water supply and irrigation component of the conjunctive scheme can be cross-subsidized, thus bringing the unit cost of water produced down to a viable and sustainable level.

In order that these benefits are realized timeously, it is recommended that this component be implemented simultaneously with the Ntabelanga components so that there is no lag in the revenue stream that produces such cross-subsidization. A draft implementation programme is included in Appendix C. This is under review by the DWS and will be regularly updated.

PRIORITY INFRASTRUCTURE

The following are considered to be associated works components that should be constructed as a priority, and should therefore be part of an advance infrastructure contract which is completed before the main works construction commences:

- *Main access roads, especially those to the dam, and to the tunnel exit portal and main HEP plant;*
- *Power supplies; and*
- *Telecommunications.*

Additional optional components are:

- *Staff accommodation if to be used by DWS and engineer's team during construction, but do not allow contractor to use;*
- *Temporary water supply and wastewater treatment works, if staff accommodation is built; and*
- *Visitor's information centre and admin/operations centre, which could be DWS and engineer's site offices.*

Most of the above works will require an environmental authorization, and are therefore included in the EIA authorization process.

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 undertaking this programme, which has commenced well ahead of the commissioning of the Ntabelanga and Lalini dams.

An impression of what the Lalini Dam could look like is given in Figure 25.

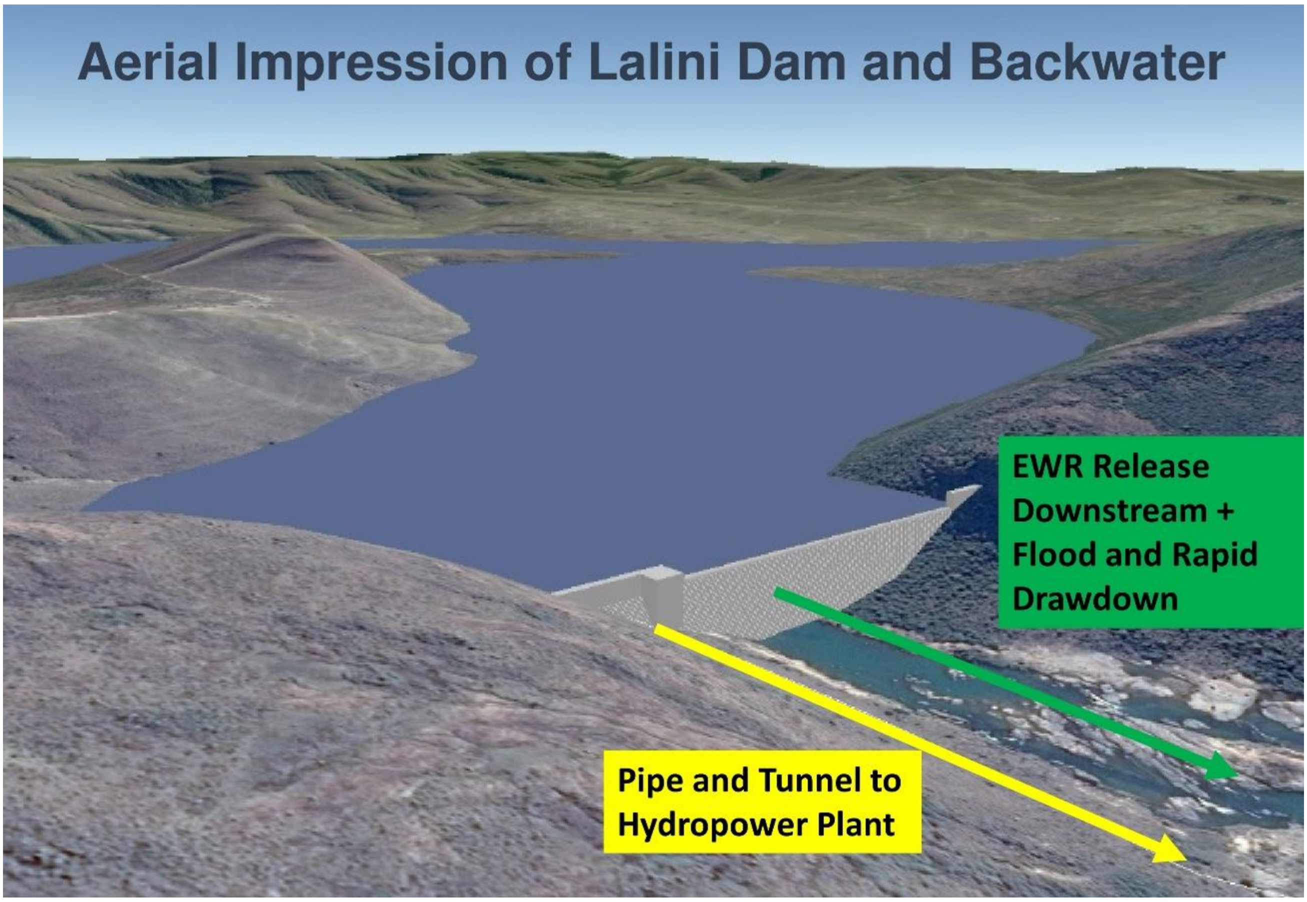


Figure 25: Impression of Lalini Dam

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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.

1.2 Study Programme

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.

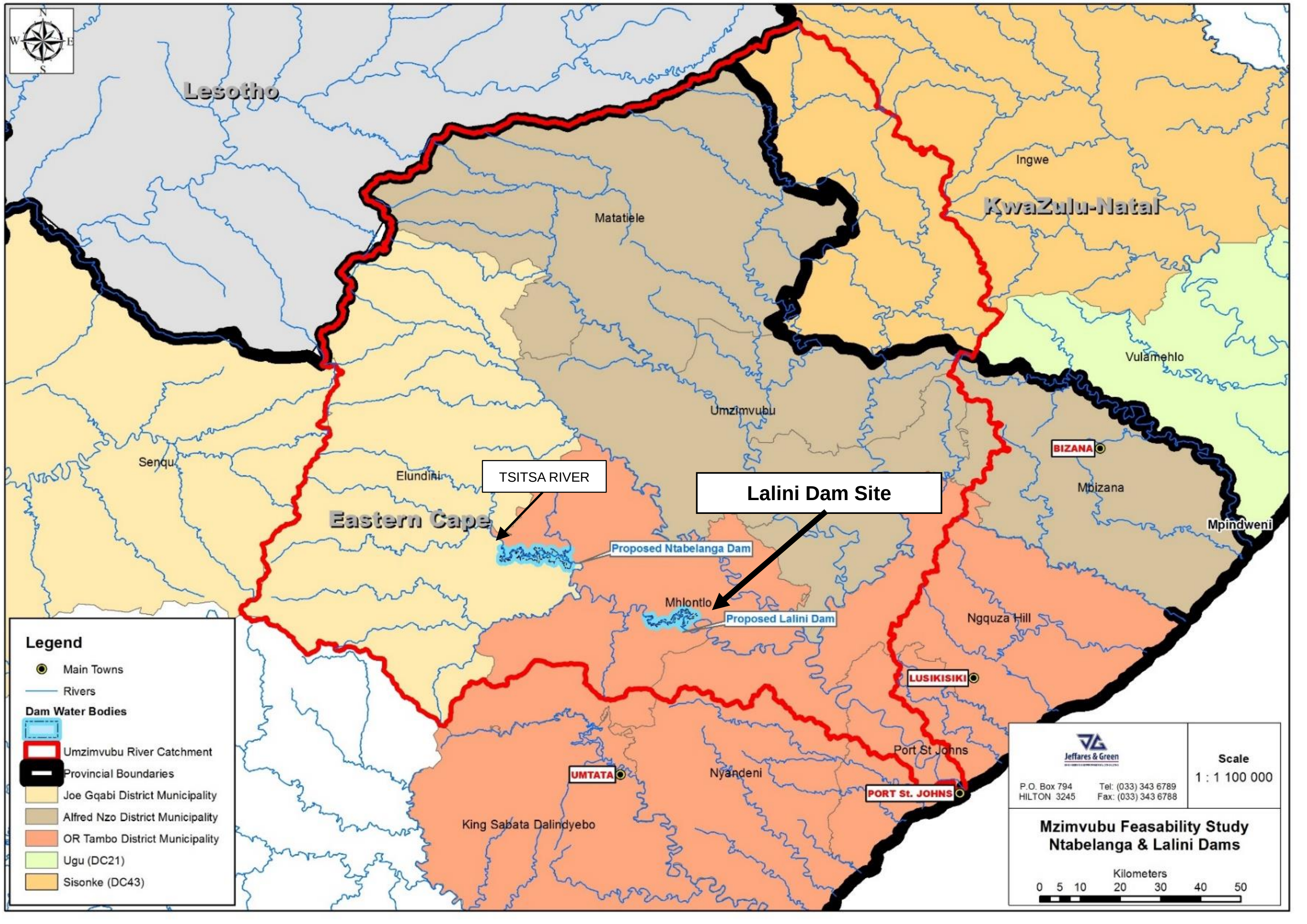


Figure 1-1: Location of Ntabelanga and Lalini Dams Relative to Overall Mzimvubu Catchment Area

The purpose of the study was 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.

1.2.1 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.

1.2.2 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:

(1) Desktop Study; and
(2) 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).

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 dam 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.

1.2.3 *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;
- Environmental Impact Assessment (to be undertaken in a separate study that will run in parallel to this one);
- Public participation;
- Regional economics; and
- Legal, institutional and financial arrangements.

1.2.4 *Additional Detailed Investigations for Lalini Dam and Hydropower Scheme*

Following a variation order which extended the study programme to the end of October 2014, further detailed investigations were undertaken for a second dam on the Tsitsa at Lalini (just above the Tsitsa Falls) and its hydropower scheme, which would be operated conjunctively with the Ntabelanga Dam to generate significant hydropower for supply into the national grid.

1.3 Purpose of this Report

This report describes the process undertaken during the extended programme period to develop an optimum selection of dam location, dam type, spillway type, and the feasibility level design of the selected type of dam, outlet works, water transfer conduit, main and mini hydropower plants, power transmission lines, and associated works, for the Lalini Dam and hydropower scheme.

It was confirmed and agreed in Phase 2 that the sizing and modus operandi of the Lalini Dam and its associated works would meet the following requirements:

- To operate conjunctively with the 1.18 MAR_{PD} (490 million m^3) Ntabelanga Dam, to generate hydropower locally at the dam wall, as well as at a location in the gorge downstream of the Tsitsa Falls, for supply into the regional grid; and
- To provide sufficient flow of water downstream of the Lalini Dam to meet environmental water requirements for an ecological Class B/C.

2. DAM LOCATIONS

A locality map of the Mzimvubu River catchment area and its position in relation to the DMs in the area is provided in Figure 1-1. The locality of the two dams that are the subject of this analysis are also shown on Figure 2-1. These are:

- Ntabelanga Dam located on the Tsitsa River within the quaternary catchment T35E; and
- Lalini Dam, which is also located on the Tsitsa River, within the quaternary catchment T33L.

The proposed Ntabelanga Dam is located approximately 55 km north of Mthatha on the Tsitsa River, and the proposed Lalini Dam is located approximately 38 km north-east of Mthatha, as illustrated in Figures 1-1 and 2-1.

The catchment areas contributing to the Ntabelanga and Lalini Dams are approximately 1 967 km^2 and 4 422 km^2 respectively comprised of the contributing quaternary catchment areas as given in Table 2-1, and as delineated in Figure 2-2.

The catchment area contributing to the Ntabelanga and Lalini Dams in the tertiary catchment T35 is somewhat developed with approximately 10% of the catchment area under commercial forestry.

Table 2-1: Contributing Catchment Areas for the Study Area

Tsitsa River Catchment	
Quaternary Catchment	**Catchment Area (km^2)**
T35A	476.5
T35B	396.8
T35C	307.0
T35D	348.9
T35E	493.5
T35F	359.6
T35G	576.2
T35H	521.0
T35J	189.0
T35K	627.1
T35L	339.5
TOTAL	**4 635.1**

2.1 Ntabelanga Dam Feasibility Design

The conclusions and recommendations of the Ntabelanga Dam Feasibility Design are described in Report No. P WMA 12/T30/00/5212/12.

All investigations for the conjunctive operation of the Ntabelanga Dam and the Lalini Dam and hydropower scheme have been based upon the 1.18 MAR_{PD} (490 million m^3) capacity Ntabelanga Dam.

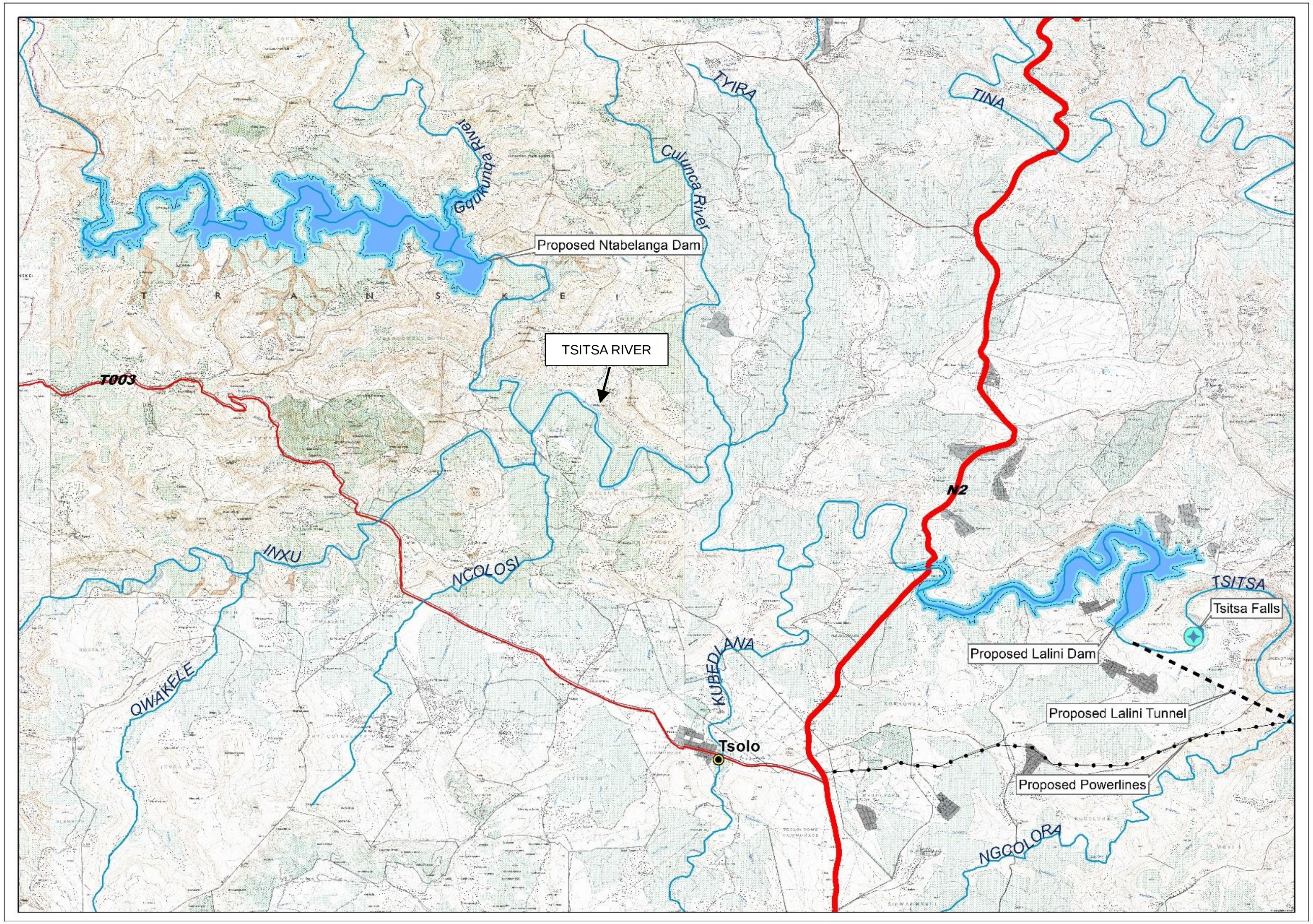


Figure 2-1: Locality of Ntabelanga Dam Relative to the Lalini Dam

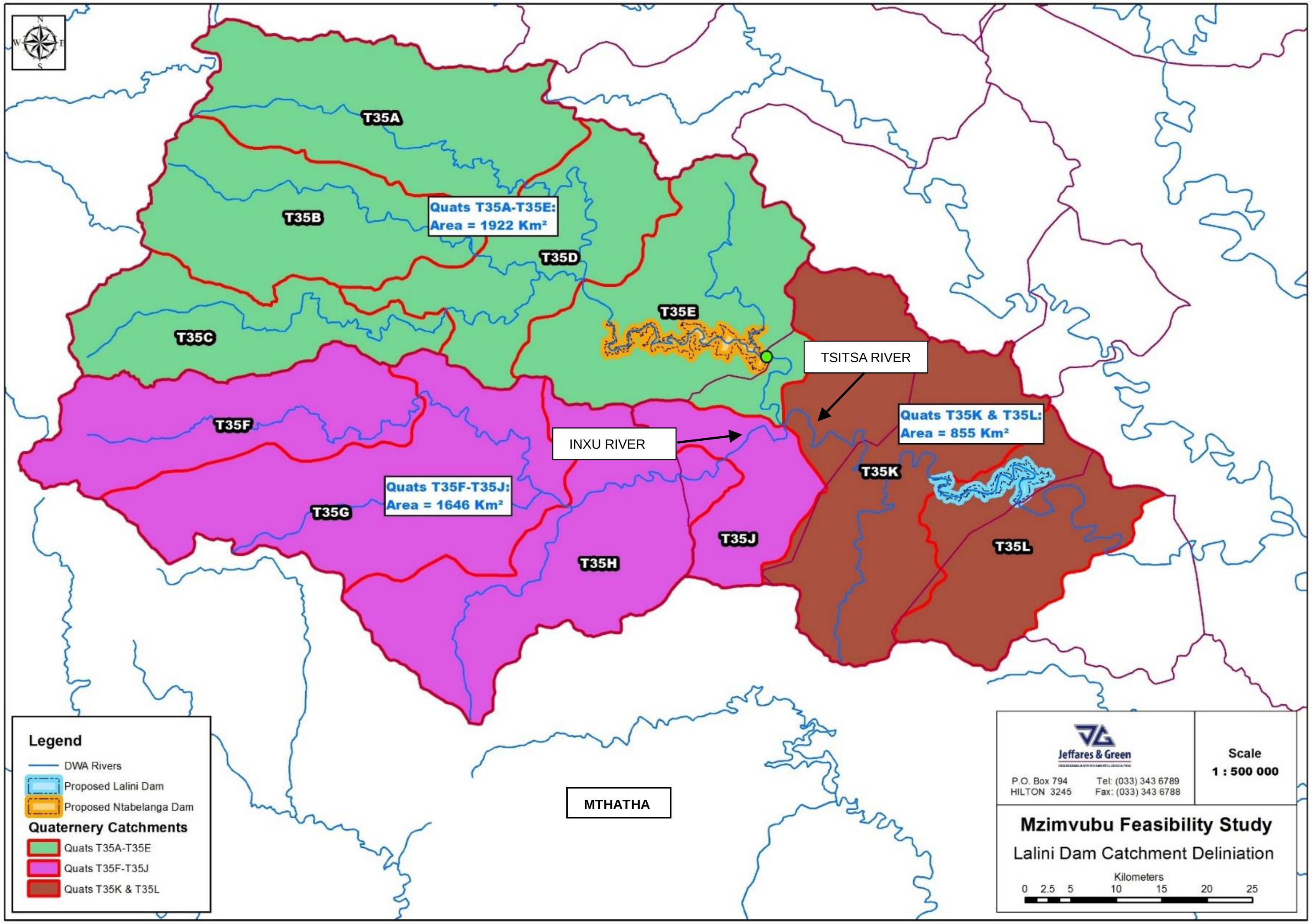


Figure 2-2: Lalini Dam Catchment Delineation

3. LALINI DAM FINAL SITING AND SIZING

3.1 Dam Site Location

A review of the location of the Lalini Dam wall identified in previous studies, was undertaken using both topographical mapping as well as field reconnaissance.

The proposed Lalini Dam is located approximately 40.5 km north of Mthatha on the Tsitsa River, at co-ordinates 31°15'44.76"S and 28°55'15.87"E.

Figures 3-1 to 3-3 are photographs that were taken during the site reconnaissance mission which was undertaken to inspect the surface morphology and implied geology, and to thus determine a preferred dam alignment.

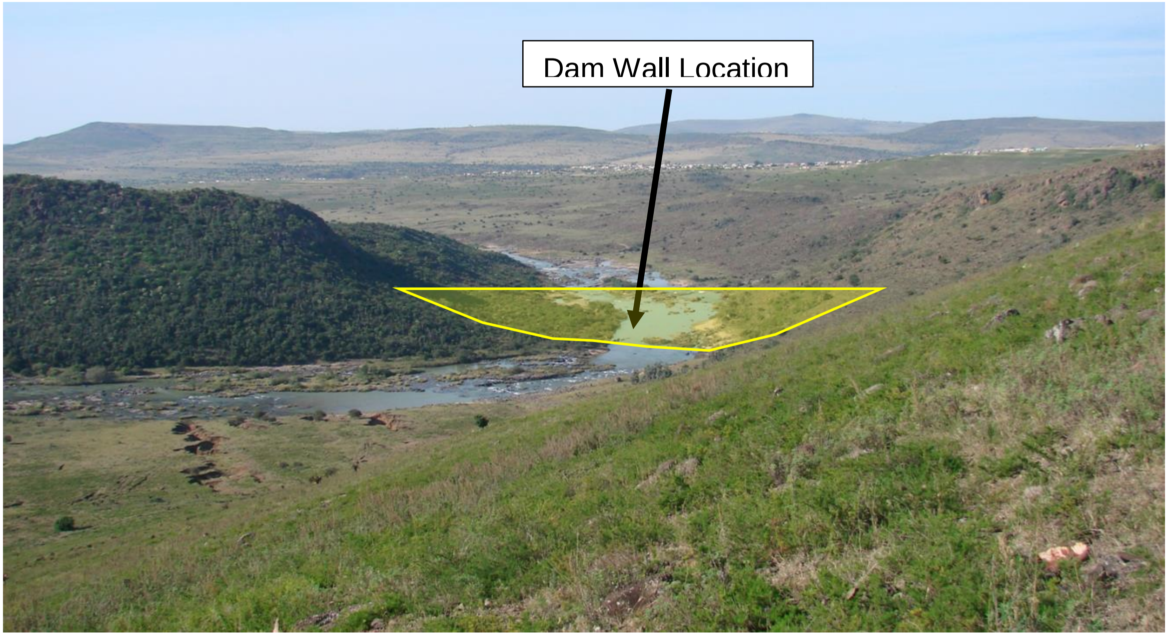


Figure 3-1: Proposed Lalini Dam Site Looking Downstream

It was concluded that the Lalini Site provided a very favourable river valley shape (with effect to 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 is below Tsitsa Falls. However, this would require a very high dam wall in order to provide sufficient head for hydropower generation. Furthermore, the Tsitsa Falls would be inundated once the dam is constructed. Therefore, the more detailed Lalini Dam wall siting investigations for the feasibility study have been focussed on the narrowest part of the Tsitsa River valley some 3.5 km upstream of the Tsitsa Falls.

The morphology and geology evident from the observations during the reconnaissance mission was that the dam would be founded on competent dolerite which extends well below the likely dam foundation level and into and up the left hand abutment. On the right hand abutment the dolerite is overlain by competent sandstone. The *prima face* evidence from the site reconnaissance was of a highly suitable dam site.

Figure 3-2: View of Foundation and Left Abutment from Centreline

Based upon these findings, geotechnical investigations (core drilling) and materials trial pitting and sampling were carried out on the dam wall alignment and potential spillway locations, as well as potential rock quarries and borrow pits, and are described in detail in Geotechnical Investigations Report No. P WMA 12/T30/00/5212/10.

3.2 Summary of Dam Site and Materials Investigations

Whilst the above geotechnical investigations report provides full details, results, conclusions and recommendations regarding the dam site investigations, and the investigations to identify suitable dam construction materials, the following is a summary thereof.

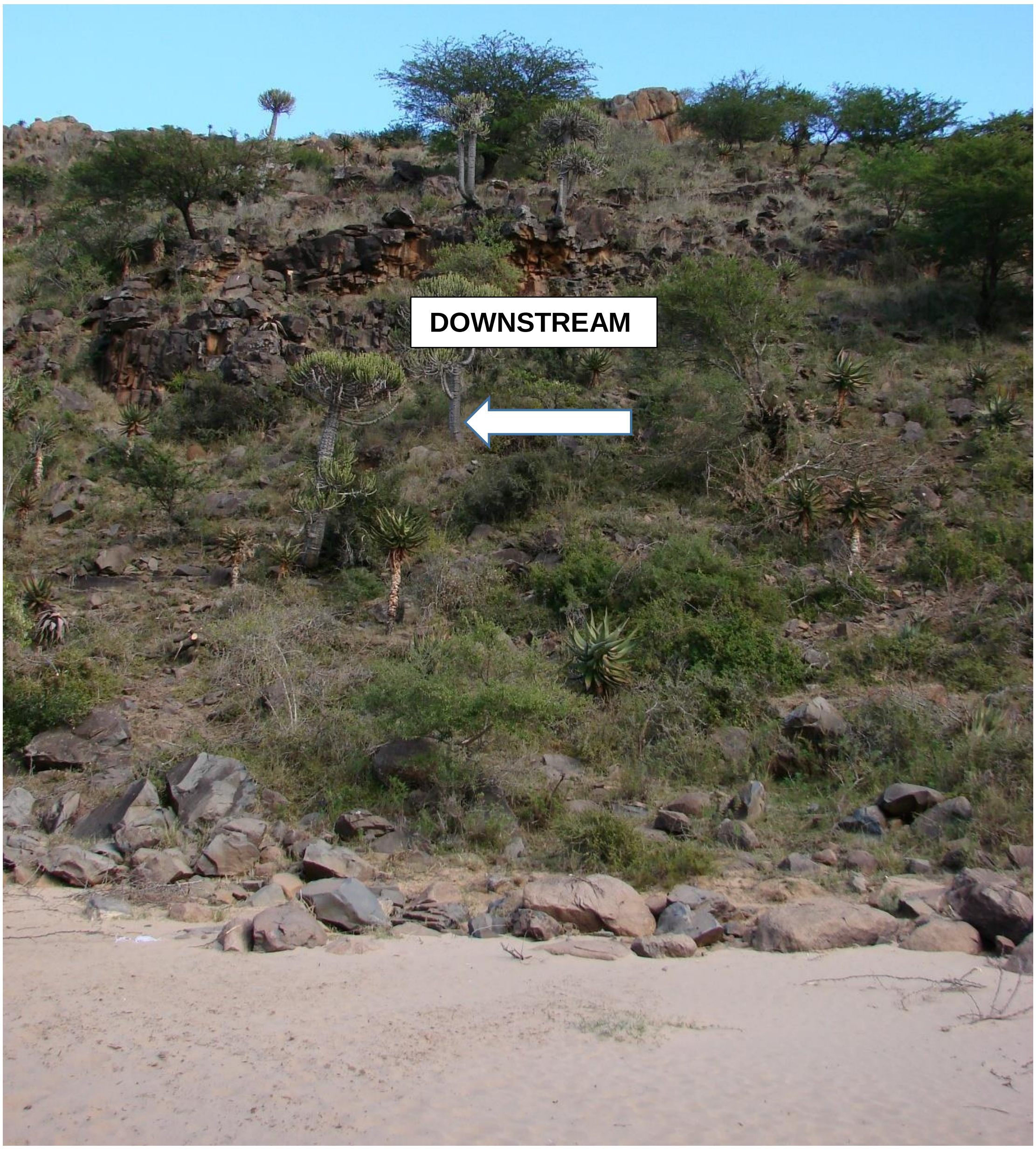


Figure 3-3: View of Right Abutment from Centreline

The feasibility level geotechnical investigation of the proposed Lalini Dam and conduit pipline and tunnel sections entailed the following:

i. The drilling of four rotary core boreholes along the proposed alignment of the dam axis, two on the left flank and two on the right flank. Dolerite outcrop occurs across the river section.

ii. The drilling of seven boreholes for the proposed hydro-power scheme, of which four were positioned along or adjacent to the preferred horizontal alignment, one just below the dam to cater for the pipeline section or an alternative tunnel alignment and one to the south west of the preferred tunnel alignment to cater for an alternative longer and deeper tunnel option. Five of the boreholes were inclined 5° off vertical to facilitate the undertaking of core orientation measurements.

iii. The drilling of six boreholes in an identified potential rock quarry site.

iv. A co-ordinated trial pitting investigation of identified potential borrow pits for earth embankment construction.

v. The excavation of trial pits along the proposed pipeline alignment.

vi. Water pressure tests were conducted at representative intervals in all the dam boreholes and in one tunnel borehole.

vii. Rock strength tests were conducted on representative borehole core samples, either by means of laboratory unconfined compressive strength (UCS) tests or point load strength index (PLSI) tests conducted on site.

viii. Representative samples were retrieved of the unconsolidated materials proposed for earthfill dam construction to facilitate testing and analysis.

ix. Water samples were retrieved from selected boreholes and from the Tsitsa River, the former for chemical aggressiveness testing and the latter to assess suitability for use in construction.

x. Associated rock exposure mapping and photography.

Figure 3-4 shows a summary of the core logs in the four boreholes drilled along the dam wall centreline.

Figure 3-5 shows the locality of identified quarries and borrow areas identified that will provide sufficient quantities of materials for each dam type construction.

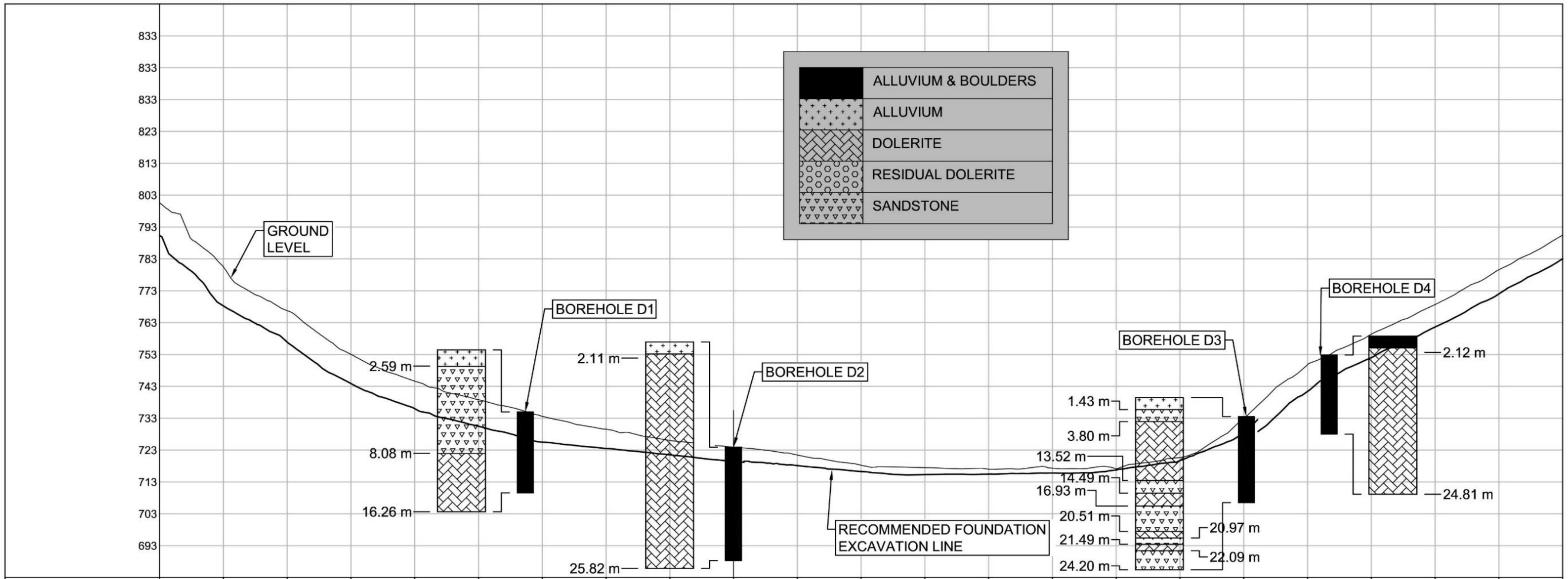
The extent of the geotechnical investigations undertaken along the proposed dam axis has concluded that the site is suitable for the construction of an earthfill embankment dam, a rockfill dam, or a RCC dam.

Based upon the drilling undertaken the dam foundation invert will vary from between 6 m and 8 m below ground level on the upper flanks to between 3 m and 4 m below ground level on the lower flanks. Dolerite outcrops, visible across the river section, implying that only moderate excavation would be required in this area.

The results of water pressure tests indicate that minor under-seepage is likely and that a cut-off grout curtain will be required. The need for consolidation grouting was not conclusively proven. Further detailed geotechnical investigations will be required to inform the detailed design process.

The reconnaissance for dam construction materials concentrated on areas falling within the future impoundment basin in order to avoid the negative environmental impacts and rehabilitation requirements associated with exploitation outside of the impoundment area.

The area investigated as a potential rock quarry lies on the left hand or eastern side of the Tsitsa River, approximately 3.5 km upstream of the dam. The investigation did prove good quality dolerite, but occurring beneath an excessively thick overburden mantle of unconsolidated, weathered and fractured materials. As a result of this, under normal circumstances, the site would be regarded as being marginal for use as a rock quarry, but the use of the overburden materials in road construction, if found suitable, could mitigate the use of the area as a rock quarry.



CHAINAGE	0	20	40	60	80	100	120	140	160	180	200	220	240	260	280	300	320	340	360	380	400	420	440
GROUND LEVEL	800.59	779.89	766.24	752.72	744.61	738.66	733.37	729.33	725.93	723.89	721.31	717.92	717.67	716.93	717.39	717.32	720.78	732.94	749.30	759.25	768.74	779.82	790.92

Figure 3-4: Lalini Dam Centreline Borehole Log Summary

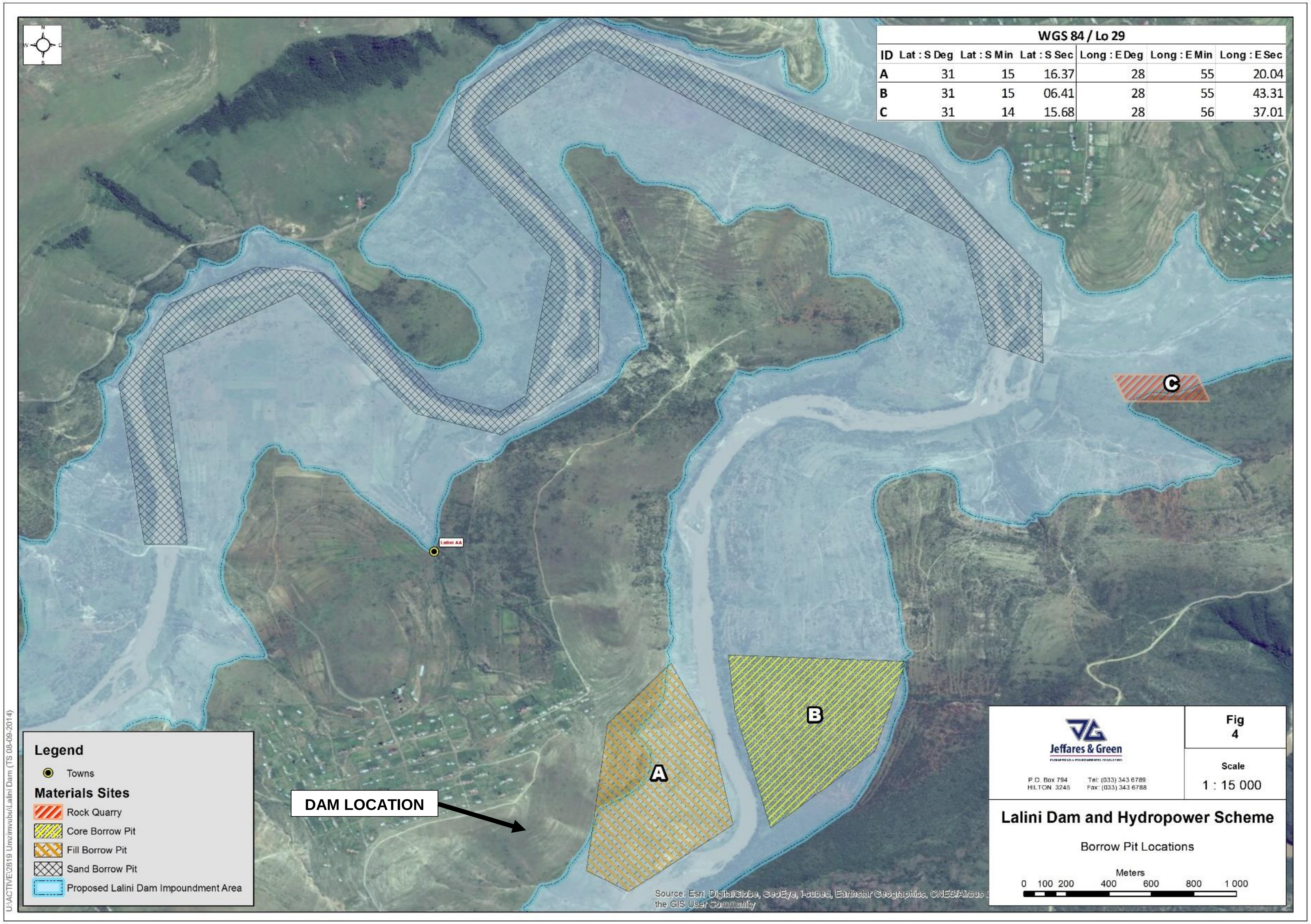


Figure 3-5: Locality of Identified Rock Quarry and Other Material Sources

The naturally occurring sand in the channel of the Tsitsa River was found to be too finely graded for use as either concrete fine aggregate or filter medium. Its use would necessitate blending with an inert crushed rock product. Alternatively sand would have to be acquired from an approved off-site source.

For embankment dam types and for use in cofferdams, suitable core material availability was proven in adequate quantities, a short distance upstream of the dam within the impoundment basin. The area investigated as a shell borrow pit lies immediately upstream of the dam, with geology comprising mudrock and intercalated sandstone. The material tested is coarse grained, but with plastic fines, due to the preponderance of mudrock.

Based upon investigation undertaken and observations made on site, adequate embankment shell (fill) material is available in terms of quality and quantity.

Given these findings, it was determined that availability of suitable materials within reasonable distance of the dam site, and located within the impoundment basin, was sufficient for the further consideration of the following dam types:

- Roller compacted concrete (RCC) dam;
- Concrete faced rockfill dam (CFRD);
- Earth core rockfill dam (ECRD); and
- Earthfill embankment dam with earth core (EF).

3.3 Dam Capacity and Wall Height

The Lalini Dam must have sufficient capacity to store and balance the inflow, in order to sustain a reliable hydropower output. It must also have outlet works of sufficient capacity to release water to the river downstream in order to meet EWR requirements for a Class B/C ecological classification, which will constitute some one-third of the MAR_{NAT} of the river at this location.

A final requirement will be that the dam has sufficient spillway capacity to deal with the Safety Evaluation Flood (SEF) of the river at this location.

The water level verses capacity and surface area characteristics for the dam are as shown in Figure 3-6.

The hydropower analysis described in Report No. P WMA 12/T30/00/5212/18 investigated two power generation scenarios with 37.5 MW and 50 MW installed plant capacity respectively.

The analyses were run for a range of Lalini Dam capacities from 0.10 MAR_{PD} (83 million m^3) to 0.75 MAR_{PD} (619 million m^3) operated conjunctively with a 1.18 MAR_{PD} capacity Ntabelanga Dam (490 million m^3).

Figures 3-7 and 3-8 show the results of these analyses, for the main Lalini hydroelectric plant (HEP) only, as well as the main plant plus the mini-HEPs at each of the two dams included.

Following the undertaking of a detailed topographical survey covering extended areas around the Lalini Dam site and impoundment areas, it was noted that the dam wall height can only be set for a maximum full supply level (FSL) of 780 m.a.s.l. (0.6 MAR_{PD}) before overtopping the terrain on the left flank.

As the construction of saddle dams is not considered necessary or acceptable, and as this size of dam would drown a large area of settlement and existing infrastructure, the upper limit for the maximum Lalini Dam capacity was set at this value for further analysis purposes.

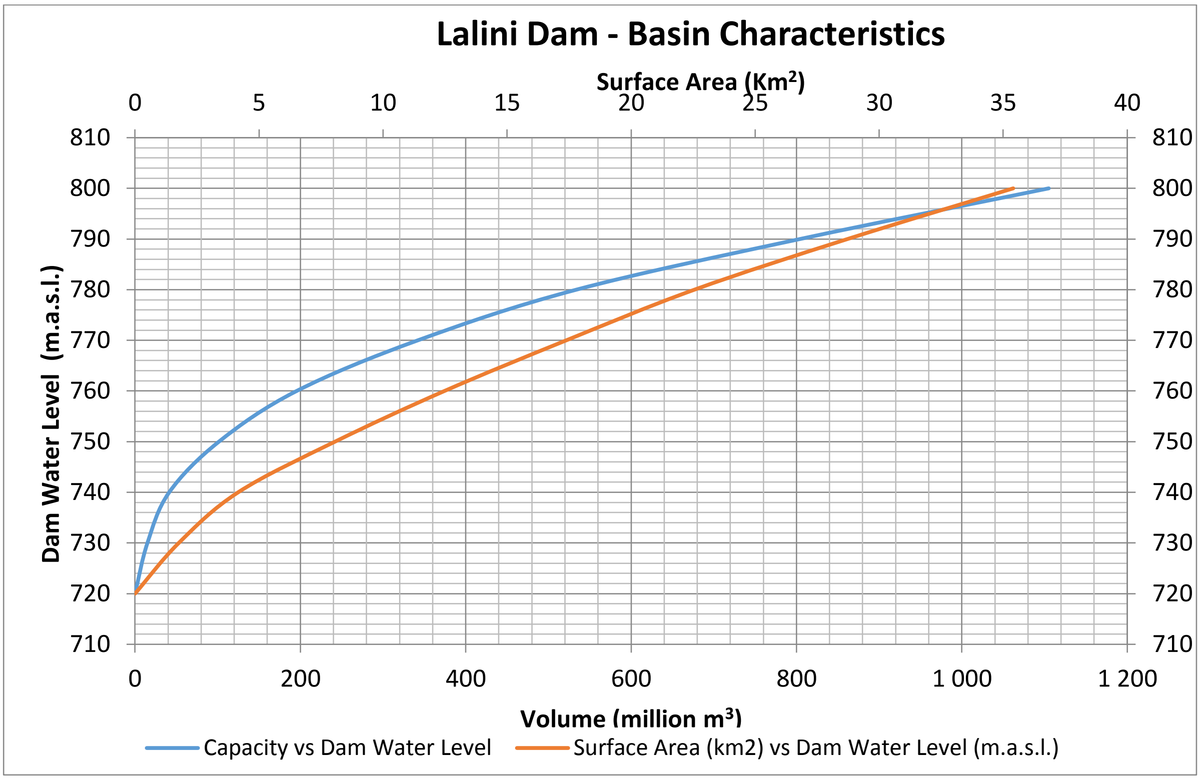


Figure 3-6: Lalini Dam Water Level verses Capacity and Surface Area

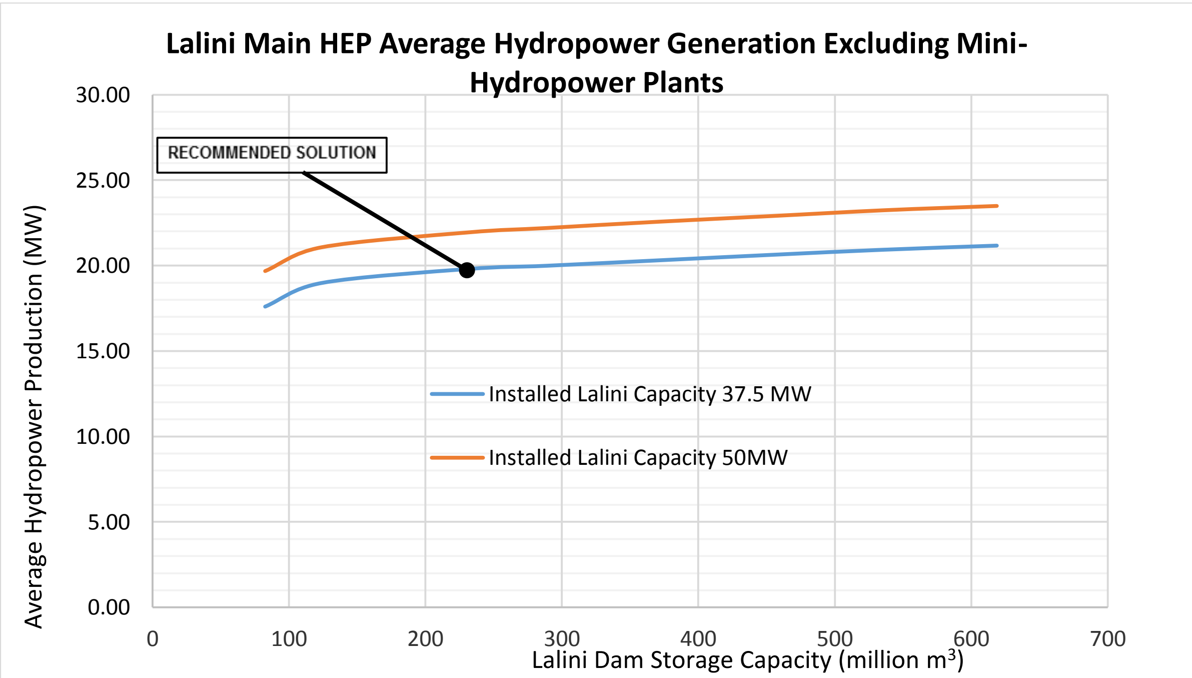


Figure 3-7: Hydropower Results: Lalini Main HEP

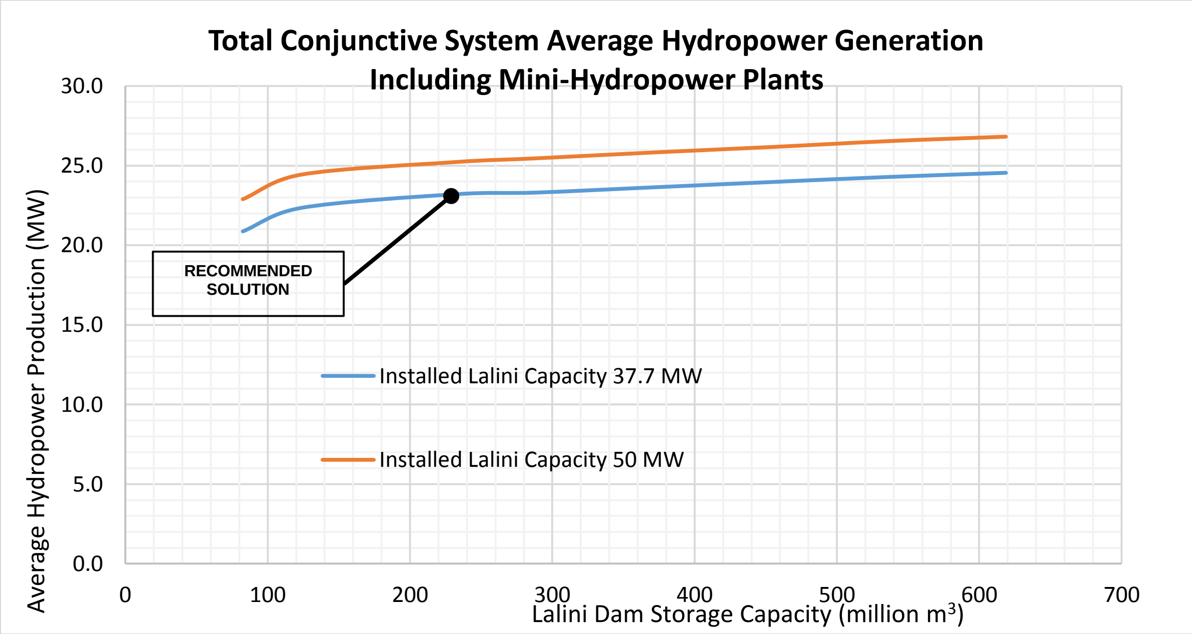


Figure 3-8: Hydropower Results: Conjunctive Scheme incl Mini-HEPs

At the lower end of the capacity options, it can be seen that there is a significant gain in power output from a capacity of 0.10 MAR_{PD} to 0.15 MAR_{PD}. As regards the sediment trapping aspects of Lalini Dam it was shown that a minimum of a 0.18 MAR_{PD} dam should be built to ensure that the dam was large enough to accommodate some 50 years of predicted sedimentation based on the updated Rooseboom methodology.

Given the large investment that a dam requires, some eminent dam proponents are suggesting that dams should be designed for much longer lifespans and that estimates of sedimentation should be made using a longer term criteria. For this, and other reasons related to increasing the output capacity of the HEP, it was decided to investigate dams of a larger capacity than the minimum recommended in Phase 2 of the feasibility study.

This has to be a trade-off between increased capital cost, increased hydropower output, and increased social and environmental impact including the drowning of land, settlement structures, and existing civil infrastructure.

In this case, one of the key infrastructure components affected by the inundation of this valley is the existing national road N2 bridge across the Tsitsa River upstream of the proposed Lalini Dam site. In addition, the existing tarred district road from the N2 to Mtshazi and Shawbury would be partly inundated and sections would require realignment, depending on the final water level in the dam.

The road and low-level river bridge crossing from this district road to the village of Lalini would be drowned under all possible Lalini Dam capacities and the cost (greater than R150 million) of a high-level replacement bridge increases with Lalini Dam water level. See Figure 3-10.

It was therefore decided to focus on a Lalini Dam capacity and water level that could be accommodated by the existing N2 road bridge with provision for SANRAL-acceptable freeboard under 1 in 100 year return period flood conditions.

This bridge is shown in Figures 3-9 and 3-11.

Figure 3-9: N2 Road Bridge Viewed Looking Upstream

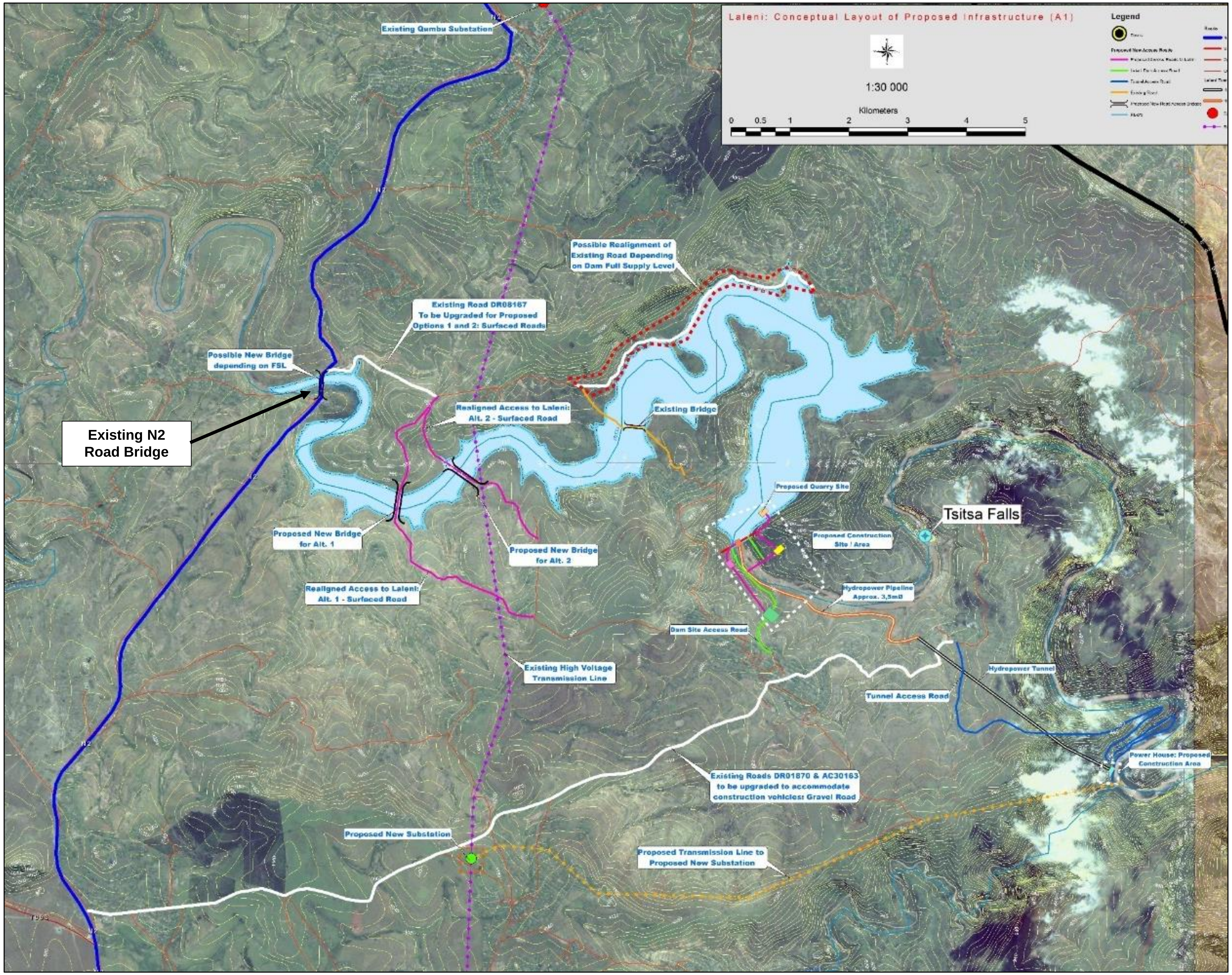


Figure 3-10: Infrastructure Affected by Rising Water Levels in the Lalini Dam

Figure 3-11: N2 Road Bridge across Tsitsa River

Using the required SANRAL design formula, the maximum dam full supply level (FSL) resulting from this bridge freeboard calculation was 765.5 m.a.s.l. which produced a Lalini Dam capacity of some 232 million m^3 or 0.28 MAR_{PD}. It was estimated that raising the bridge to accommodate higher water levels in the Lalini Dam would cost approximately Rand 150 million, as well as significantly increasing the cost of the above described district road realignments and new Lalini access bridge.

As described in Report Nos. P WMA 12/T30/00/5212/15 and P WMA 12/T30/00/5212/18, increasing the Lalini Dam capacity beyond this size also results in significant increased costs for the hydropower water transfer conduit and power generation plants, for which the additional return in terms of energy sales is not proportionately higher.

The social and environmental impacts in terms of lost land and resettlement impacts also start to significantly increase above this proposed “optimum” dam capacity, and the EIA team also concurred that the dam FSL be set no higher than 765.5 m.a.s.l.

The process of dam type analysis was undertaken in parallel to the above investigation of “optimum” dam capacity sizing, and, for the purposes of comparison of different dam types, two capacities were adopted for detailed costing, namely 0.3 MAR_{PD} and 0.6 MAR_{PD}, in order to evaluate the likely range of viable dam capacity options.

4. DAM TYPE ANALYSES

4.1 Dam Options Investigated

Given the morphology and geology of the Lalini area are similar in nature to that found in the Ntabelanga area, the same dam types as were investigated for Ntabelanga were also investigated for the Lalini dam. These were:

- Roller compacted concrete (RCC) dam;
- Concrete faced rockfill dam (CFRD);
- Earth core rockfill dam (ECRD); and
- Earthfill embankment dam with earth core (EF).

Figure 4-1 shows the cross-section profile of the valley at the dam centreline, together with the probable FSL and NOC of the preferred Lalini Dam solution discussed above.

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

- The availability of sufficient quantities and quality of construction materials in the vicinity of the dam wall;
- Constructability issues, especially relating to river diversion during construction;
- The spillway type, location and capacity requirements;
- Operational requirements and outlet works arrangements;
- Environmental and aesthetic impacts; and
- The cost of the works.

In order to assess materials requirements, quantities were calculated for all of the above dam types and spillway options, based upon design criteria (foundation excavation depths, embankment slopes, etc.) and costing models developed during the Ntabelanga Dam analyses, which utilized materials with very similar properties as were identified in the Lalini Dam site area.

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

The cross-sectional profile in Figure 4-1 (note the exaggerated vertical scale) produces a crest length of some 365 m for the maximum FSL, and if an in-channel uncontrolled ogee spillway solution were to be employed, this spillway would occupy virtually all of the valley width, which means that the dam wall would be virtually all concrete in construction. Therefore, for rockfill or earthfill dam embankment options, other spillway solutions would be required, and both cut-through and side channel excavated spillways were therefore also investigated.

One advantage that these excavated spillways offer would be that the large volume of material excavated from them could be used in the construction of the main dam embankment. However, this has to be offset by the actual cost of such large excavation volumes in excess of actual construction requirements which would have to be disposed of as spoil, with the associated environmental consequences.

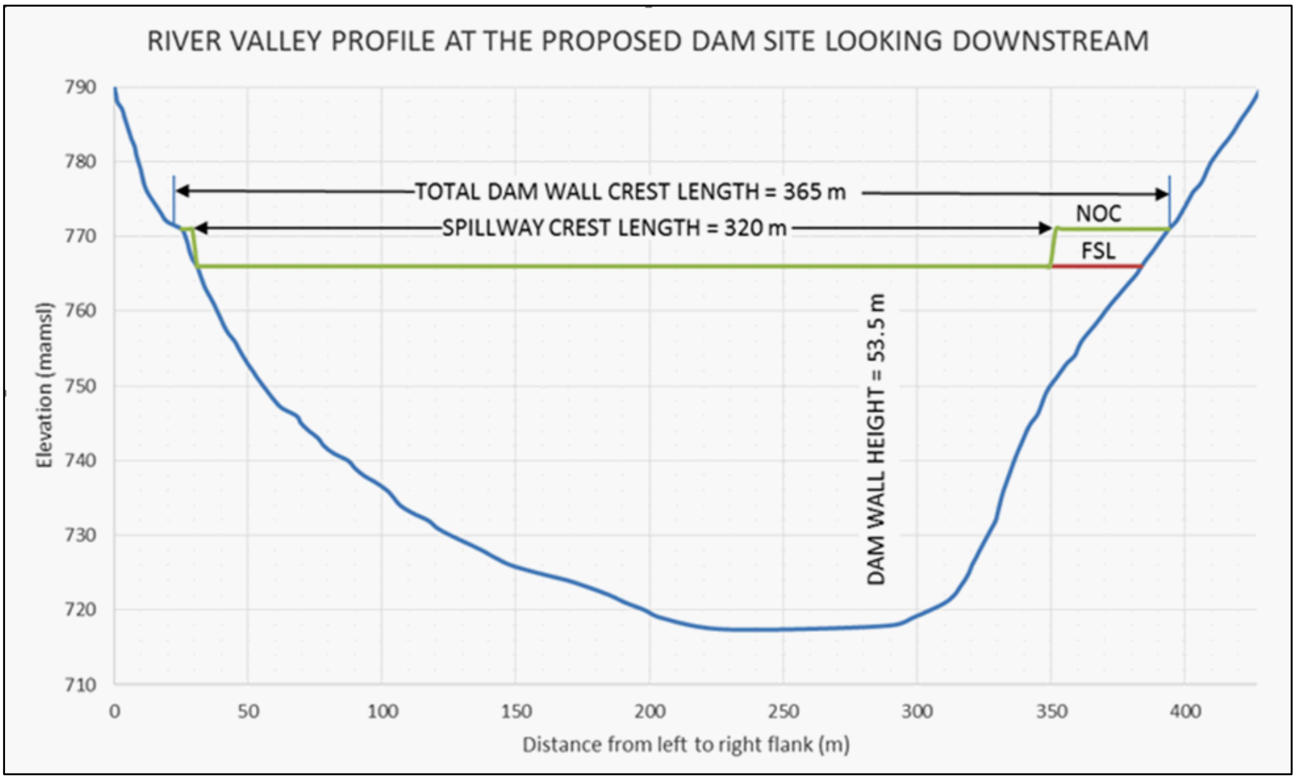


Figure 4-1: Cross-section of Valley (looking downstream) at Dam Wall Centreline

4.2 Spillway Options

4.2.1 Spillway Capacity Requirements

The Design Flood has been determined as described in the design note included herein as Appendix A. This report was submitted to the DWS Hydrology Directorate for review and comment and was finalized after taking into consideration the comments received from DWS Hydrology Section thereon.

From that analysis it was determined that the Recommended Design Flood (RDF) value (which was equivalent to some 1 in 200 year return period flood) would be of the order of 3 500 m^3/s, and the Safety Evaluation Flood (SEF) would be 7 100 m^3/s (both un-routed values at this feasibility stage).

Following further analysis, it was confirmed that the freeboard requirements for the SEF would be the controlling case, and for the purposes of this spillway and dam options comparison, a total freeboard of 5.5 m was derived using the current SANCOLD guidelines on freeboard for dams (2011). See Appendix B.

Following further dam configuration considerations, a freeboard from spillway crest to NOC crest level of 4.83 m was used, with the difference of 0.67 m freeboard being accommodated using a 1 m high parapet wall.

For the case of a RCC dam option, it was considered that some overtopping could be allowed on the left flank crest as a result of wave run up under SEF conditions, since this dam type is concrete and it is considered to be more resilient to overtopping. Such details should also be revisited as a part of the dam configuration optimisation during the detailed design stage.

To illustrate the implication of the quantum of these flood flow rates, 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 200 m and 320 m.

As can be seen from the above cross-section, such a "conventional" spillway would constitute up to 88% 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 left 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 would result in an impractical and uneconomic structure for either the earthfill or rockfill embankment options.

In such embankment options, 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 left 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 rockfill and earthfill 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.

4.2.2 Spillway Design Approach

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

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

The spillways were compared for a full supply level (FSL) at 765.58 m.a.s.l. which is the proposed optimum dam capacity solution confirmed below.

a) *Spillway floods*

i. Flood peaks

The following flood peaks were used to size the spillway:

- Recommended Design Flood (RDF) (1:200 year RI): 3 500 m^3/s; and
- Safety Evaluation Flood (SEF): 7 100 m^3/s.

The selected flood peaks disregarded the routed flow attenuation effects of the dam basin for the sake of being conservative.

The feasibility design of the Lalini Dam allowed for a total freeboard between the full supply level and non-overspill crest of 4.83 m.

b) *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 4-2 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 = approach velocity head component of total energy head

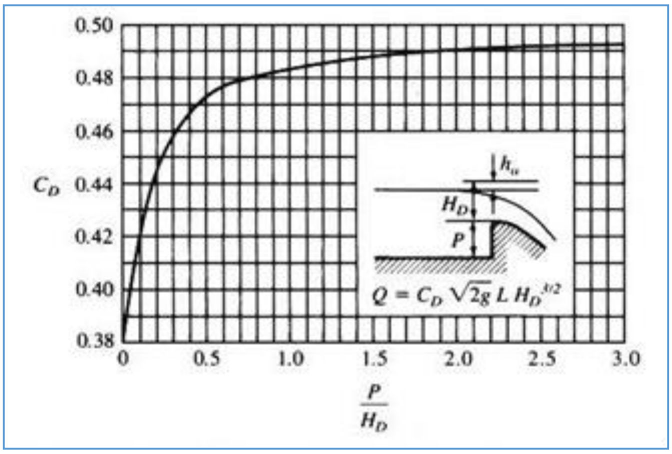


Figure 4-2: Ogee Discharge Co-efficient

As suggested in Figure 4-2, 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 4-3 illustrates the effect on the discharge coefficient under flow conditions other than the design flow. In order to size the ogee section of the spillway, a flow depth of 4.83 m was selected representing SEF conditions.

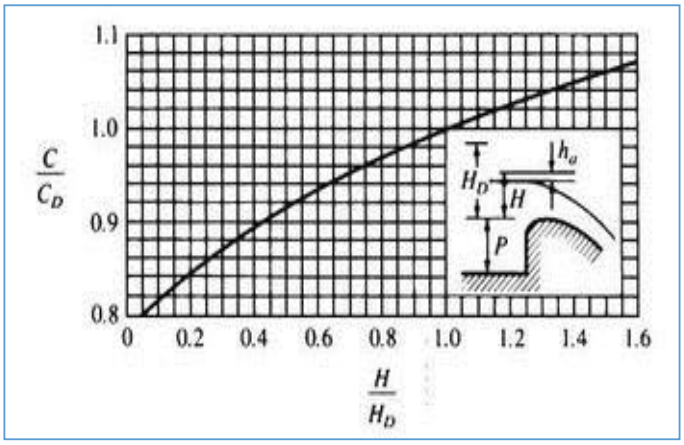


Figure 4-3: Change of Co-efficient Under Deeper Flow Conditions

c) *Spillway Crest Length*

i. Sizing
With the spillway height at 48.58 metres (P) and with a design flow depth of 4.83 metres (H) for the SEF, 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.

The ratio of spillway height (P) to the energy head (H_D) will then be 10.06 which represents an ogee discharge coefficient (C_D) of 0.492 (Figure 4-2).

Table 4-1 shows the depth and flow capacity for ogee spillways of various lengths, using the compound discharge coefficient of 0.492 x $(2g)^{0.5}$ = 2.1.

For a side channel spillway, a preliminary crest length of 300 m was 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 rockfill dam.

Table 4-1: Flow Depth and Capacity Table for Ogee Crest Spillway

Coefficient (C)	**2.179**

$$Q = C \times L \times H_o^{1.5}$$

			Capacity @ Crest Length = L m						
H0 =	**Q/m =**	**V=**	**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	28.8 8	5.1 6	288 8	361 0	**433 1**	**505 3**	577 5	649 7	721 9
5.8	30.4 4	5.2 5	304 4	380 5	**456 6**	532 6	608 7	684 8	760 9
6	32.0 2	5.3 4	320 2	**400 3**	480 4	560 4	640 5	720 6	800 6
6.2	33.6 4	5.4 3	336 4	**420 5**	504 6	588 7	672 8	756 9	841 0
6.4	35.2 8	5.5 1	**352 8**	441 0	529 2	617 4	705 6	793 8	882 0
6.6	36.9 5	5.6 0	**369 5**	461 8	554 2	646 6	738 9	831 3	923 7
6.8	38.6 4	5.6 8	386 4	483 0	579 6	676 2	772 8	869 4	966 0
7	40.3 6	5.7 7	403 6	504 4	605 3	706 2	807 1	908 0	100 89
7.2	42.1 0	5.8 5	421 0	526 2	631 5	736 7	841 9	947 2	105 24
7.4	43.8 6	5.9 3	438 6	548 3	658 0	767 6	877 3	986 9	109 66
7.6	45.6 5	6.0 1	456 5	570 7	684 8	798 9	913 1	102 72	114 13
7.8	47.4 7	6.0 9	474 7	593 3	712 0	830 7	949 4	106 80	118 67
8	49.3 1	6.1 6	493 1	616 3	739 6	862 8	986 1	110 94	123 26
8.2	51.1 7	6.2 4	511 7	639 6	767 5	895 4	102 33	115 12	127 91
8.4	53.0 5	6.3 2	530 5	663 1	795 7	928 4	106 10	119 36	132 62
8.6	54.9 5	6.3 9	549 5	686 9	824 3	961 7	109 91	123 65	137 39
8.8	56.8 8	6.4 6	568 8	711 0	853 2	995 4	113 77	127 99	142 21
9	58.8 3	6.5 4	588 3	735 4	882 5	102 96	117 67	132 37	147 08

Note: $C = C_D \times (2 \times 9.81)^{0.5}$

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 4-3, returns a 3.2 m flow depth over the spillway.

For the SEF, the depth of flow would increase to 4.83 m and if the dam embankment NOC is at this level then this represents a freeboard of 3.0 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 1.0 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 240 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 rockfill or earthfill 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 a 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.

d) *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. The options 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.

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 therefore considered that this would have to be sited at a high elevation above the river 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 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 earthfill and rockfill dam types, two spillway alignments were considered, as follows:

i. Spillway Option 1 (the “side-channel on 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 4-4.

ii. Spillway Option 2 (the “Cut-Through on left flank” (CT) Option) proposes an excavation through the hill upstream of the dam as indicated on Figures 4-5 and 4-6.

Given that I and ii 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 the stilling basin, and to avoid downstream ancillary works being affected by floods. This floodline was also determined using the HEC-RAS modelling software.

e) *RCC Dam Spillway Option (RCC)*

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 1V:0.75H, 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 200 m to 320 m was considered which limits the SEF unit discharge rate to between 22 and 35 $m^3/s/m$ which is considered an acceptable discharge rate, given the infrequency that such high discharges would occur. For comparison with other dam type options, a 320 m spillway crest length was used.

A spillway chute with varying step sizes and a slope of 1V to 0.75H was used to determine the costing of an RCC dam. Allowance was made for a significantly sized stilling basin.

As discussed above, if a 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.

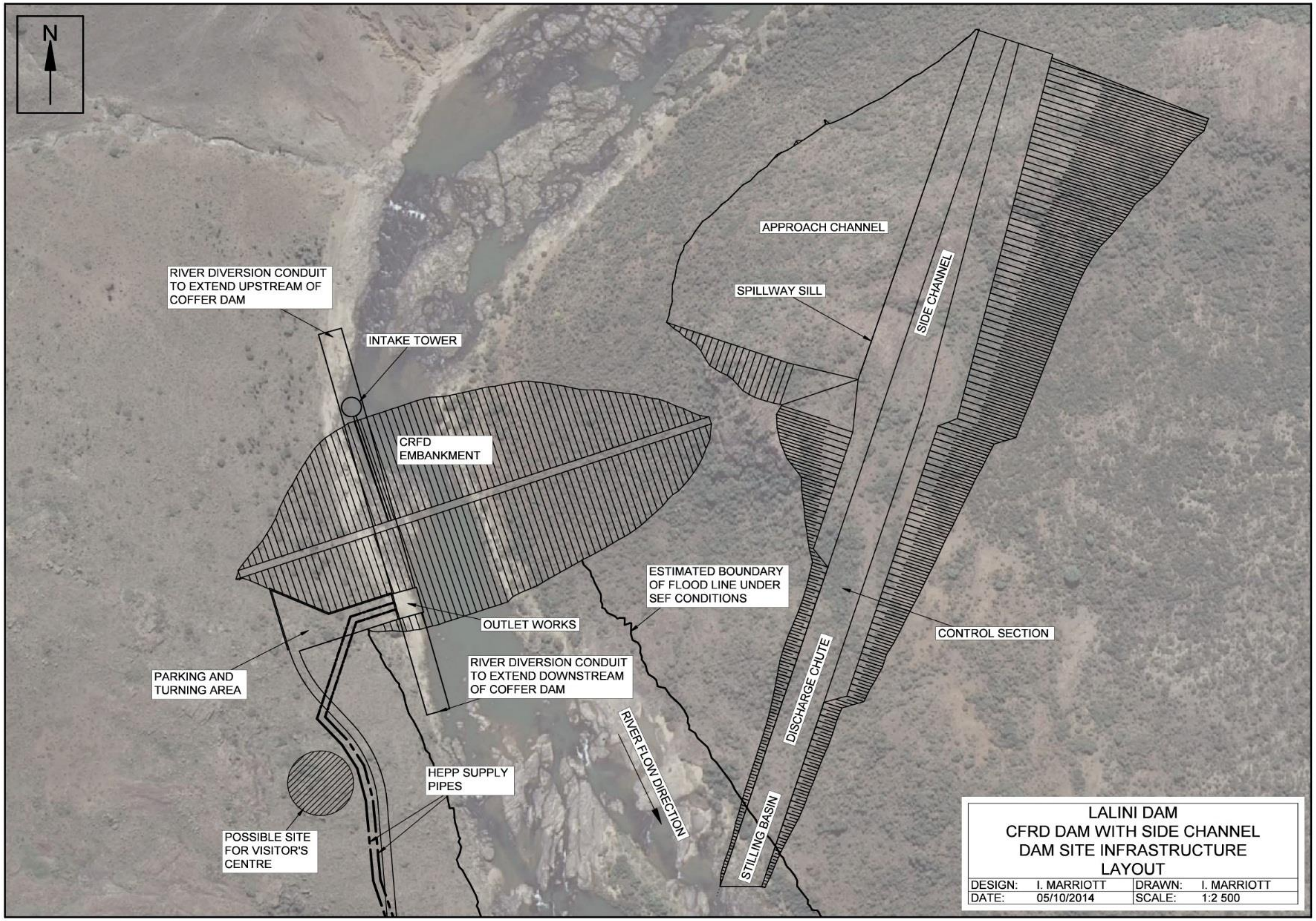


Figure 4-4: Side Channel Spillway Option Arrangement on Left Flank for CFRD

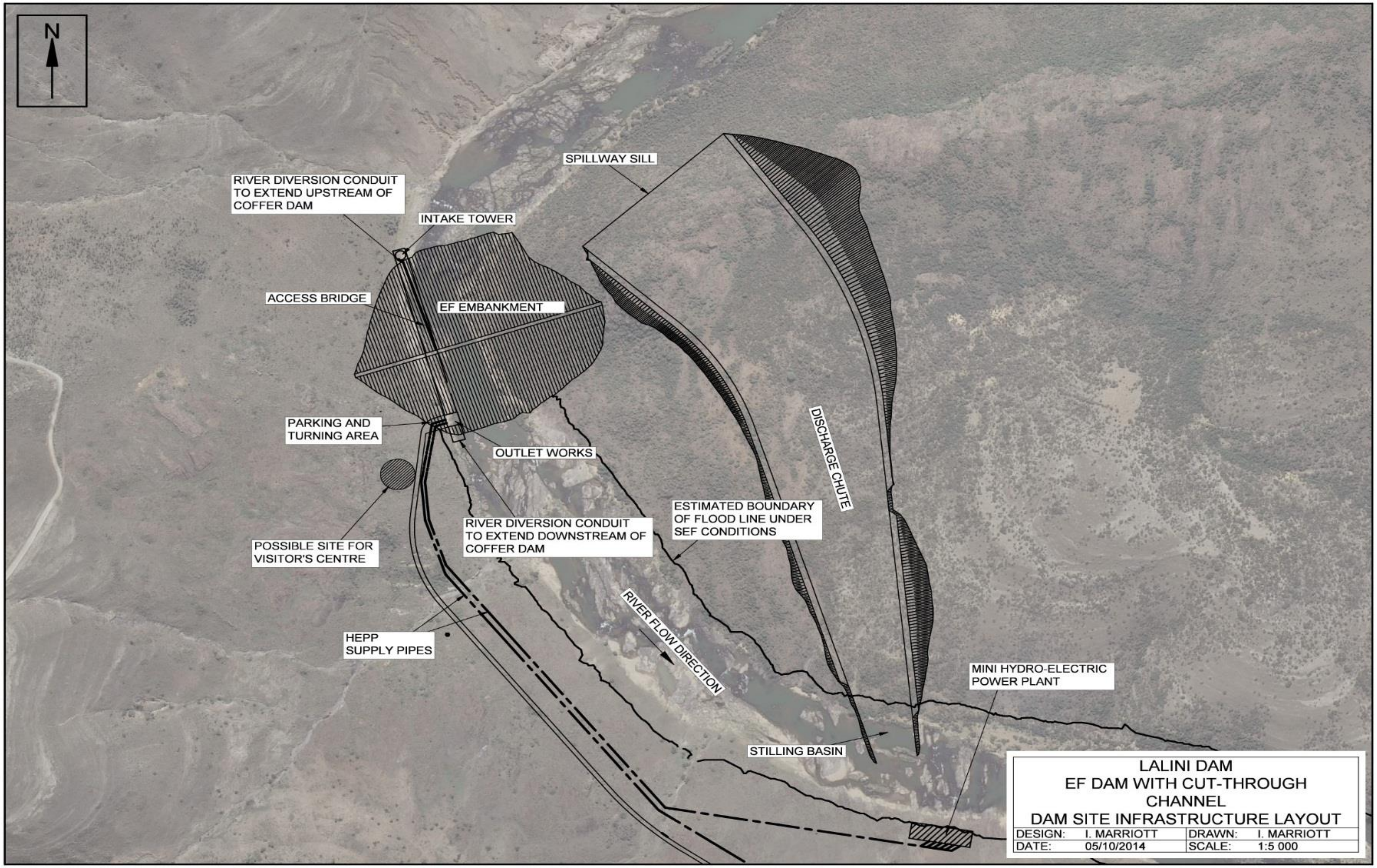


Figure 4-5: Off-Channel "Cut-Through" Spillway Option through Hill on Left Flank for EF

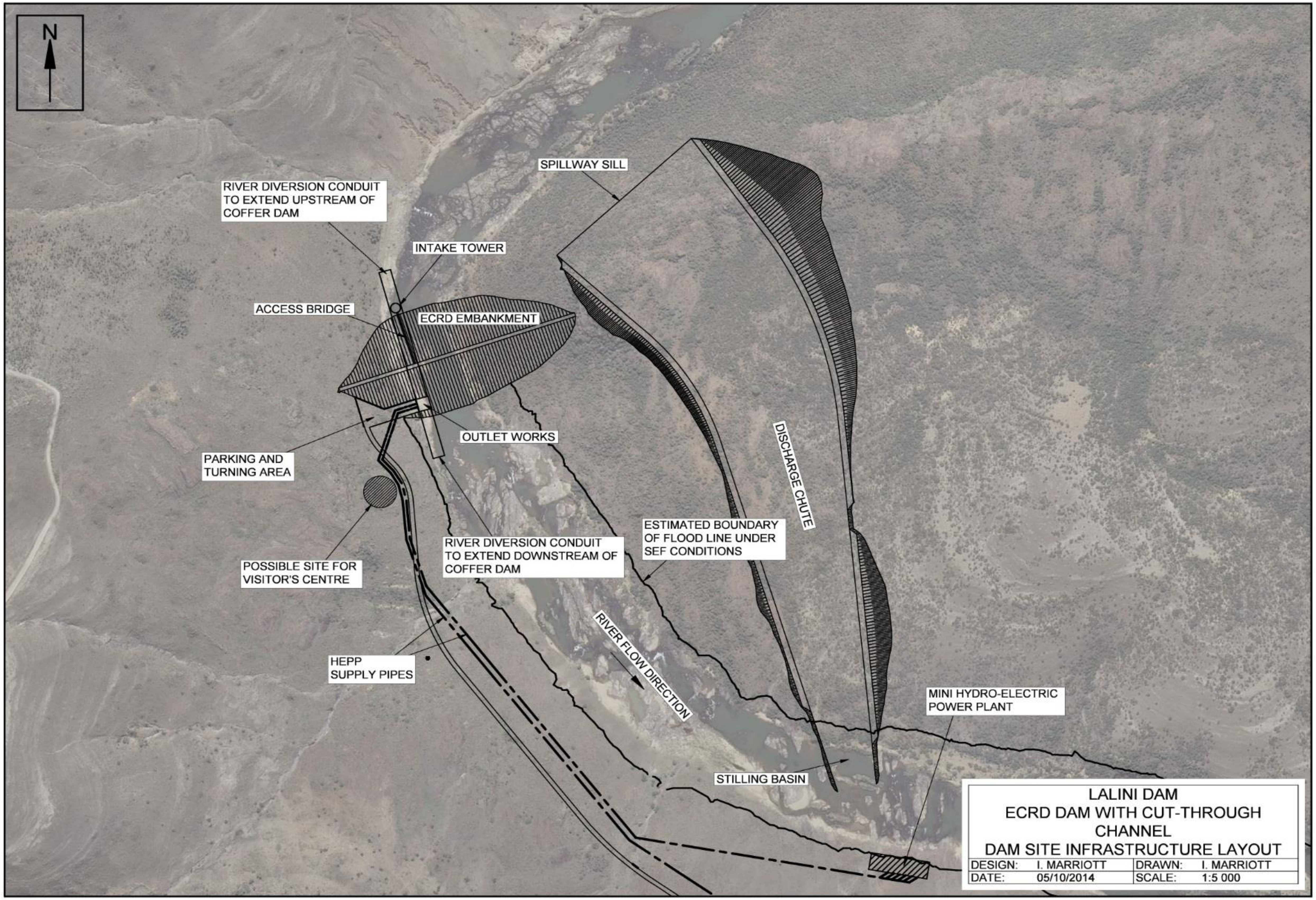


Figure 4-6: Cut Through Spillway Option Arrangement on Left Flank for ECRD

4.3 Other Considerations

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

An earthfill or rockfill 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 a 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.

4.4 Dam Construction Materials Requirements

For each dam wall type described above, cross-sections where prepared, primarily based on a full supply level (FSL) for the 0.3 MAR_{PD} capacity dam, plus a freeboard allowance of 4.83 metres, to determine the non-overspill crest level (NOC).

The freeboard allowance was based on the un-routed Safety Evaluation Flood of 7 100 m^3/s and a spillway crest length of 320 metres. Once again, these factors should be revisited in the detailed design stage, but are considered suitable for feasibility study purposes.

The typical dam sections and arrangements shown in Figures 4-7 to 4-10 were used to calculate the quantities of the various construction materials. The sections are based on previously designed and constructed dams of similar materials, sizes and types being investigated.

Materials properties were also very similar to those found during the Ntabelanga Dam feasibility study, hence similar embankment profiles were used for comparison purposes.

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

As a guide to the site investigations, approximate volumes required for the various potential alternative dam options were determined and listed in Table 4-2.

Table 4-2: Estimated Material Volumes for Alternative Dam Types

Dam Type	Rock/Rockfill	Shell (General Fill)	Core	Sand
Concrete-faced Rockfill (CFRD)	1 345 575 m³	n/a	n/a	17 000 m³
Earth Core Rockfill (ECRD)	1 100 181 m³	n/a	237 896 m³	33 357 m³
Earth Core Earthfill Embankment (EF)	17 078 m³	2 008 678 m³	262 914 m³	142 234 m³
Roller Compacted Concrete (RCC)	235 258 m³	n/a	n/a	39 210 m³

Concrete aggregate

Rockfill

Core

Filters

Rip-rap

Concrete sand

Earthfill

4.5 Construction Materials and Foundation Investigations

As reported in the Lalini 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.

Figure 3-4 above showed the interpretation of the founding conditions as identified through the core drilling undertaken on the proposed dam centreline. Figure 3-5 showed the finally selected materials quarries and borrow pits locations.

4.5.1 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 are indicated in the Geotechnical Investigations report, but a summary is described herein.

The boreholes drilled generally show a deep weathering profile over the area investigated with a thick overburden mantle, which under normal circumstances would render the site marginal to unsuitable for exploitation as a rock quarry, due to the excessive thickness of unusable overburden material that would require removal and spoiling. In this case, the residual and weathered dolerite overburden has potential usage as road construction material, which if confirmed as being suitable could make the site feasibly exploitable. This would require verification by means of a more detailed investigation and testing programme.

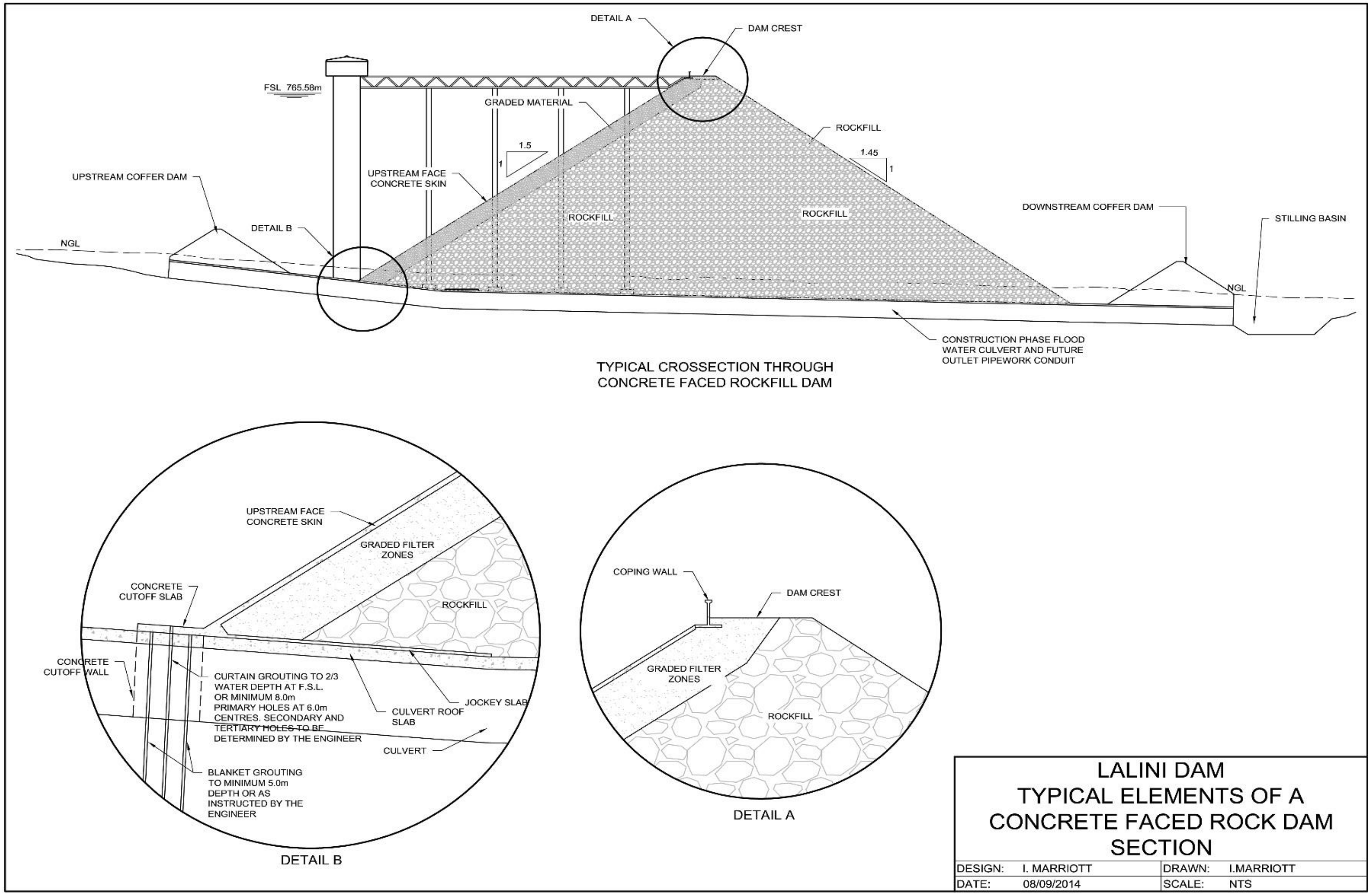


Figure 4-7: Typical Sections and Details for CFRD Type Dam

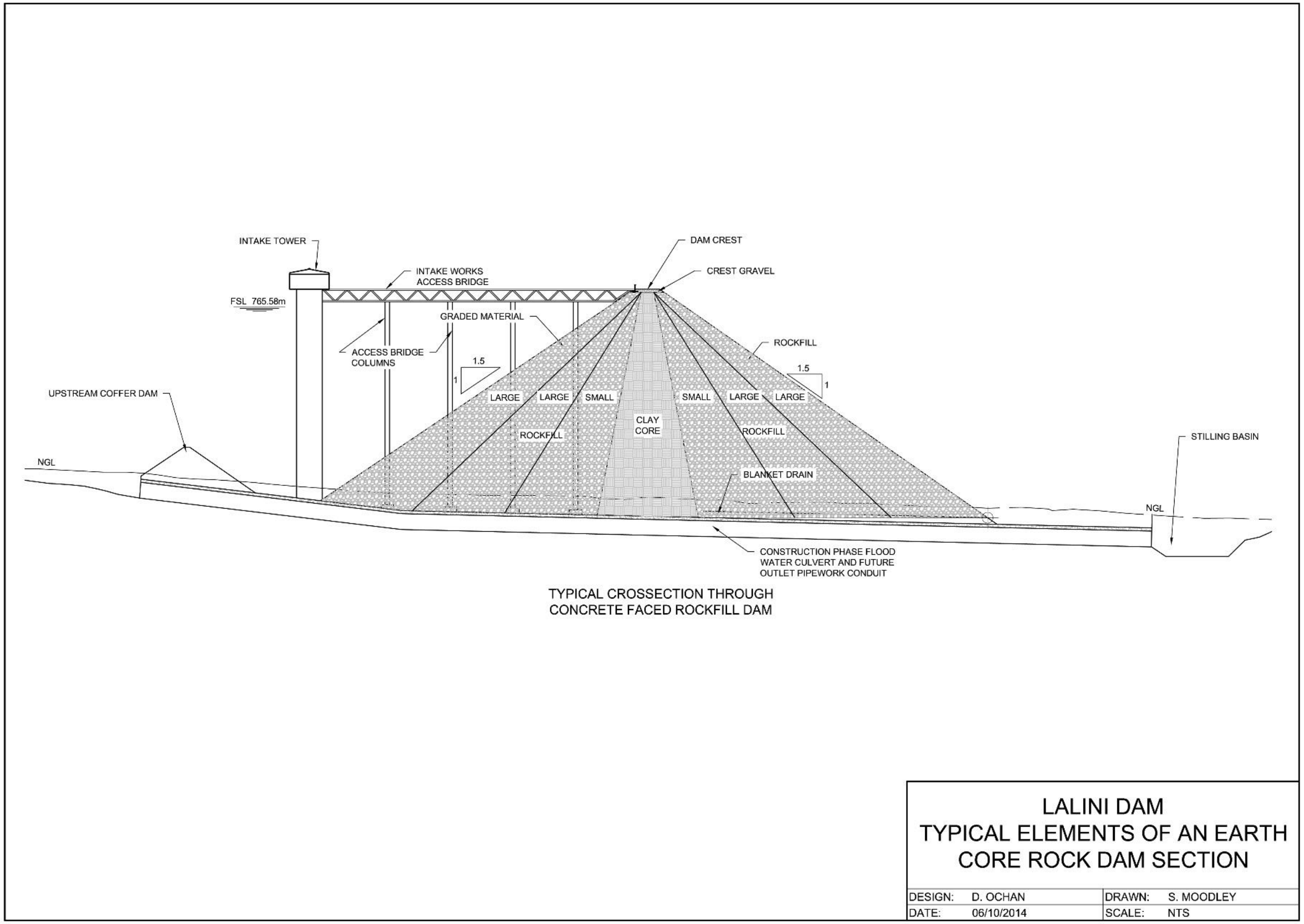


Figure 4-8: Typical Sections and Details for ECRD Type Dam

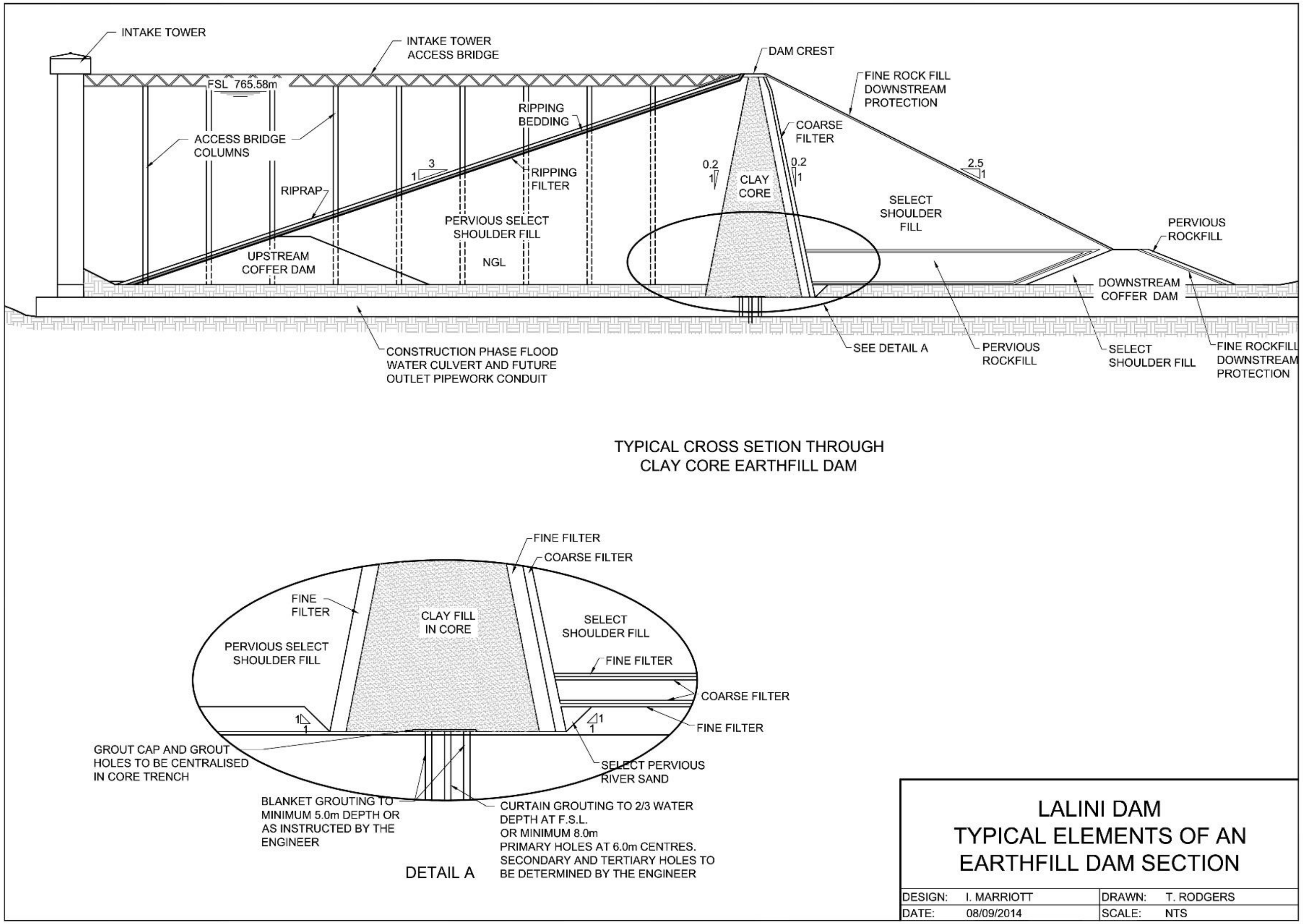


Figure 4-9: Typical Sections and Details for EC Earthfill Type Dam

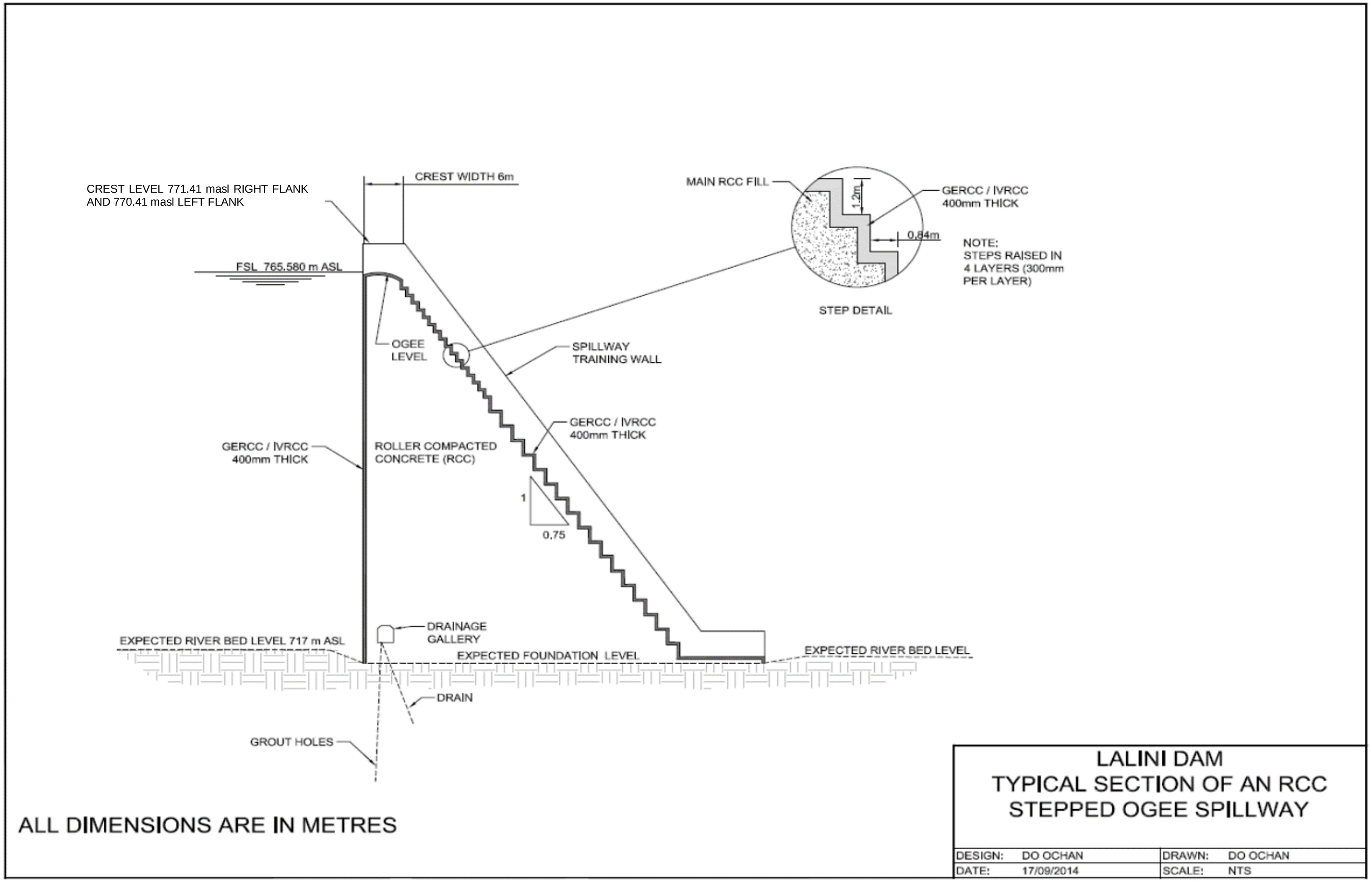


Figure 4-10: Typical Sections and Details for RCC Type Dam

The potential quarry site identified as having suitable dolerite material, (see Site C on Figure 3-5) and that is located within the future inundated basin is located approximately 3.5 km upstream of the dam on the eastern side of the Tsitsa River. Whilst this is some distance from the dam site, existing tracks can be developed by the contractor as temporary haul roads, which would all be drowned after construction. The drilling undertaken at this site indicated adequate rock aggregate for both dam and concrete structures construction.

Once encountered below the overburden, the un-weathered dolerite is of good quality, as confirmed by the strength, mineralogical and durability tests undertaken. The estimated volume of good quality dolerite rock available for the manufacture of crushed rock aggregates, excluding poor quality overburden, is in excess of 400 000 m^3 which id double that required for a RCC dam type.

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 rockfill and concrete aggregate purposes.

4.5.2 Sand for Concrete Aggregate and Filters

A stretch of the Tsitsa River, which lies within the impoundment basin has been proposed as a potential sand source. Sand samples were retrieved from within the river channel at various locations along this section of the river. The estimated volume of exploitable sand from this section of the river is approximately 960 000 m^3. This is in excess of what is needed for any dam type investigated. 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.

The laboratory test results carried out on the sand indicate that, chemically, the sand complies with the minimum requirements specified by SANS 1083 (2006) for fine concrete aggregate. However, the grading of the sand indicates the sand is too fine both for concrete and filter design (FEMA 2011). The current grading can be modified by blending the sand with crusher sand to comply with the above mentioned design standards.

An alternative to this would be to import sand from suitable sources a distance away from the dam site. It is recommended that this be noted when finalising the detailed design and the eventual contractor could be given the option of either blending or sourcing from offsite to achieve the correct grading at the lowest cost.

4.5.3 Clay Core Material

The proposed core material borrow pit is located at Site B on Figure 3-5, on the left flank, and less than a kilometre upstream of the dam. The area of potential exploitation was investigated and delineated by means of trial pits, sampling and testing. Seven trial pits were excavated across the proposed borrow pit by means of a tractor-loader-backhoe (TLB). The material comprises red-brown, sandy silty clay, colluvium of doleritic origin.

According to the USBR (1974), the relationship between the liquid limit and plasticity index plotted on the Casagrande plasticity chart and Laboratory test results, this material would be suitable for core material.

The estimated volume of exploitable materials from within the area investigated is of the order of 1 000 000 m^3, which is far in excess of the estimated project requirements of 270 000 m^3.

4.5.4 *Embankment Shell Material*

The proposed embankment shell or fill borrow pit is located at Site A on the right flank, less than a kilometre upstream of the dam, as indicated on Figure 3-5. The area investigated was confined to below the full supply level within the lower valley section of the impoundment basin.

The area of potential exploitation was investigated and delineated by means of trial pits, sampling and testing. Thirteen trial pits were excavated across the proposed borrow pit by means of a tractor-loader-backhoe (TLB). The material comprises weathered sedimentary rock of the Adelaide Formation, comprising mainly mudrock with subordinate, interlayered sandstone.

The grading of the materials indicated compliance with the grading specification for pervious shell, but the Atterberg limits (liquid limit, plasticity index and linear shrinkage) indicate a materials whose fines are too plastic for both pervious and semi-pervious fill. This will negatively affect the free draining characteristics required of a pervious and semi-pervious embankment. This material should however be suitable for construction of a cofferdam, which should have limited seepage without a clay core.

As previously mentioned, the investigation concentrated on the area of future impoundment, which is dominated by mudrock. In addition, due to the generally shallow excavation depths achieved using a TLB, the volume proved is 740 000 m^3, which is below the project requirements for shell.

Incorporating a higher proportion of sandstone, which is more prolific at higher elevations in the valley sides, has the potential to produce a material with a reduced plasticity and with increased free drainage. Weathered dolerite could also be considered, which is abundant in the area. This aspect of the investigation would require further assessment should an earthfill dam option be pursued.

4.5.5 *Spillway Materials Investigations*

As shown on Figures 4-4 and 4-5, two alternative spillway alignments on the upper left flank are proposed for the embankment dam type. It should be noted that due to budgetary constraints no investigation was carried out in the proposed spillway section. However the geotechnical reconnaissance gave a good indication of likely underground conditions at these locations, which expected excavation conditions were used in the costing analyses.

It is recommended that a detailed geotechnical investigation be carried out on the proposed spillway area if the embankment dam type proves to be the preferred option.

For a RCC dam alternative, the spillway would be designed as a central in-channel spillway. The aggregate for the RCC dam and spillway would require a rock aggregate source. Again, the ideal locality for rock aggregate would be the proposed quarry described above.

4.5.6 *Site Investigations 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 with minimum modifications. The exception is the embankment options for which large quantities of embankment shell material would have to be sourced from outside of the basin to meet the required volume of fill material. This will require significant haulage cost and potential environmental impacts.

Further site and materials investigations will be required to properly provide assurance for the detailed design process.

4.6 Dam Type Analyses

4.6.1 Embankment Stability and Seepage Analyses

As part of the dam type analyses, feasibility level assessments of dam stability and seepage are based on the analyses reported in P WMA 12/T30/00/5212/12 for the following three similar dam types arrangements; earthfill embankment with a clay core (EF), earth core rockfill dam (ECRD) and a concrete faced rockfill (CFRD) dam.

The roller compacted concrete dam (RCC) option has been checked for safety factors against overturning and sliding under SEF conditions. 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 the foundation than on the concrete itself.

Based on the geotechnical investigations reported under P WMA 12/T30/00/5212/10, 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 Rockfill Dams, by Martin Wieland and R. Peter Brenner*. The following conclusions were drawn:

"The main disadvantages of RCC are the following:

i) Watertightness: Due to the construction of the dam in thin cold 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 watertightness, 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 in the dam body, is tedious and slows down the construction process.

The main weaknesses of RCC dams are the watertightness 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 and Spring Grove dam in South Africa, and Metolong dam in Lesotho which is currently under construction.

The stability and leakage analyses undertaken on other dam types 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 analysis 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 analyses considered the following scenarios:

a) *Rapid Drawdown (RDD)*
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 dam wall.

It is not possible to ‘instantaneously drawdown the reservoir level as the outlet works would usually be designed to empty the dam over a period of two to three months. In terms of stability, RDD it is deemed to be a critical case.

b) *Seismic Event*
An earthquake event would cause cyclic dynamic loading of the embankment, predominantly in the horizontal direction. This may cause damage to the embankment but must not cause a total dam failure.

According to the seismic hazard map published in 2003 by the South African Council for Geoscience, Figure 4-11 (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.

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 been 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 for this local region.

From the research undertaken, indications are that, there have been some historical earthquake events in the area of influence. This could merit the consideration of analysis using higher risk factors than those published in SANS 10160-4, 2009.

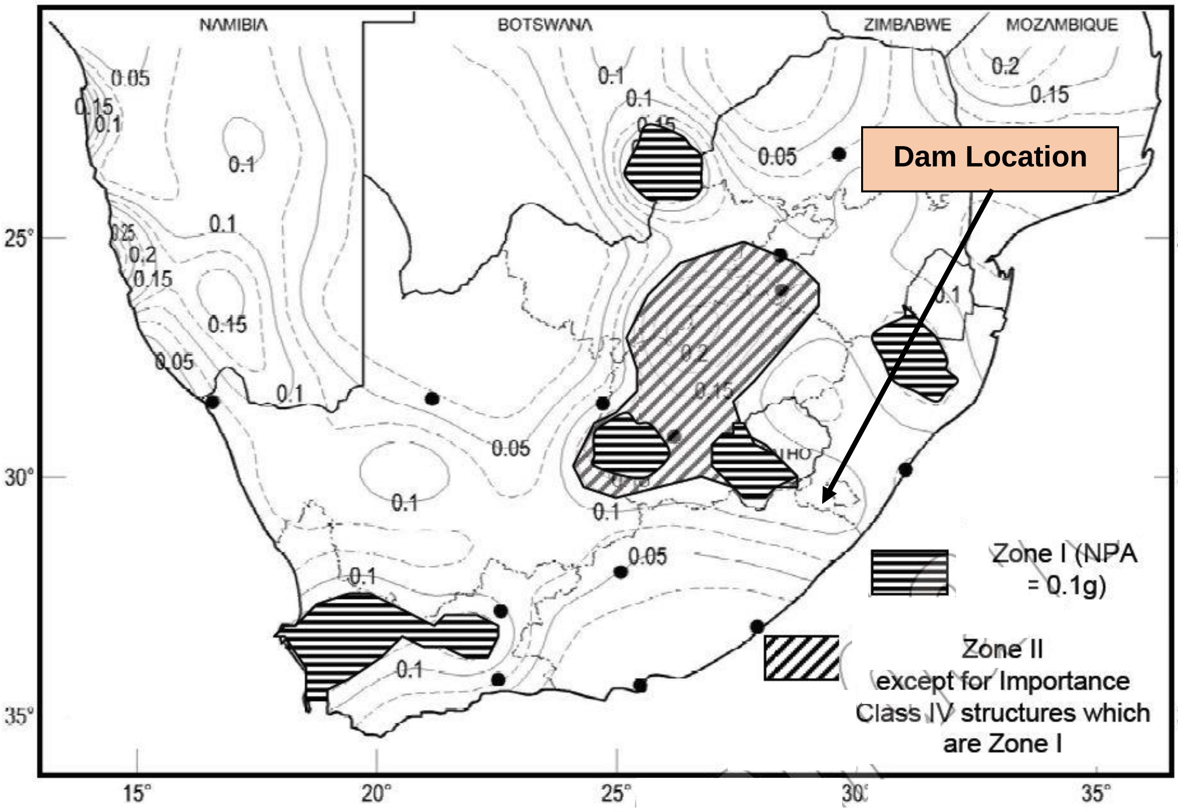


Figure 4-11: Peak Horizontal Ground Acceleration

The report and results of the above seismicity study are included as Appendix D to the Feasibility Design: Ntabelanga Dam Report No. P WMA 12/T30/00/5212/12.

An excerpt 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 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 regulations as per e.g. Eurocode 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 region 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 Mzimvubu dam sites in the Eastern Cape Province are:

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

"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 in report P WMA 12/T30/00/5212/12.

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 Lalini 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, if the embankment is raised too quickly, the build-up in pore pressure can result in lowering of the effective strength of the materials and can lead to failure.

e) *Full Supply Level (FSL)*
This is where the reservoir level is at its maximum operating level, and a steady state seepage condition exists within the embankment.

The recommended minimum acceptable factors of safety for each case discussed above are presented in Table 4-3.

Table 4-3: Recommended Factor of Safety

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

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

As discussed earlier, the laboratory test results available for the various construction materials motivated the use of the previous stability analysis, given that the geotechnical investigations reported similar engineering properties as were found in the Ntabelanga Dam feasibility analysis. More detailed site investigations during the detailed design stage will significantly improve the information available on the materials properties.

4.6.2 *Embankment Dams Stability and Seepage Analyses Findings*

Based upon the previous analysis of these embankment dam types undertaken, the results of the stability and seepage analyses are given in Tables 4-4 and 4-5.

These indicated that the typical embankment profiles selected in order to compare dam types were viable for feasibility design purposes.

Table 4-4: Summary of Calculated Factors of Safety

DAM TYPE	ANALYSIS DESCRIPTION	FACTOR OF SAFETY
Earthfill 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[3]
Earth Core Rockfill 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 Rockfill 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] *This did not meet the required factor of safety for the dam type and particular condition tested. This would require further optimisation of embankment design if this was the preferred dam type. However, the geometry used of this dam type was considered to be adequate for comparative costing analysis at this level of study.*

Table 4-5: Summary of Seepage per Dam Type

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

4.6.3 *RCC Dam Option Analysis*

Given the strong indications that the RCC option is likely to be a preferred option, a stability analysis was run to ensure that the typical dam profile being used for comparison purposes would be viable. CADAM software was used for the structural analysis.

Figure 4-12 shows a general plan view layout of the proposed RCC Dam.

The model was set up based on simple beam theory. This is a methodology mainly used for gravity dam design.

Figure 4-10 above showed the proposed cross section of the central uncontrolled Ogee spillway. This is considered to be the deepest section and for which the structural analysis was performed.

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

- Lalini Dam would have a maximum height of 53.41 m from the river bed level and a total crest length of 365 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;
- [4]Solid dolerite founding condition with minimum cohesion of 0.3 MPa and minimum angle of friction of 35°;
- Horizontal component of peak ground acceleration = 0.15 g; and
- Vertical component of peak ground acceleration = 0.08 g.

[4] *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.*

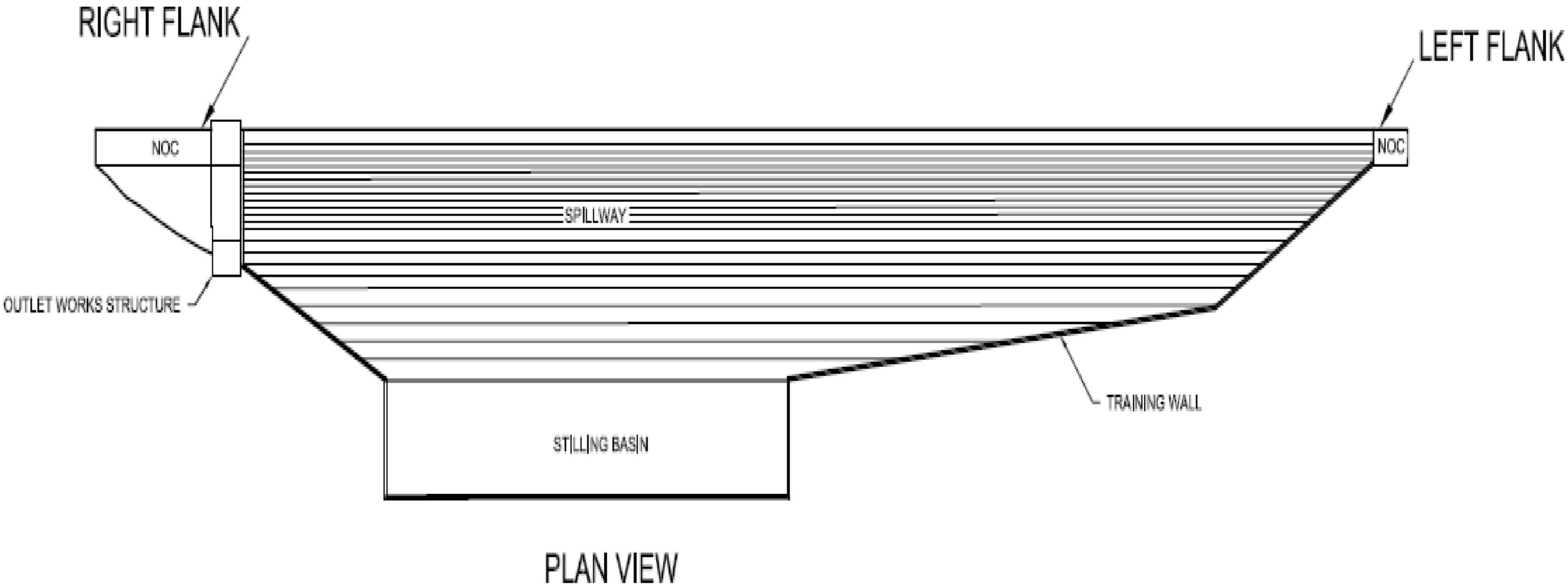


Figure 4-12: General Plan Layout of the Proposed RCC Dam

The loading conditions adopted are shown in Table 4-6.

Table 4-6: 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 4-7 and 4-8 present the results obtained from the various load cases in Table 4-6. 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 1V:0.70H and 1V:0.75H.

Table 4-7: Analysis Results and Comparison (1V:0.70H Slope)

Type	Case	Tensile Stress (MPa)		Compressive Stress (MPa)		Sliding (residual) Factor of safety (FOS)		Downstream overturing Factor of safety (FOS)	
		R	A	R	A	R	A	R	A
Normal	1	+0.15	0.0	-1.5	-3.0	1.4	1.5	1.32	1.5
	2	+0.35	0.0	-1.7	-3.0	1.3	1.4	1.3	1.4
Abnormal	3	+0.52	0.2	-1.7	-4.5	1.1	1.1	1.1	1.2
	4	+0.47	0.2	-1.8	-4.5	1.2	1.1	1.2	1.2
	5	-0.2	0.2	-1.0	-4.5	2.4	1.1	1.7	1.2
Extreme	6	-0.03	0.35	-1.3	-4.5	2.1	1.0	1.5	1.1
	7	+0.64	0.35	-1.8	-4.5	0.98	1.0	0.87	1.1

*<u>Legend</u> - **A** = Allowable - = Compression **R** = Result + = Tension*

Table 4-8: Analysis Results and Comparison (1V:0.75H Slope)

Type	Case	Tensile Stress (MPa)		Compressive Stress (MPa)		Sliding (residual) Factor of safety (FOS)		Downstream overturing Factor of safety (FOS)	
		R	A	R	A	R	A	R	A
Normal	1	+0.02	0.0	-1.3	-3.0	1.8	1.5	1.54	1.5
	2	+0.2	0.0	-1.5	-3.0	1.47	1.4	1.4	1.4
Abnormal	3	+0.34	0.2	-1.6	-4.5	1.2	1.1	1.2	1.2
	4	+0.3	0.2	-1.7	-4.5	1.3	1.1	1.3	1.2
	5	-0.32	0.2	-0.94	-4.5	2.7	1.1	1.8	1.2
Extreme	6	-0.15	0.35	-1.14	-4.5	2.3	1.0	1.6	1.1
	7	+0.48	0.35	-1.7	-4.5	1.1	1.0	1.1	1.1

*<u>Legend</u> - **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 1V:0.70H downstream slope option, and are conservative for the 1V:0.75H 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 1V:0.70H and 1V:0.80H, 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 Lalini Dam, a simple symmetrical profile as given in Figure 4-10 has been adopted, with a slope of 1V:0.75H. Outputs from the CADAM stability model runs on the RCC dam option are given in Figures 4-13 and 4-14.

4.6.4 River Diversion Works

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

Two different flood events were considered for the design of the diversion works. The 1 in 5 year flood of magnitude 750 m^3/s was used for the RCC dam type and the 1 in 20 year flood of magnitude 1 400 m^3/s was used for embankment dam types.

A diversion tunnel is a possibility but this was considered to 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.

CADAM - Stability drawing

Page 1

By Martin Leclerc, M. Ing.
NSERC / Hydro-Quebec / Alcan Industrial Chair on Structural Safety of Concrete Dams, École Polytechnique de Montréal, Canada

Project:		Dam location:		Analysis performed by:	
Dam:		Date:	26/09/2014	Project engineer:	
Owner:					

Usual combination (effective stress analysis)

Joint #	Crack (% joint)	Normal (kPa)	Principal (kPa)	Uplift (kPa)	Crack (% joint)	Normal (kPa)	Principal (kPa)	Uplift (kPa)
1		0.261	0.261	5.690		-8.227	-13.917	0.000
2		1.163	1.163	25.310		-36.597	-61.906	0.000
3		2.064	2.064	44.930		-64.966	-109.896	0.000
4		2.966	2.966	64.550		-93.336	-157.885	0.000
5		3.867	3.867	84.170		-121.705	-205.875	0.000
6		4.769	4.769	103.790		-150.075	-253.864	0.000
7		5.670	5.670	123.410		-178.444	-301.854	0.000
8		6.572	6.572	143.030		-206.813	-349.843	0.000
9		7.473	7.473	162.650		-235.183	-397.833	0.000
10		8.375	8.375	182.270		-263.552	-445.822	0.000
11		9.276	9.276	201.890		-291.922	-493.812	0.000
12		10.178	10.178	221.510		-320.291	-541.801	0.000
13		11.079	11.079	241.130		-348.661	-589.791	0.000
14		11.990	11.990	260.750		-377.040	-637.796	0.000
15		13.102	13.102	280.370		-405.620	-686.142	0.000
16		14.674	14.674	299.990		-434.659	-735.265	0.000
17		16.837	16.837	319.610		-464.291	-785.389	0.000
18		19.647	19.647	339.230		-494.568	-836.606	0.000
19		23.112	23.112	358.850		-525.502	-888.933	0.000
20		27.220	27.220	378.470		-557.078	-942.346	0.000
21		31.940	31.940	398.090		-589.266	-996.796	0.000
22		37.239	37.239	417.710		-622.033	-1052.224	0.000
23		43.079	43.079	437.330		-655.341	-1108.566	0.000
24		-101.372	-101.372	306.156		-689.151	-1165.761	0.000
25		-101.037	-101.037	319.302		-723.429	-1223.744	0.000
26	100.000	694.101	N/A	613.910		-1269.281	-1269.281	613.910

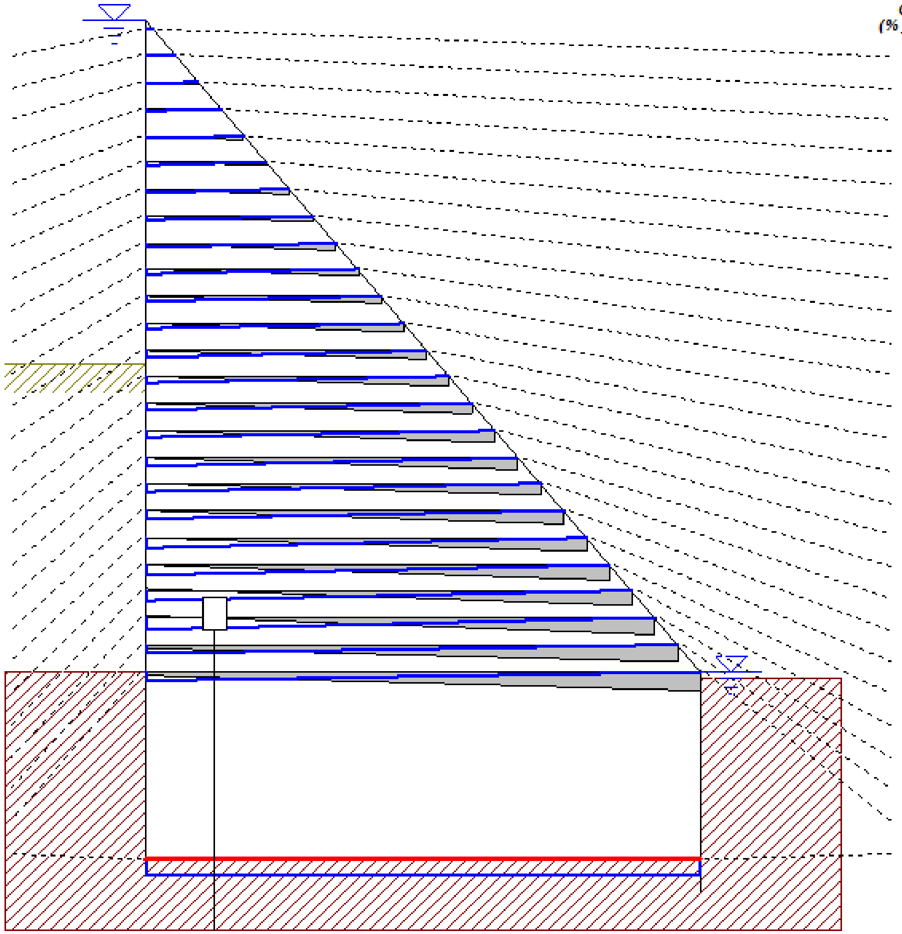


Figure 4-13: Stress Distribution on the Lift Joint Under Service Load

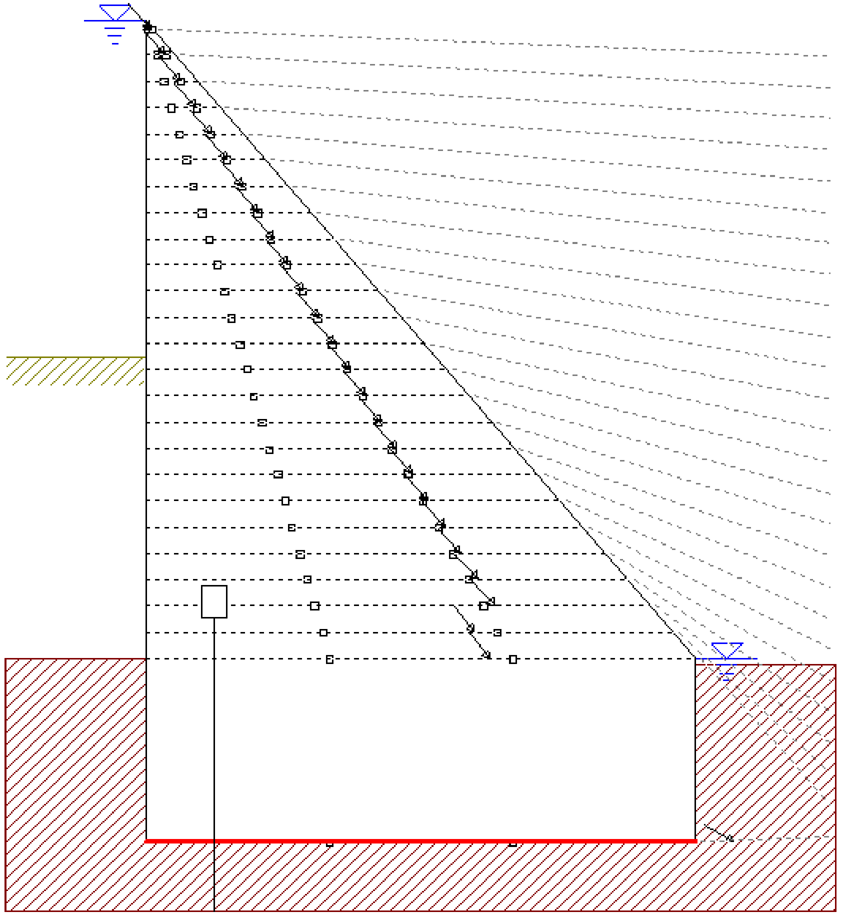
CADAM - Stability drawing

Page 2

By Martin Leclerc, M. Ing.
NSERC / Hydro-Quebec / Alcan Industrial Chair on Structural Safety of Concrete Dams, École Polytechnique de Montréal, Canada

Project:		**Dam location:**		**Analysis performed by:**	
Dam:		**Date:**	26/09/2014	**Project engineer:**	
Owner:					

Usual combination (stability analysis)



Joint #	SSF (peak)	SSF (residual)	OSF (U/S <-)	OSF (-> D/S)	USF	Normal (kN)	Shear (kN)	Moment (kN·m)	Res. Pos. (% joint)
1	> 100	1.164	3.846	1.393	2.400	-1.9	1.7	0.2	67.761
2	78.092	1.164	3.846	1.393	2.400	-38.0	32.6	14.5	67.761
3	44.717	1.164	3.846	1.393	2.400	-119.8	102.9	81.0	67.761
4	31.631	1.164	3.846	1.393	2.400	-247.3	212.4	240.3	67.761
5	24.645	1.164	3.846	1.393	2.400	-420.4	361.1	532.8	67.761
6	20.301	1.164	3.846	1.393	2.400	-639.2	549.0	998.9	67.761
7	17.337	1.164	3.846	1.393	2.400	-903.8	776.2	1679.2	67.761
8	15.187	1.164	3.846	1.393	2.400	-1214.0	1042.7	2614.2	67.761
9	13.556	1.164	3.846	1.393	2.400	-1569.9	1348.4	3844.4	67.761
10	12.276	1.164	3.846	1.393	2.400	-1971.4	1693.3	5410.2	67.761
11	11.244	1.164	3.846	1.393	2.400	-2418.7	2077.4	7352.1	67.761
12	10.396	1.164	3.846	1.393	2.400	-2911.6	2500.8	9710.6	67.761
13	9.685	1.164	3.846	1.393	2.400	-3450.3	2963.5	12526.2	67.761
14	9.075	1.163	3.846	1.393	2.400	-4034.6	3467.7	15840.1	67.761
15	8.515	1.158	3.847	1.393	2.400	-4664.6	4027.2	19711.4	67.779
16	7.998	1.150	3.849	1.392	2.400	-5340.3	4644.4	24216.4	67.831
17	7.526	1.140	3.853	1.390	2.400	-6061.7	5319.3	29432.6	67.921
18	7.096	1.128	3.858	1.388	2.400	-6828.7	6051.8	35437.3	68.046
19	6.706	1.117	3.864	1.386	2.400	-7641.5	6842.0	42307.9	68.200
20	6.352	1.105	3.872	1.383	2.400	-8499.9	7689.9	50121.6	68.379
21	6.032	1.094	3.880	1.379	2.400	-9404.1	8596.4	58955.8	68.577
22	5.741	1.083	3.889	1.376	2.400	-10353.9	9558.6	68887.8	68.789
23	5.478	1.073	3.899	1.372	2.400	-11349.4	10579.4	79995.0	69.012
24	5.863	1.313	5.833	1.685	3.582	-15311.2	11658.0	73498.8	62.392
25	5.648	1.302	5.848	1.680	3.582	-16654.2	12794.1	84653.7	62.582
26	0.852	0.597	1.993	0.965	1.468	-11618.6	19466.1	267046.0	106.892
	3.000	***1.500***	***1.200***	***1.200***	***1.200***				

Figure 4-14: Stability Analysis Results for Service Load

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 draw-off 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, again including delivery of water to the mini-hydropower generation plant building, and to the main hydropower plant water transfer conduit.

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. 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 a 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 several functions in the case of embankment dams, namely:

i. To deliver raw water to the outlet works supplying:
   a. 25 m^3/s peak flow to the main Lalini hydropower conduit;
   b. 16 m^3/s peak flow to the Lalini dam mini-hydropower plant located just downstream of the dam;
ii. up to 60 m^3/s flow to be released downstream for the EWR and to effect rapid drawdown if required for operational purposes, and
iii. to convey flood waters away from the works during construction

The size of conduit required to convey the flood condition would thus depend on the type of the dam and the construction sequencing as well as the eventual Contractor's approach and methodology. Analysis of river diversion works has been undertaken at a feasibility level of detail so that costs can be estimated for comparison purposes.

c) *River Diversion Conduit Sizing for Embankment Dams*

Using the depth verses capacity curve for the Lalini Dam, and calculating the hydraulic capacity for various cross-sections and lengths of conduits below the dam wall, modelling was undertaken to route a 1 in 20 year (1 400 m^3/s) peak flood hydrograph through the reservoir with a duration of 72 hours, 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.

Figure 4-15 below shows an example of the flood routing model outputs.

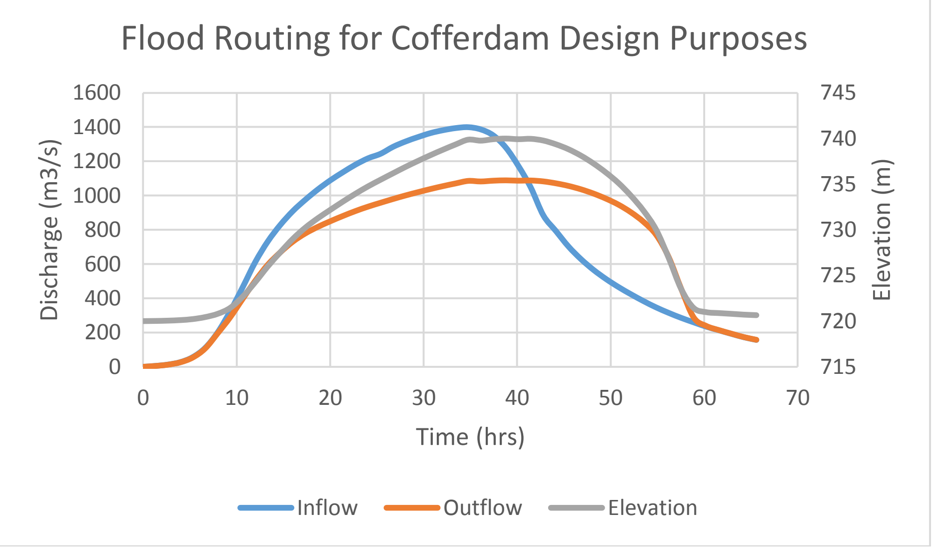


Figure 4-15: Example Routing Curve for 1:20 years Flood Condition

The analysis was repeated for a range of conduit sizes from 3.0 m to 5.4 m diameter.

A pair of 5.4 m diameter conduits would, under the 1 in 20 year flood condition, produce a rise in upstream water level of some 27 m above river bed level. This requires a very high cofferdam.

Three 5.4 m diameter conduits would, under the 1 in 20 year flood condition, produce a rise in upstream water level of some 20 m above river bed level. This is considered acceptable for cofferdam.

At this feasibility level of analysis, for the purposes of comparing of embankment dam options, costing of the river diversion works for all embankment dam options were based on the 3 x 5.4 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 1 400 m^3/s for the embankment type of dam to some 600 to 750 m^3/s (depending upon the flood assessment method), and a 750 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.

4.6.5 Outlet Works Capacity to Discharge EWR Floods/Freshets

The EWR values and release rules thereof were only developed at the end of this study period following an additional reserve determination exercise undertaken through the separate EIA study. The recommendations made, following a basic assessment, were as shown in Table 4-9.

Whilst the proposed 60 m^3/s capacity of the flood release/rapid drawdown facility meets Class 1 and 2 flood release requirements, outlet cannot meet the requirements for Class 3 and 4.

In recent years, there has been ever increasing attention paid to the flood/freshet releases aspects of large dam design. The installation of outlet works capable of discharging high flood values are costly and must be designed and operated with great care. For example, the Berg River dam has a flood release capacity of 200 m^3/s, and some vibration caused by transient pressure was experienced which required further studies and remedial actions to be undertaken.

The Lalini Dam hydropower simulations indicated that the dam would spill only 74 times in 1080 months, which spills would likely not always be sufficient to meet the Class 3 and Class 4 flood release requirements as described in Table 4-9. The main question is how large to size the flood outlet works capacity.

Table 4-9: Recommended EWR Flood Rules for the Tsitsa River below Lalini Dam

Floods	Flood size (range) m^3/s	Fish	Invertebrates	Vegetation	Geo-morphology	Actual Flood Value in SPATSIM
Class 1	0-10				10 m^3/s Average 10 days	10 m^3/s Average 10 days Sep, Oct, Nov, 2 x Dec, 3 x Jan, 2 x Feb
Class 2	11-25	25 m^3/s Average 4 days	20 m^3/s Average 4 days	20 m^3/s Average 4 days	20 m^3/s Average 6 days	25 m^3/s Average 6 days Sep, Oct, Nov, Dec, 2 x Jan, Feb, Mar
Class 3	100-170	100 m^3/s Peak 6 days	170 m^3/s Peak 5 days	150 m3/s Peak 6 days	200 m^3/s Peak 4 days	170 m^3/s Peak 5 days Feb
Class 4	200-350			200 m^3/s Peak 6 days		200 m^3/s Peak 4 days Mar

Recent studies have been undertaken on this subject on the Smithfield Dam, which were reported at SANCOLD 2014 in the paper entitled "Evaluating the sizing of the outlet infrastructure of Smithfield Dam to accommodate EWR flood flow releases"[5].

The paper discussed the optimum sizing of the dam outlet works comparing the designed "limited" outlet capacity of 41 m^3/s with the "unlimited" peak EWR flood requirement of 235 m^3/s. The modelling ran scenarios for various flow release trigger levels based upon precedent inflow and dam water level taking into consideration that it would not be normal to release large floods in a drought year.

Given that the distribution of flow duration curve of that dam was heavily skewed such that flows above 41 m^3/s only occurred with a probability of occurrence of less than 3%, it was concluded as follows:

"The impact of outlet capacity limits on EWR for Smithfield Dam:

- *Negligible difference in EWR supply between unlimited and limited dam release capacities*
- *EWR target and supply volumes are identical above an exceedance probability of 3%*
- *Undersupply is less than 10% of EWR target*

In comparing a limited 41 m^3/s verses the maximum 235 m^3/s outlet capacity, the conclusion was that there was:

- *only marginal improvement in benefit to the downstream environment if a maximum flood outlet capacity was installed,*
- *such a large outlet works increased construction difficulties, and*
- *this could result in massive financial over-expenditure."*

[5] *J Lombard, FGB de Jager & E van Niekerk, AECOM*

The B/C Class EWR and naturalized flow duration curves at Lalini Dam for the wettest month of March are given in Figure 4-16.

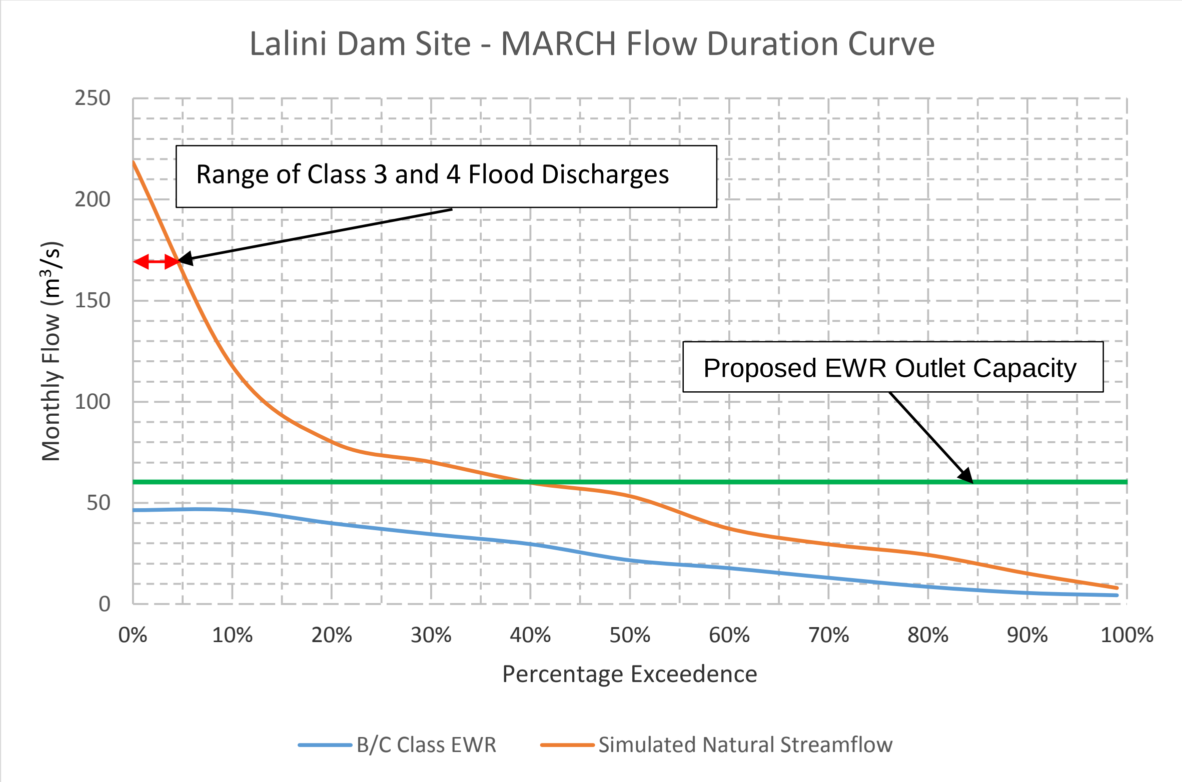


Figure 4-16: Flow Duration Curves at Lalini Dam for March

As can be seen the 60 m^3/s outlet works capacity are more than adequate to meet the Class B/C EWR at all times, and the percentage occurrence of flows in the range of 170 m^3/s and above (see Table 4-9) are less than 4%. This is a similar situation to the Smithfield Dam example.

Given the above, it is recommended that more detailed consideration of this issue be undertaken during the detailed design stage of scheme implementation, before a final decision is made on the optimum Lalini Dam flood release outlet works capacity. Discussion of this issue at SANCOLD 2014 included suggestions that the flood regime should be modelled in more detail – probably at a finer resolution by employing daily flood simulation modelling – and consensus reached with the reserve determination team as to the flood release rules that would trigger the various classes of floods, and the impact of limiting the installed flood outlets capacity to less than the peaks indicated under Classes 3 and 4.

If there is still an insistence that these larger flood/freshet releases be catered for, then a single dedicated larger gated outlet would need to be incorporated into the detailed design. The capacity of this facility would be designed to release the incremental discharge above 60 m^3/s. This outlet facility would typically take the form of a rectangular conduit through the body of the dam, which is controlled by a downstream radial or vertical gate with an upstream vertical service gate (refer EWR outlet on Midmar Dam). The service gate would remain closed during normal operation of the dam as an additional safety measure.

4.6.6 *Other Operational Aspects*

Figure 4-17 shows a typical operational water level trajectory for Lalini Dam taken from the hydropower simulation modelling.

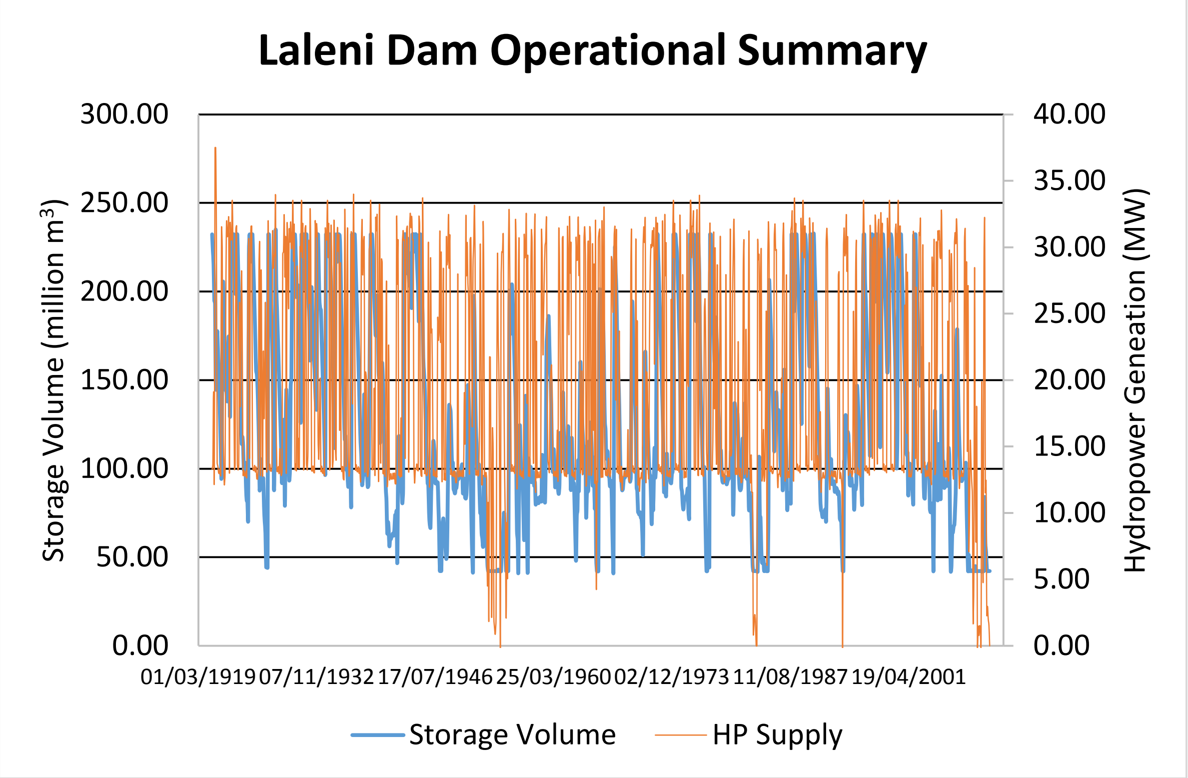


Figure 4-17: Example of Operational Regime of the Lalini Dam

The blue plot shows that the dam will draw down and fill frequently as the balancing storage in the dam is used to its full advantage. This has implications in that the reservoir will not always be very full and its amenity value as a recreational lake must therefore be considered to be limited.

This will also impact the local access of the dam waterline as it rises and falls and may leave tracts of muddy sediment along the shoreline which could be a risk to people and livestock. This may require consideration of the prevention of access to these areas and the provision of dedicated hard access points for livestock (refer to cattle access points provided along the Ash River between Clarens and Bethlehem).

This also emphasizes the need for multiple draw-offs to be installed so that the quality of water abstracted for hydropower generation and released downstream can be appropriately selected by discretionary usage of particular draw-off levels.

4.6.7 *Eel Ladder*

The environmental study has found evidence that, having spawned in the sea, eels migrate up the Mzimvubu and Tsitsa Rivers and have been observed in the Tsitsa River above the Lalini Dam locality. This means that young eels have managed to climb and traverse the Tsitsa Falls.

It is recommended that this be further investigated during the detail design stage and if an ‘eel ladder’ is found to be a necessity, this detail should be incorporated into the detailed design.

5. SELECTION OF PREFERED DAM TYPE AND SPILLWAY OPTION

5.1 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.

The cost models consider not only the dam wall and spillway costs, but also take into account the preliminary estimates of costs of outlet works, stilling basins, other associated works, access roads, existing road and bridge realignments, 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 variation 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 a RCC mix, a costing was developed for both low and high paste solutions from basic principles. This costing takes 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 on-going dam construction projects, including the following dams:

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

The sensitivity analyses carried out on the major cost items included soft and hard excavation, reinforced concrete, steel works, RCC, embankment shell 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 Lalini 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 unit rate scenario.

Table 5-1 below summarises the results of this analysis.

Table 5-1: Summary of Lalini Dam Types Cost Comparisons

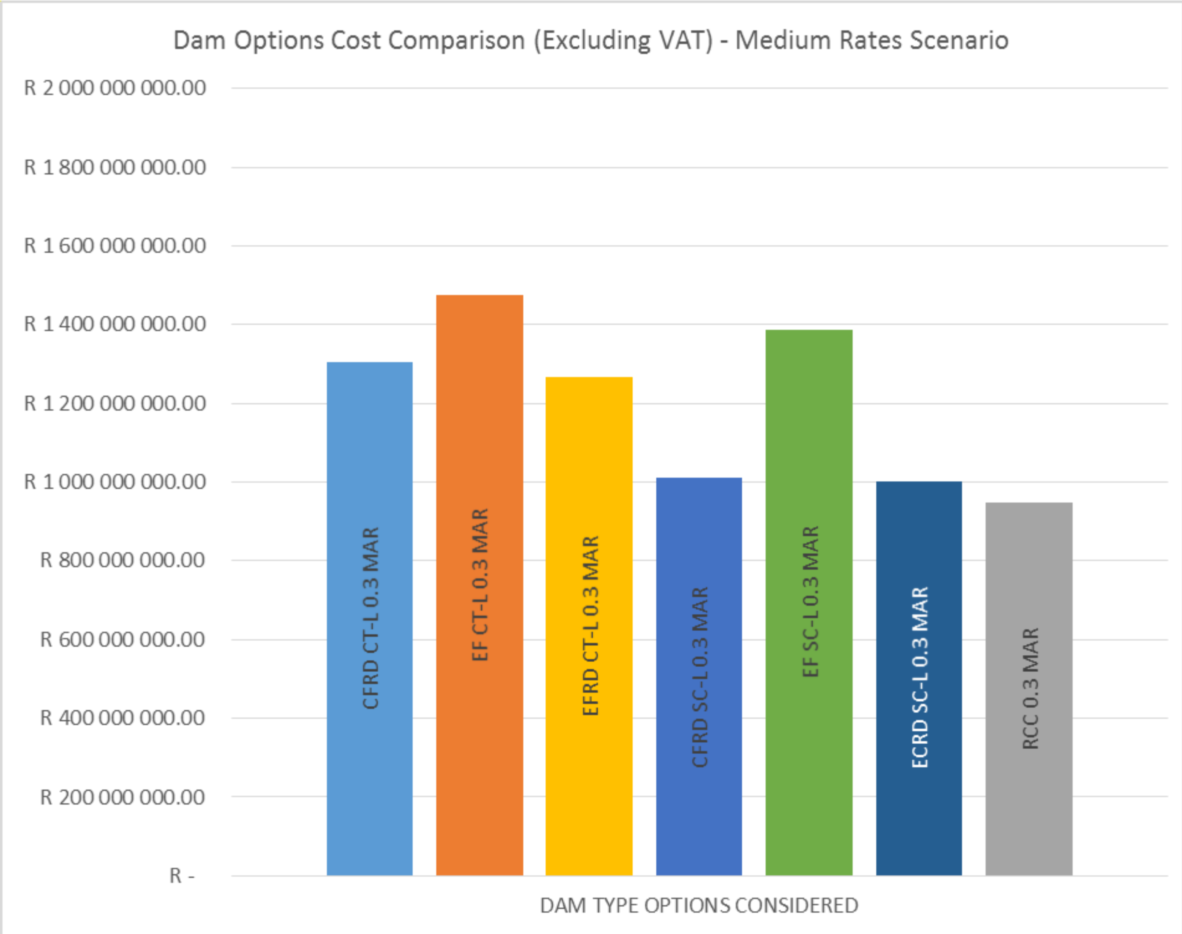
Option No.	Dam Wall Type	Spillway Type	Option Nomenclature	Estimated Capital Cost R'million (excl VAT)		
				Low	Medium	High
1	Concrete Faced Rockfill Dam (CFRD)	Cut-Through on Left Flank (CT-L)	CFRD CT-L 0.3 MAR	1206	1304	1402
2	Concrete Faced Rockfill Dam (CFRD)	Side Channel on Left Flank (SC-L)	CFRD SC-L 0.3 MAR	924	1010	1095
3	Earth Core Rockfill Dam (ECRD)	Cut-Through on Left Flank (CT-L)	ECRD CT-L 0.3 MAR	1178	1268	1358
4	Earth Core Rockfill Dam (ECRD)	Side Channel on Left Flank (SC-L)	ECRD SC-L 0.3 MAR	923	1002	1081
5	Earthfill Dam with Earth Core (EF)	Cut-Through on Left Flank (CT-L)	EF CT-L 0.3 MAR	1385	1475	1564
6	Earthfill Dam with Earth Core (EF)	Side Channel on Left Flank (SC-L)	EF SC-L 0.3 MAR	1296	1386	1475
7	**Roller Compacted Concrete**	**Central Ogee**	**RCC 0.3 MAR**	**826**	**947**	**1069**
				Lowest		
				Second Lowest		

The green highlighted cells show the lowest cost option, which is, for all ranges of major quantity unit rates, Option No. 7, a RCC dam. Option No.4, the ECRD dam with a side channel spillway cut through on the left-hand flank, is second lowest. Even for the highest rates, this ranking remains the same.

Figure 5-1 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.

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, CFRD (with left hand side channel spillway) and ECRD (with left hand side channel spillway) options are ranked very closely, within 7% of each other. All other options are more than 33% higher in cost.

It is therefore concluded that, within the feasibility confidence levels of cost estimation, there is little to choose between the three lowest ranked options as far as costs are concerned, and other factors are therefore considered to inform the decision-making process.



Percentage of lowest cost option	138%	156%	134%	107%	146%	106%	100%
Cost Excluding VAT R'millions	1 304	1 475	1 268	1 010	1 386	1 002	947
Total rock excavation used in embankment	1,350,000	17,000	1,100,000	1,350,000	23,000	1,100,000	N/A
Total rock excavation to spoil	3,534,000	5,090,000	3,785,000	1,436,000	2,952,000	1,779,000	N/A
Total all materials to spoil	3,644,000	5,090,000	3,796,400	1,436,000	2,952,000	1,779,000	N/A

Figure 5-1: Dam Options Cost Comparison

5.2 Other Factors considered in Decision-Making for Dam Type Selections

The following considerations were made:

- The speed of implementation to first water delivery;
- Simplified infrastructure layout and access;
- Low maintenance inputs;
- Less risk for the river diversion during construction, and
- Minimum environmental and aesthetic impacts.

5.2.1 *Speed of Implementation*

One of the advantages of a RCC solution over the embankment dam is faster speed of construction. This can lead to significant cost savings provided that proper planning and management procedures are followed. It is estimated that a RCC dam could be constructed approximately one year faster than the equivalent capacity embankment type of dam. This means that if the associated works can also be completed in the same time period as a RCC dam, then power could be generated a year earlier, thus earning additional revenue of more than R 150 million in that year.

5.2.2 *Low Maintenance Inputs*

Generally, an all-concrete solution such as a RCC dam, has 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.

5.2.3 *Low Risk with River Diversion During Construction*

A 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 (embankment type dams) had unforeseen, and previously unrecorded flood events which damaged the works under construction, and delayed the completion of the works, with consequential increased costs.

5.2.4 *Minimum Environmental Impacts*

An ECRD will require more rock excavation than the RCC dam option, and would source such rock from the left bank side channel spillway, whereas the rock for concrete aggregate for the RCC dam would be sourced from a quarry which 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, which will probably need to be obtained from commercial sources and not from the river basin.
Once again, whilst the temporary environmental impacts of the winning 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.

In terms of aesthetics, a RCC solution would have less impact as it would not require a large rock excavated spillway channel in the left flank hillside, which leaves a permanent scar. This large excavated channel would also have a significant impact on protected and threatened faunal and floral species, as reported in the Environmental Impact Report.

5.3 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.

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

An impression of what the Lalini Dam would look like is given in Figure 5-2.

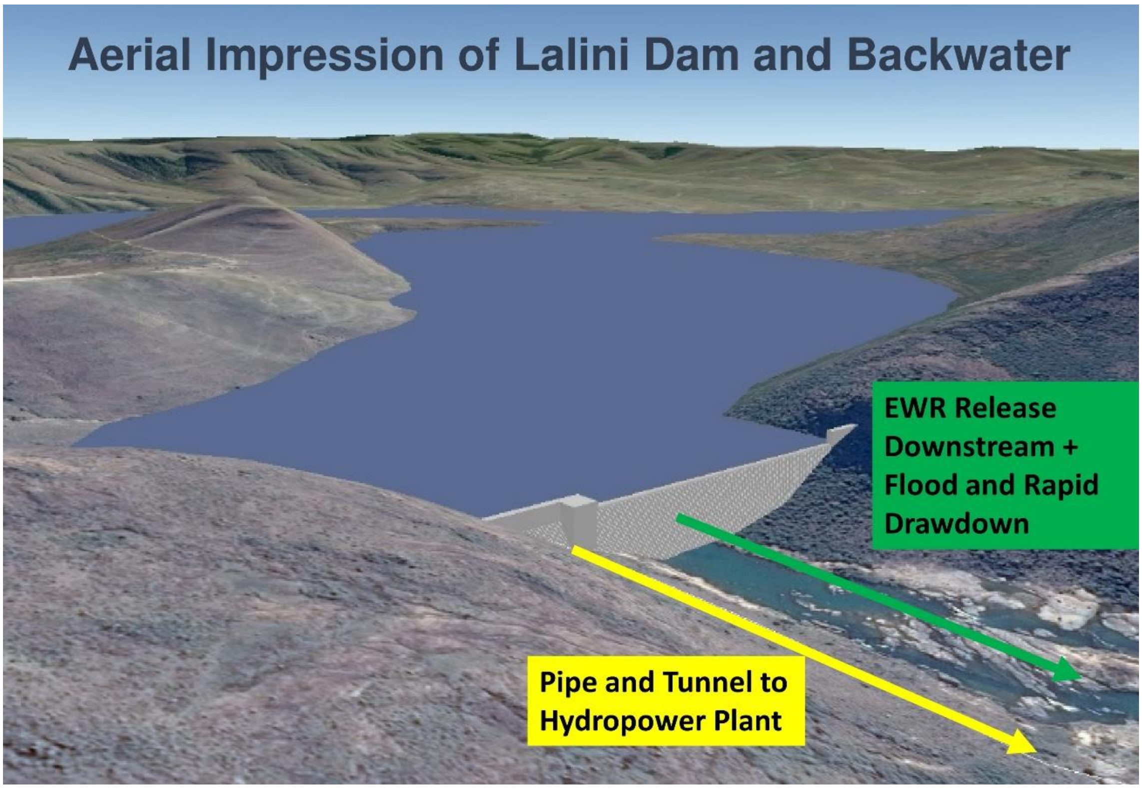


Figure 5-2: Impression of Lalini Dam

6. FEASIBILITY DESIGN: LALINI DAM, SPILLWAY AND RELATED WORKS

6.1 Dam Wall and Spillway

As described in the preceding sections, an RCC gravity dam is recommended, with an ogee spillway with stepped downstream face, and a slope of 1V to 0.75H, with a gradually varied step dimensions. The step dimensions could be in smaller increments in the upper area to reduce nap separation, but this must be verified and refined by spillway modelling in the detailed design stage.

The proposed layout plan, typical wall and spillway cross-sections, and longitudinal cross-sections for the recommended dam type and spillway are shown in Figures 6-1 to 6-3.

The proposed Lalini Dam has the following characteristics:

Full Supply Level (FSL):	765.58 m.a.s.l.
Non-Overspill Crest Level[6] – Left flank (NOCL):	770.41 m.a.s.l.
Minimum bed level in river at dam:	717.00 m.a.s.l.
Crest width:	6 m
Minimum operating level (MOL):	740.14 m.a.s.l.
Main outlet conduit minimum invert level:	736.14 m.a.s.l.
Maximum dam wall height to NOC:	53.41 m
Wall crest length (incl spillway):	365 m
Spillway crest length:	320 m
Gross stored volume at FSL:	232 million m^3
Mean Annual Runoff (Present Day) at dam:	828 million m^3
Storage below MOL (V_{50} sedimentation):	31.2 million m^3
Surface area of lake behind dam:	31.5 km^2
Backwater reach upstream of dam:	22 km

The dam wall height, impoundment volume, and downstream risk factors for the Lalini Dam put this structure into a Category 3 dam under Gazetted Dam Safety regulation R139 of 2012.

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 (RDF):	3 500 m^3/s
Safety Evaluation Flood (SEF):	7 100 m^3/s

The dam releases flow into the river below the dam to meet the EWR requirements, which flow can be simultaneously used to generate an average of 1.8 MW of hydropower at the dam wall.

The dam also transfers water by gravity through a pipeline, tunnel and penstock system to provide water to the main Lalini hydro-electric power scheme (HEP), which can generate up to 37.5 MW, before releasing this water back into the river below the HEP return flow outlet works.

[6] *Right-hand flank NOCL is 1 m higher than this flank*

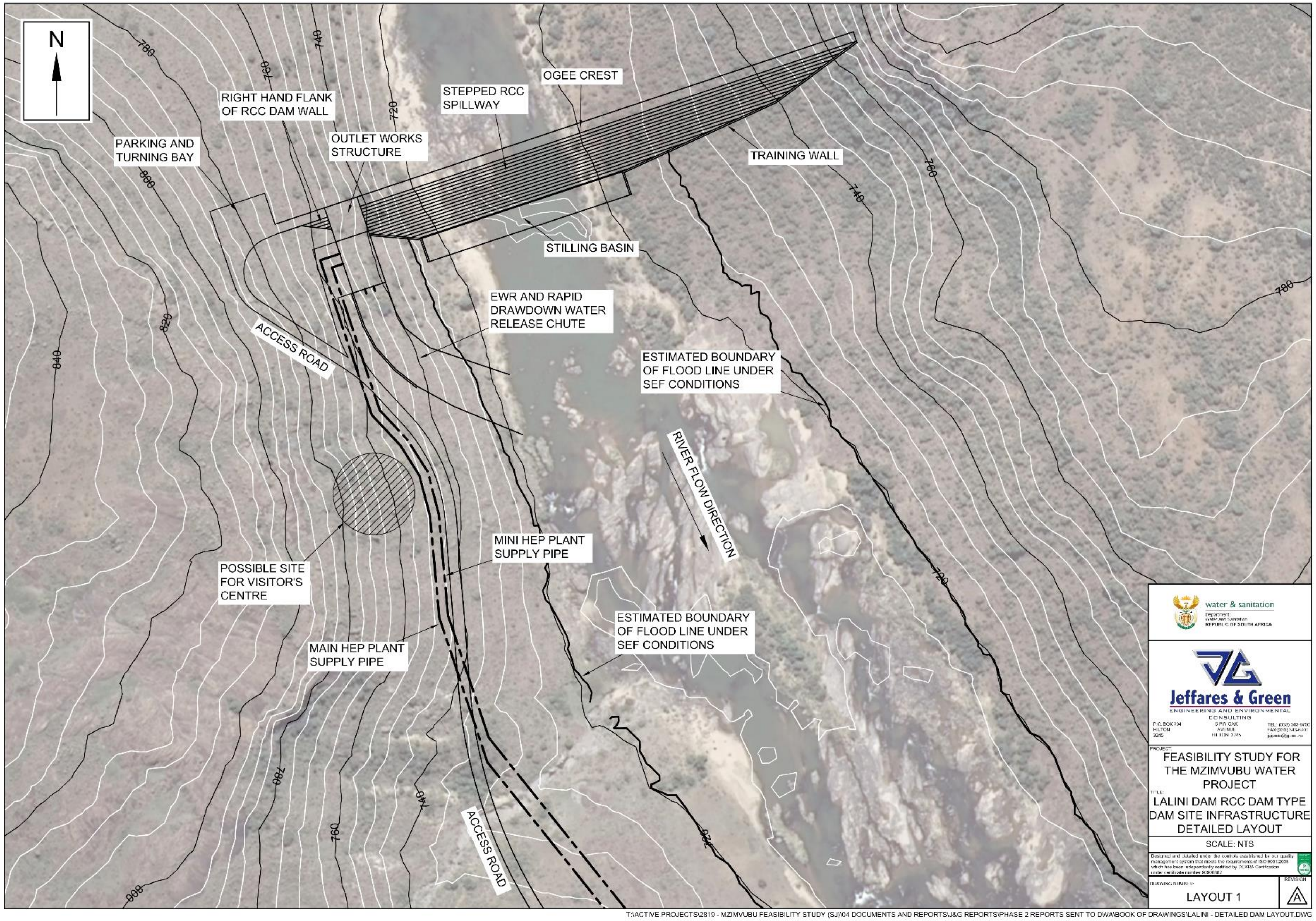


Figure 6-1: General Arrangement of the RCC Dam Option and Associated Infrastructure

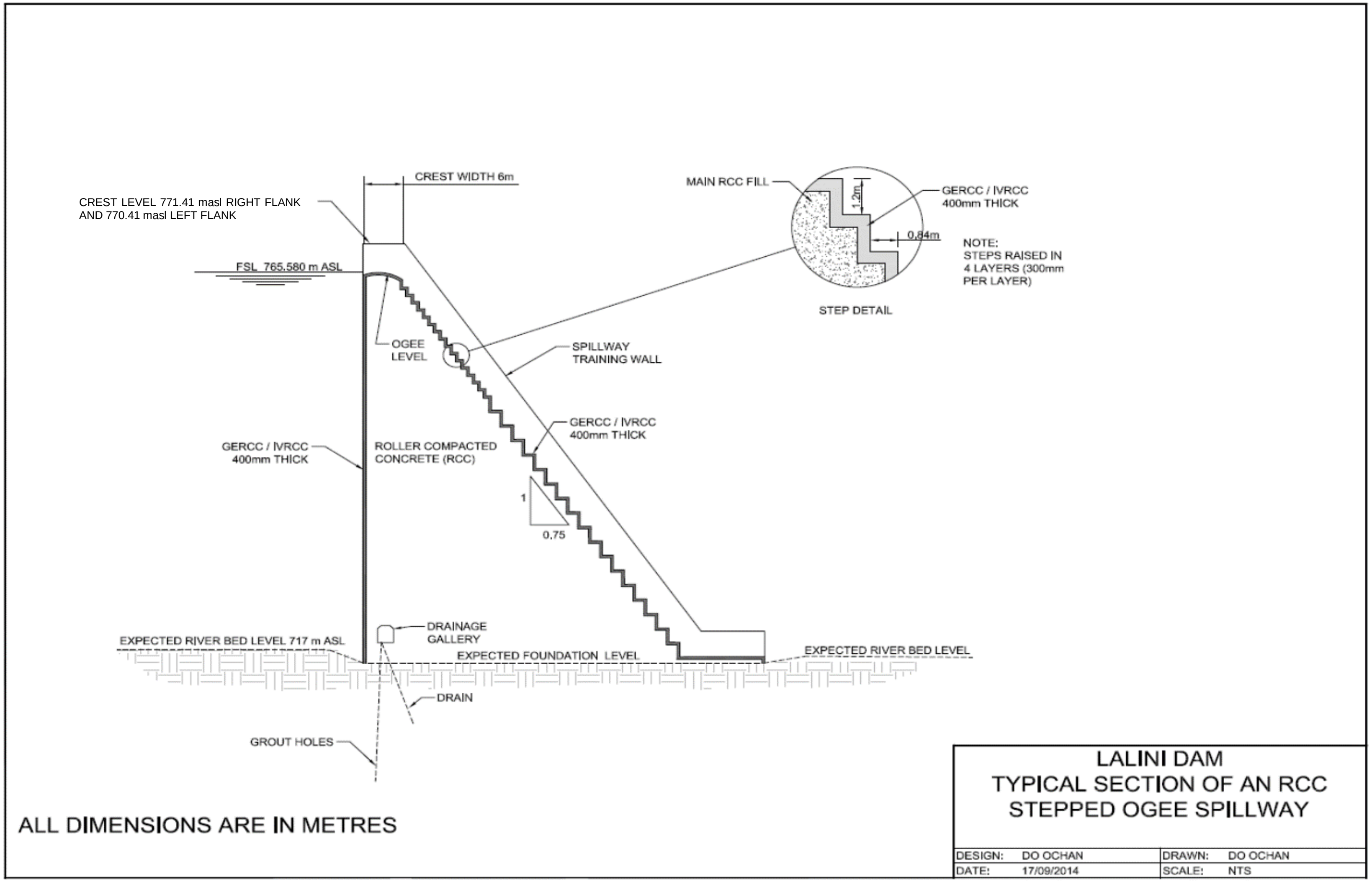


Figure 6-2: RCC Dam Wall and Spillway Typical Cross Section

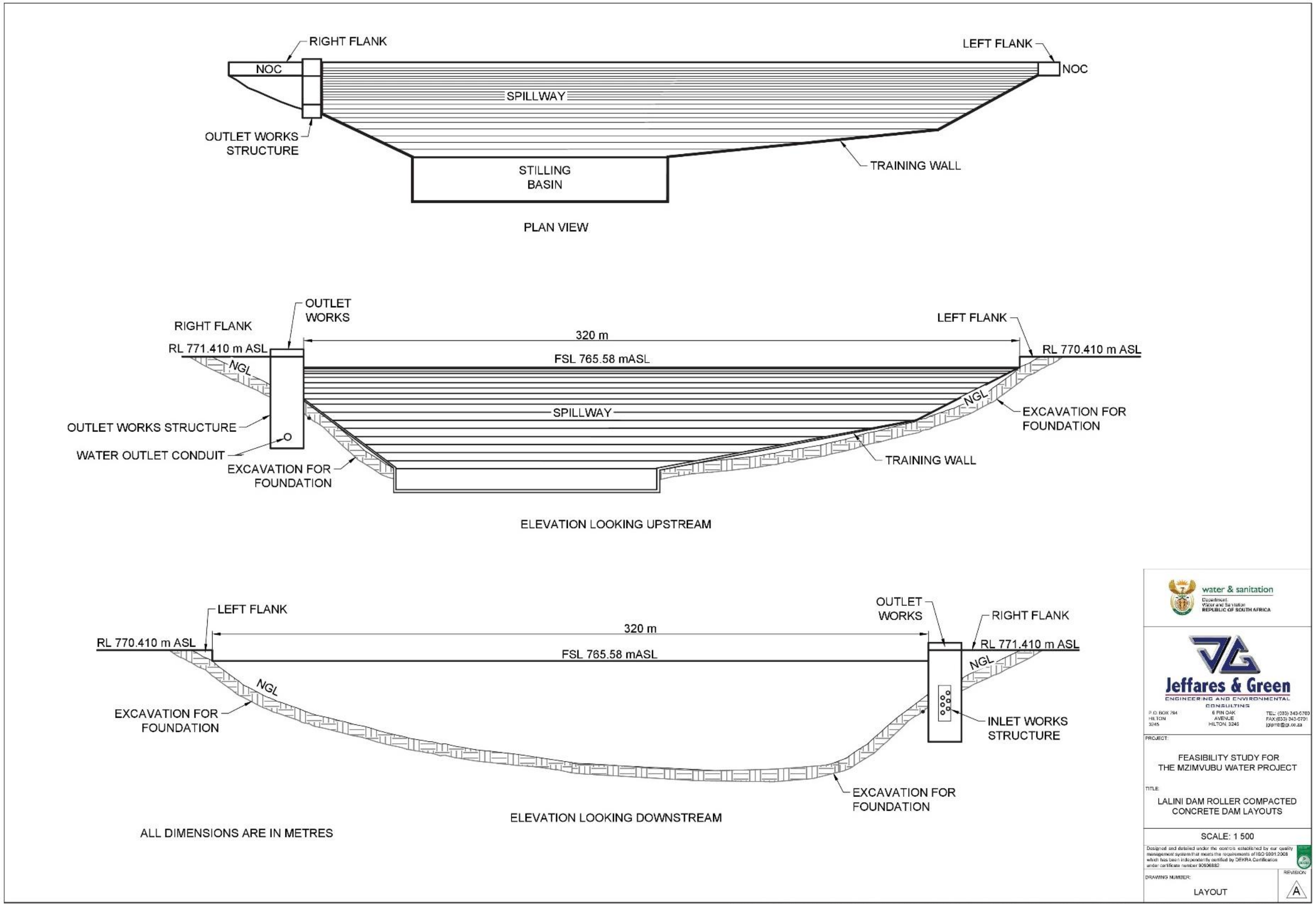


Figure 6-3: RCC Dam Wall Plan and Elevations

The geotechnical investigations have indicated that the founding conditions of both dam wall and stilling basin are in competent dolerite, which will exhibit low erodibility. The stilling basin can thus be of modest dimensions, and it is 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 is therefore suggested to reduce cost by not constructing the non-overspill crest to the full total free board level (as determined from Guidelines on Freeboard for dams, 2011) of the dam on the left flank. This would result in approximately a 0.67 metre wave over-splash during a design flood event. The NOC level is therefore set to 5.83 m above spillway crest level on the right flank, to prevent overtopping of the outlet works, and 4.83 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. Using a spillway crest length of 320 m, which, under the 3 500 m^3/s recommended design flood discharge, results in a flow depth over the crest of 3.0 m. This limits the unit discharge rate to an acceptable 10.9 m^3/s/m.

The flow depth over the 320 m spillway during the SEF event, which has a flow rate of 7 100 m^3/s, is 4.83 m with zero freeboard. The SEF event flood produces a unit discharge rate over the spillway crest of 22.2 m^3/s/m. This is at the upper end of that recommended for stepped spillways to reduce nappe separation and cavitation action.

Another issue would be the erosion impact on the left abutment of the dam under high spillage rates, and the design of a training wall and/or other methods of reducing this impact. Options might be to step the left flank downstream dolerite rock face to form an energy dissipating cascade, or to build a side chute in concrete to protect the rock from erosion. Another option might be to have a two stage ogee crest level which channels the design flood through the centre of the dam, and only when flows are above this value would the left hand section of the spillway come into play. This may require a slightly lower ogee crest level and higher freeboard due to the upstream maximum water level constraints imposed by the N2 bridge.

The spillway, chute and stilling basin arrangement must therefore be investigated in more detail and optimised during the detailed design stage, which could include both Computational Fluid Dynamics (CFD), and physical laboratory 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 nappe adheres to the ogee crest and does not separate. Physical modelling will therefore inform the design and, if necessary, changes in freeboard, ogee length and/or step profile might result.

6.2 Outlet Works

As described above, the dam wall and spillway would 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 right hand section of the dam wall, as is shown on the layout drawings.

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

6.2.1 EWR Releases

The Reserve Determination Report No. P WMA 12/T30/00/5212/7 determines the Environmental Water Requirements (EWR) to be released downstream of the Lalini Dam. This included a basic assessment of the expected EWR at the Tsitsa Falls site.

It was based upon running WRYM hydrological simulations and took into account the expected spills during the same period of simulation.

Additional Reserve Determination investigations were undertaken downstream of the Tsitsa Falls by the EIA PSP, and operational rules were developed for the Lalini Dam to comply with the updated EWR thus developed.

The recommended total releases at Lalini Dam are those required to maintain an intermediate ecological Class B/C of 287.1 million m^3 per annum (i.e. some 33% of MAR_{NAT}), which equates to an average of some 23.93 million m^3 per month.

The EWR is required to be released according to a seasonal pattern and this also depends on whether the river is in a state of flood or drought. EWR release rules are proposed in the reserve determination report, and release criteria are based upon preceding inflows.

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. The hydropower outlet pipeline requires are described below, but it was also recommended by the reserve determination team that freshets should be released periodically replicate natural flood occurrences, and that the capacity of the separate EWR outlet should be 60 m^3/s. As described below, this allowed the Emergency Drawdown Pipe and EWR release freshet outlet to be combined, which was sized at 3.0 m diameter.

6.2.2 Hydropower Generation

The investigation and analysis of hydropower generation at Lalini Dam is summarised in detail in the Hydropower Analysis: Lalini Dam Report No. P WMA 12/T30/00/5212/18.

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 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 outlet works pipework configuration allows for large and small release discharges directly into the stilling basin. The off-take pipework to the Lalini mini-hydropower plant is sized for the maximum hydropower output which equates to 16 m^3/s. In this case, a 3.0 m diameter pipe was deemed to be sufficient.

A second outlet conduit is required to supply the main HEP, and from the hydropower analysis it was determined that the maximum flow in this conduit would be 25 m^3/s. A 3.0 m diameter outlet pipeline was also recommended in this case.

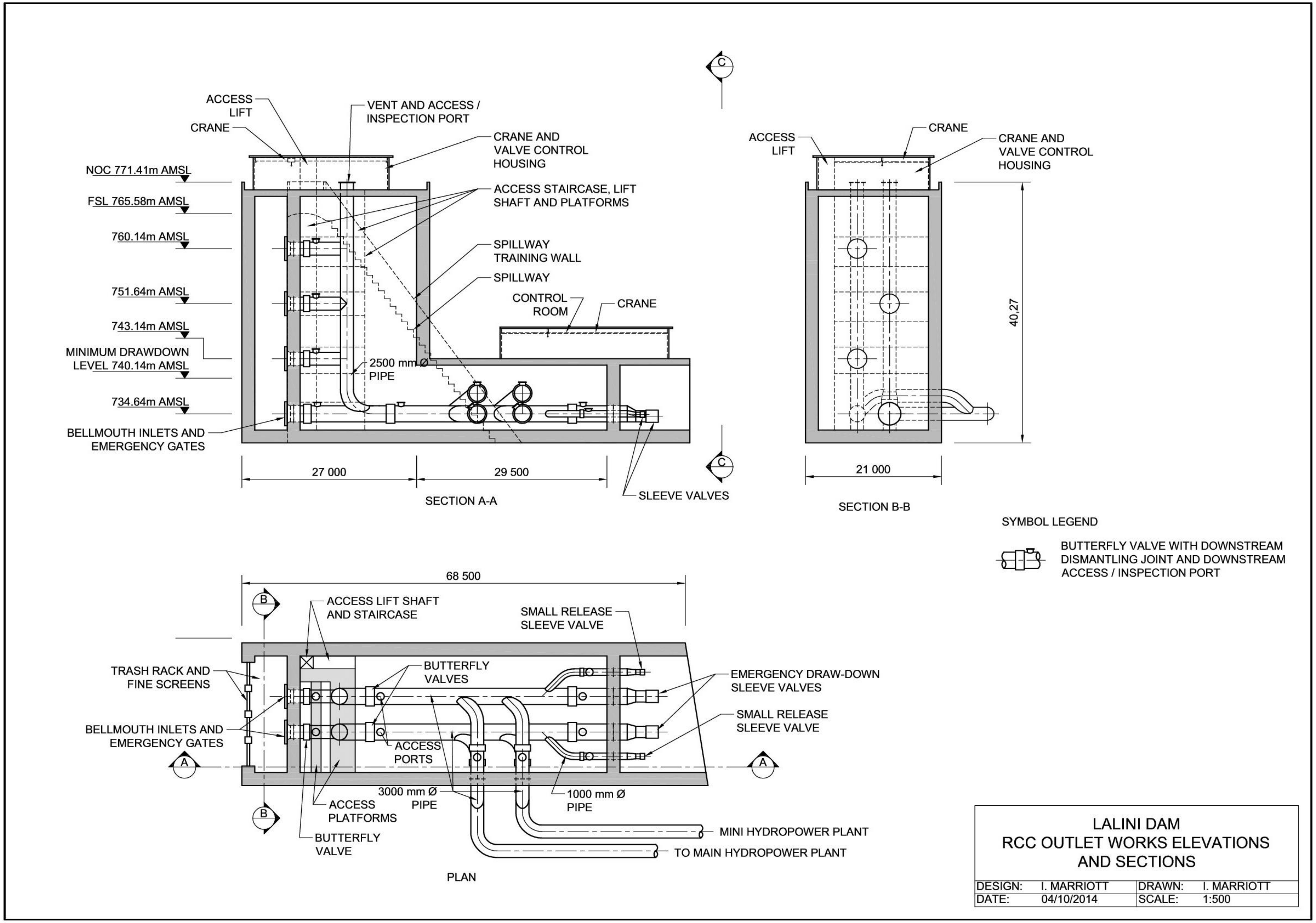


Figure 6-4: Outlet Works Elevations and Sections

6.2.3 *Emergency Drawdown Facilities*

It is a normal requirement to be able to rapidly drawdown (RDD) 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.

For the Lalini Dam, this means that some 214 million m^3 of water would need to be released in 90 days. This is an average flow of 27.5 m^3/s, with a peak flow of approximately 40.5 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 six outlet bellmouths 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 40.5 m^3/s and average of 27.5 m^3/s will be achieved. Given that a 3.0 m diameter outlet is recommended for the EWR case, the maximum velocity under RDD conditions would be 2.9 m/s which is acceptable.

In addition to the upstream emergency gates and butterfly valves on all of the 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 more suitable for flow control and tight closure.

It is 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 and vibration effects are minimised or avoided altogether.

6.2.4 *Summary of Outlet Works Parameters*

Table 6-1 summarised the outlet works and pipeline parameters required to meet the above functionality requirements.

Table 6-1: Summary of Outlet Works Parameters

		Flow scenario					
		Peak demand		EWR		RDD	
Description	Pipe dia.	Flow	Velocity	Flow	Velocity	Flow	Velocity
Intake stack	2.5 m	25.0 m^3/s	5.1 m/s	60.0 m^3/s	6.1 m/s	40.5 m^3/s	4.1 m/s
EWR and RDD pipe	3.0 m	N/A	N/A	60.0 m^3/s	4.2 m/s	40.5 m^3/s	2.9 m/s
Outlet pipe to mini HEP	3.0 m	16.0 m^3/s	2.3 m/s	N/A	N/A	N/A	N/A
Outlet pipe to main HEP	3.0 m	25.0 m^3/s	3.5 m/s	N/A	N/A	N/A	N/A

EWR: *Environmental Water Requirements* ***RDD***: *Rapid Draw Down*

7. LALINI HYDROPOWER SCHEME

7.1 Introduction

As discussed in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15, the objective of development of a hydropower component on the project at Lalini, is to create an income stream from energy sales to subsidize the energy, operations and maintenance costs of the main Ntabelanga potable and irrigation water supply scheme.

One means to achieve this would be for the hydropower component of the project to sell energy through ESKOM's Non-ESKOM Generator (NEG) programme, which allows for bilateral trading of energy using an existing ESKOM transmission or distribution grid via a "wheeling" arrangement, or a billing offset arrangement. Other options are available including the trading of Green Energy Certificates. The above report discusses this further, and the following section of this report focuses on the technical feasibility of development of such a hydropower component.

The basic principle being applied is to maximise the energy output of the Lalini scheme when operated conjunctively with the recommended 1.18 MAR_{PD} capacity Ntabelanga Dam. The Lalini Dam capacity limits, as determined by the energy output verses capacity characteristics, as well as the impacts of the rise in water level on the existing infrastructure and land use, social and resettlement issues, and other environmental impacts, were considered.

As described above, the practical range of Lalini Dam capacities investigated was between 0.10 and 0.60 MAR_{PD}. The Hydropower Analysis Report No. P WMA 12/T30/00/5212/18, show that there are steep gains to be made in energy production by increasing the storage capacity of the Lalini Dam from 0.10 to 0.30 MAR_{PD}, but that further gains were small beyond that capacity. Similarly, it was considered that there is no merit in developing a dam below 0.25 MAR_{PD} capacity as this produces a very small operating range in the dam water levels above that allowed for sedimentation. The smaller dam has a shorter spillway resulting in a higher flood rise, which is actually counterproductive.

Based upon the above constraints, the recommended dam storage capacity used for this hydropower scheme component of the feasibility study is 232.50 million m^3, equivalent to 0.2818 x MAR_{PD}. Hereafter this is referred to as the preferred dam capacity, or the "0.28 MAR_{PD}" capacity dam.

7.2 Lalini Hydropower Scheme Components

Typically, the main scheme components comprise:

- The Lalini Dam, with inflow supplied by natural runoff from the upstream catchment, as well as the spillage and the controlled release of water from the Ntabelanga Dam;
- Lalini dam outlet works for the conveyance of raw water to a mini-hydro-electric plant (HEP);
- Lalini dam outlet works to release water downstream to supply Environmental Water Requirements (EWR), and to rapidly draw down the reservoir in an emergency situation;
- A gravity flow raw water conveyance conduit and penstock from the Lalini Dam to the main HEP;
- An HEP plant, control room and switchgear, and output transformer station; and
- Inter-connecting power lines to evacuate the energy into the ESKOM grid.

The power lines will be constructed as advance works and configured so that they will also supply power from the national grid to the works during the construction period.

Other associated infrastructure to be developed would be:

- Temporary and permanent access roads and servitudes for the construction and operation of the scheme;
- New, replacement or realigned roads, power lines, services, buildings, and other infrastructure impacted by the dam and its high flood level;
- Water supply, power supply and telecommunications to the dam, tunnel, and HEP sites for the construction period and operational stage;
- Administration and operations buildings;
- Operations staff housing;
- Wastewater treatment works for the above; and
- Solid waste disposal facilities.

As with the Ntabelanga Dam, the release of water for the EWR provides an opportunity for additional generation of power at a "mini"-HEP which could be constructed just downstream of the dam. This is considered as an option herein to increase the energy produced by the conjunctive scheme.

It is recommended that this scheme should have a visitor's or information centre due to the greater interest presented by the hydropower plant. It is also situated in close proximity to the N2 and the Tsitsa Falls and gorge, which would be tourist attractions. This information centre should be developed in conjunction with the centre proposed at Ntabelanga Dam, so that they can complement each other and not be a duplication.

7.3 Lalini Dam

The main purpose of the Lalini Dam is to be the head race for the Lalini hydropower scheme, as well as a flow control structure to release EWR downstream of the dam. The storage capacity of the dam, as described in the previous sections, has been selected to provide adequate balancing storage and water pressure head to enable the hydropower scheme to operate on a continuous basis as far as possible.

At the minimum level of its proposed capacity range, the dam is able to absorb the estimated sediment deposition, which would be accumulated over a minimum period of 50 years, and still be able to supply water to both the hydropower scheme and for release downstream to meet the EWR. From this minimum operational draw-off level to the full supply level, an outlet works with multiple inlets would be constructed so that both hydropower and EWR flows can be drawn from a specific water level chosen to ensure that the best possible quality of water is released.

With regard to its operational range as a hydropower head race, the proposed Lalini Dam has the following characteristics:

- Capacity: 0.2818 x MAR (825 million m^3/a): 232.50 million m^3
- Full supply level: 765.58 m.a.s.l.
- 1 in 100 flood: 2 700 m^3/s
- 1 in 100 year flood level: 768.11 m.a.s.l. (FSL + 2.53 m)
- RDF (equivalent to 1 in 200): 3 500 m^3/s
- RDF water level: 768.58 m.a.s.l. (FSL + 3.00 m)
- SEF: 7 100 m^3/s
- SEF water level: 770.41 m.a.s.l. (FSL + 4.83 m)
- Minimum operating water level (MOL): 740.14 m.a.s.l.
- Invert level of main outlet conduit: 736.14 m.a.s.l. (MOL minus 4m)
- Environmental Water Requirements: 287.1 million m^3/a (33.05% of MAR_{NAT})

The above design flood water level footprint is indicated on Figure 7-1.

As can be seen, the dam wall is located approximately 3.5 km upstream (river centreline measurement) of the Tsitsa Falls at which point the river drops vertically some 85 m into a deeply incised gorge from where it continues to meander and fall in elevation by some 430 m over the next 60 km, (an average gradient of 0.7%), to its confluence with the main Mzimvubu River itself.

This vertical drop at the falls and steep gradient thereafter provided the opportunity to develop a hydropower scheme using the significant pressure head of water available over a relatively short distance from the Lalini Dam wall.

The primary target of the preferred scheme is that it could produce at least the same, but preferably significantly more energy per year than the expected energy consumption per year of the potable and irrigation water scheme supplied by the Ntabelanga Dam and associated works.

7.4 Hydropower Plant Locations and Conduit Options

Given the geometry of the river course and the undulating topography and rugged nature of the terrain, it was obvious that all hydropower water delivery conduit options would require a tunnel component in order to allow gravity flow from the dam outlet works to the location of the hydroelectric plant (HEP).

Initial desktop planning was undertaken before investigating several potential HEP locations and conduit routes during a field reconnaissance mission attended by the study team’s dam, tunnelling, geotechnical, hydrology, hydraulics and roads specialists.

Three potential HEP plant locations were investigated based upon the feasibility of delivery of water to them from the dam by gravity, and this resulted in options which had increasing conduit length but with increased available generating head.

The alignments of these three options were as shown on Figure 7-2.

The conduit lengths and elevations of the three possible HEP sites, and the average power generation static head, are listed in Table 7-1.

Table 7-1: Hydropower Plant and Conduit Options

Option No.	Conduit Length (m)	HEP Elevation (m.a.s.l.)	Average Static Head (m)
1	2 100	615.00	135.00
2	5 950	500.00	250.00
3	7 850	440.00	310.00

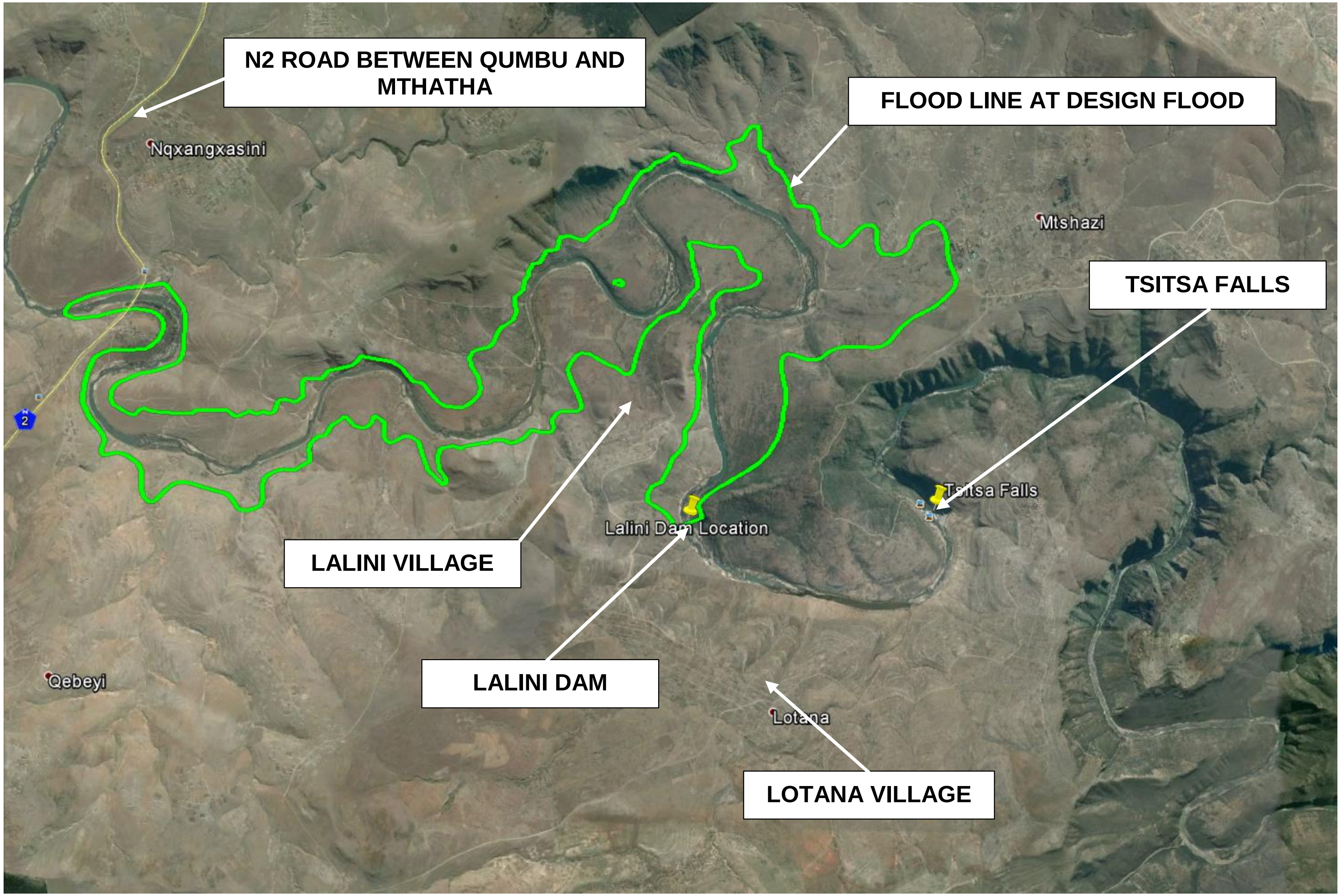


Figure 7-1: Lalini Dam, Water Level Footprint and Location

Several criteria were set for determining which option would be preferred:

a) ability to generate more than the annual power consumed by the main water supply scheme (87.3 million kWh/a), to facilitate energy wheeling/offset;
b) adequate vehicular access along piped conduit routes, to tunnel portals, and to the HEP;
c) avoidance of very expensive deep shafts and underground caverns for the HEP, which would significantly increase the construction period, the capital cost, and complicate operations and maintenance; and
d) maximising piped sections of the water conveyance conduits verses tunnelled sections, to reduce capital costs.

As can be seen in Figure 7-2 and Table 7-1, the conduit for this option is upstream of the dam wall and would involve the construction of a separate outlet tower. The driving of a 2.1 km long tunnel through the hill separating the dam basin and the gorge, immediately downstream of the Tsitsa Falls, will be required.

Upon exiting in the valley, the tunnel would need to transform to a near vertical penstock to an HEP built just above the high flood level in the river.

Figure 7-3 shows the terrain at the location of the penstock and HEP, as viewed from the Tsitsa Fall Lodge tourist viewpoint.

The site visit showed that whilst a drill and blast tunnel construction would not be a problem (once access to the starting portal across the river had been established from the Lalini village side), construction of access roads into the gorge to connect the tunnel to the HEP would be a significant problem. The problems that would be encountered to construct, maintain and operate the HEP, are seen to be a very difficult and expensive proposition, which would have a significant and permanent environmental and aesthetic impact on the falls viewpoint area itself.

The gradients of such access roads would be too steep to allow transport of the 70 ton power plant components to the HEP building.

Alternatives such as sinking a very deep shaft and construction of an underground powerhouse cavern in mid-tunnel would be expensive and difficult to operate and maintain, and were not considered to be acceptable.

Finally, the lower generating head that would be available for this option would not be sufficient to produce surplus energy revenue over and above the cost of energy to operate the water supply components of the scheme.

This option was therefore not considered further.

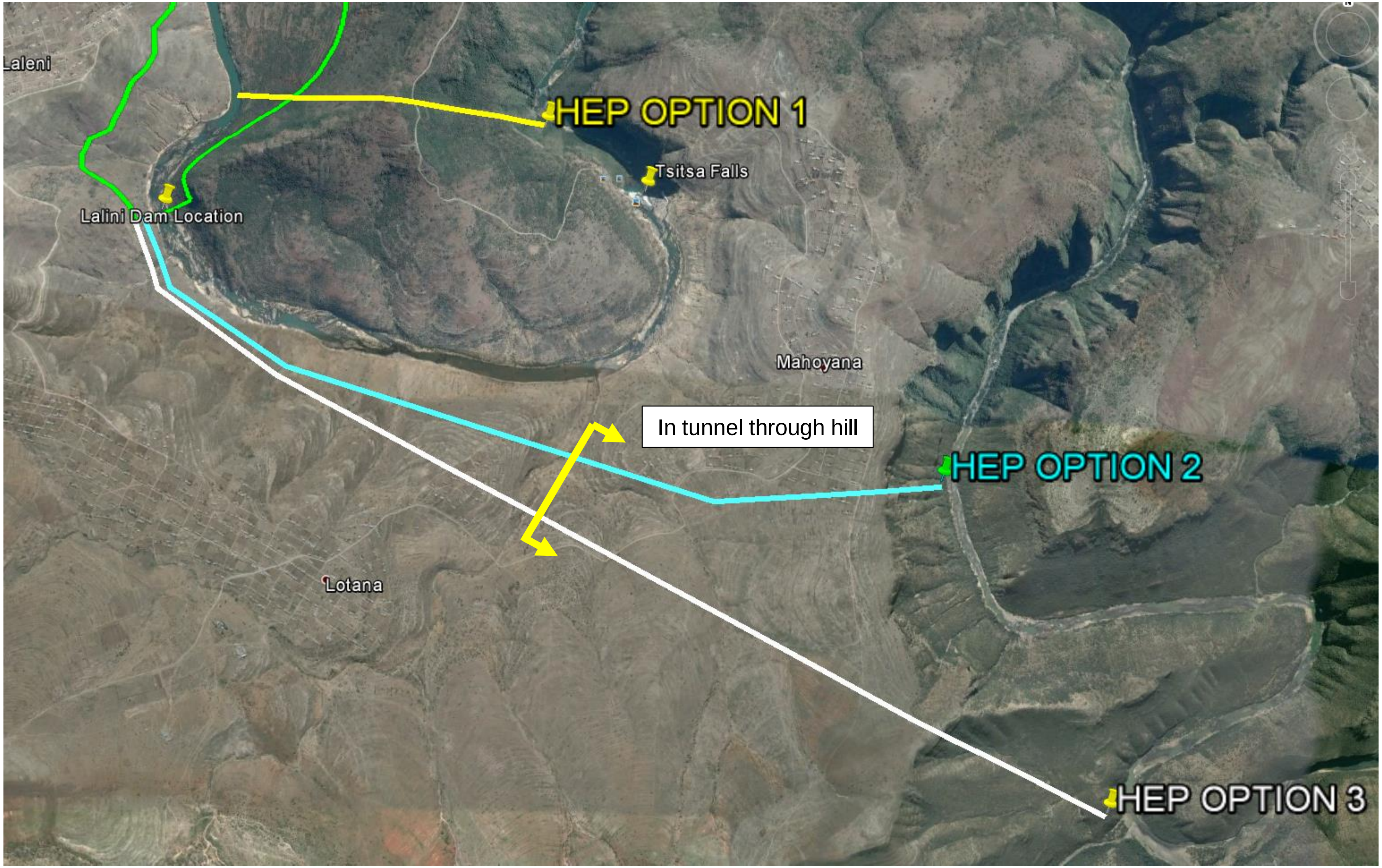


Figure 7-2: Hydropower Plant and Conduit Options

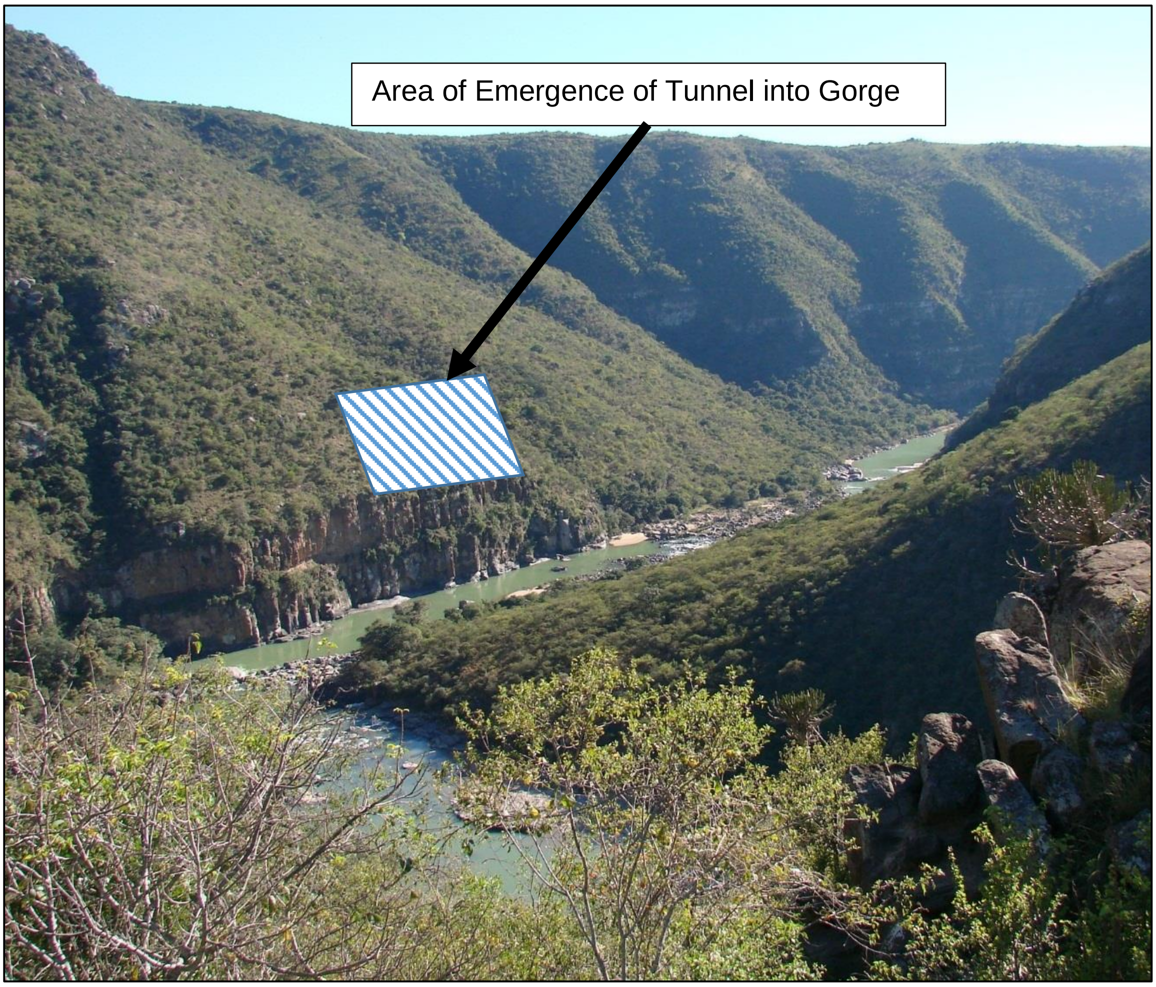


Figure 7-3: Option 1: Location of HEP in Gorge

7.4.1 Option 2

This option comprises an outlet works within the dam wall and a composite pipeline and tunnel conduit delivering water to an HEP plant located in the gorge further downstream of the falls.

The advantage of this location would be that significant additional generating head would be available over that offered by option 1, which would achieve the objective of generating more energy and revenue than that utilized by the water supply scheme.

The downside is the longer conduit length as well as the location of the HEP and access thereto.

Upon visiting this site it was immediately obvious that the construction of an access road down into the gorge to the tunnel exit point and the HEP would be virtually impossible.

Figure 7-4 shows the terrain at this tunnel emergence point.

Figure 7-4: Option 2: Gorge Terrain at Tunnel and HEP Location

7.4.2 *Option* 3

The presence of a tributary valley at the next westward meander provided a third possible option. The effect of this tributary was to widen the gorge and to make the valley slopes shallower, creating flatter terrain adjacent to the river at an elevation of some 450 m.a.s.l. It also provided suitable topography to route an access road at shallow enough gradients to allow heavy vehicles to deliver the turbine generators and associated equipment to the plant.

Figure 7-5 shows this location as viewed from the ridge through which the tunnel passes.

Having a conventional HEP building built above ground with easy access by road, is also considered to be a major advantage in terms of long-term operation and maintenance, when compared with having to locate and construct the HEP in an underground cavern.

This conduit routing, albeit longer than the other options at 7.85 km, allowed for 3.65 km to be constructed as a pipeline, either buried or on plinths. The tunnel section is therefore required for less than half the total length.

This dam and HEP combination can also produce more than double the energy requirements of the water supply scheme, offering the highest annual revenue surplus for the long-term subsidy of the water supply scheme.

This was therefore proposed as the preferred option to investigate in detail, and it was agreed to proceed on this basis. A preliminary longitudinal section of this option is shown in Figure 7-8.

Figure 7-5: Option 3 – General Location of HEP

Figure 7-6 shows the approximate location of the HEP at an elevation of some 450 m.a.s.l.

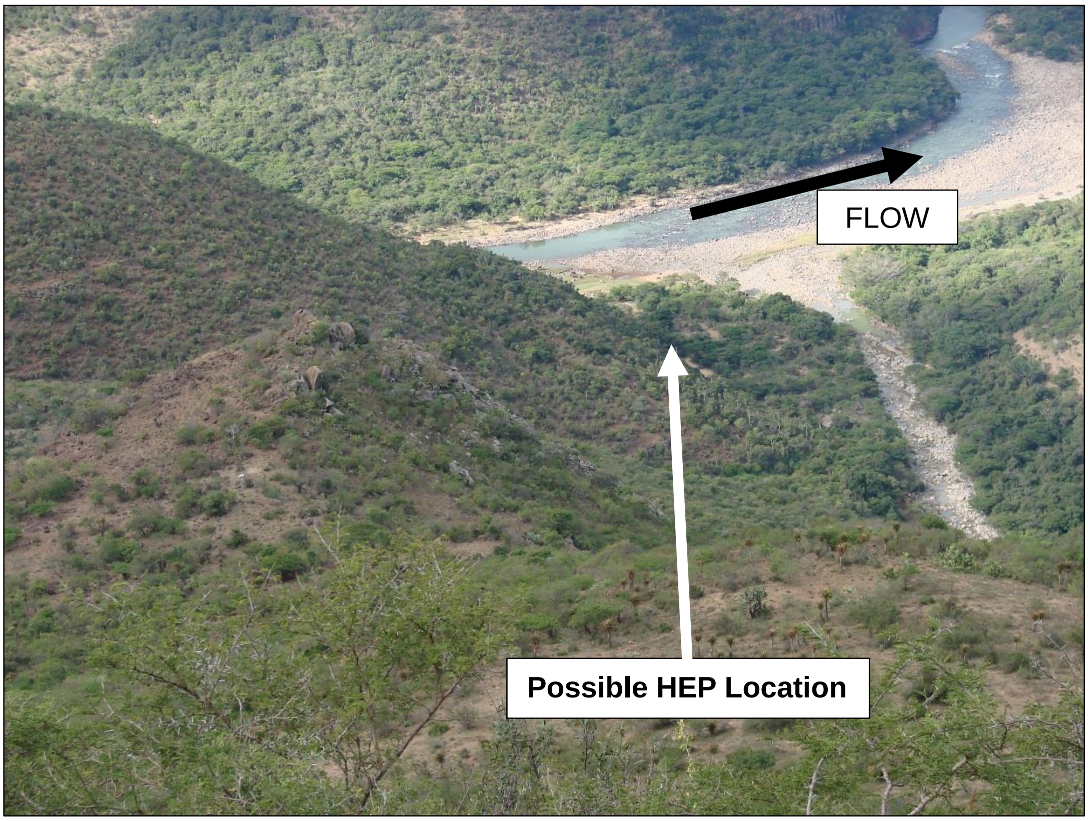


Figure 7-6: Option 3 – Possible HEP Location

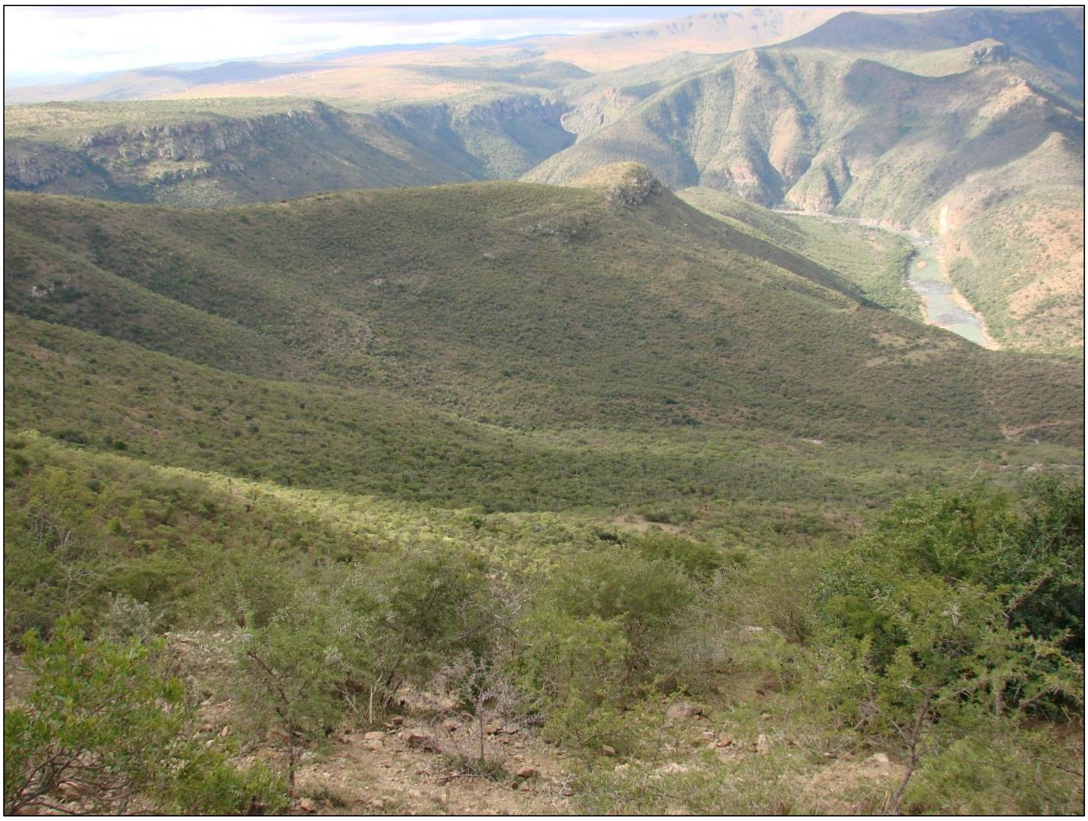


Figure 7-7: Option 3 – Terrain on Line of Penstock Route to HEP Location

7.5 Conduit Sizing and Material

7.5.1 Hydropower Plant Sizing

The Hydropower Analysis Report No. P WMA 12/T30/00/5212/18 describes the findings of the modelled hydropower outputs of the Ntabelanga and Lalini Dams when used conjunctively, and recommends an optimum HEP configuration. This analysis was undertaken for the base load case of continuous 24 hour per day operation.

The monthly hydropower generating regime is affected by the seasonal variations in river flow, the availability of water in each dam, the operational rules that determine minimum EWR releases at both dams, as well as maximum flow releases at Ntabelanga Dam in the dry season months.

Peaking options have also been considered to determine the cost benefits of operating the scheme to maximize income from energy sales by supplying higher power for fewer hours per day (using the same available daily water allowance) and targeting peak tariff periods.

The recommendations of the cost benefit analysis was to operate the scheme as a base load plant, but to be able to utilize the fully installed capacity for peaking during winter months when prevailing circumstances allow, and if environmentally acceptable.

The result of this is that, for the preferred 0.28 MAR_{PD} Lalini Dam, the HEP plant should have an installed generating capacity of 37.5 MW in the form of 3 x 12.5 MW Pelton wheel turbine generator sets.

7.5.2 *Impact of EWR and HEP Operating Rules*

The EWR release rules affect the maximum flow that can be passed through the hydropower component of the scheme.

The specific rule relates to the total recombined flows downstream of the HEP, which should not exceed the naturalized flow value for the same recurrence percentage value as the prevailing present day inflow. If strictly applied, this would significantly limit the flows through the plant in winter months and for between 30 and 70% of the time.

This would have the effect of reducing the average power generation given in Table 7-2 (below) by some 15%.

It was therefore proposed to the reserve determination study team and the responsible DWS EWR Directorate that additional releases from the Lalini Dam through the main hydropower plant of between 0.3 and 6 m^3/s (average 2.1 m^3/s) be permitted in very low flow months. That is when the naturalized flow in the river would otherwise be insufficient to meet the minimum EWR as well as operating one turbine of the HEP (12.5 MW).

A meeting was held at DWS to present the findings of the Lalini EWR Study and the implications of the EWR rules to these same parties, and the case for allowing such hydropower releases was also presented. The general consensus was that such releases could be tolerated and might even be of benefit to the ecological system in the very dry years.

It was explained that an alternative to the above would be to shut down the HEP in these particularly dry months, and to increase the installed capacity of the HEP in order to increase the average power output in the wetter months.

In such a case a larger installed HEP turbine capacity and larger transfer conduit system would be required to generate this additional average power, but at significant additional capital cost.

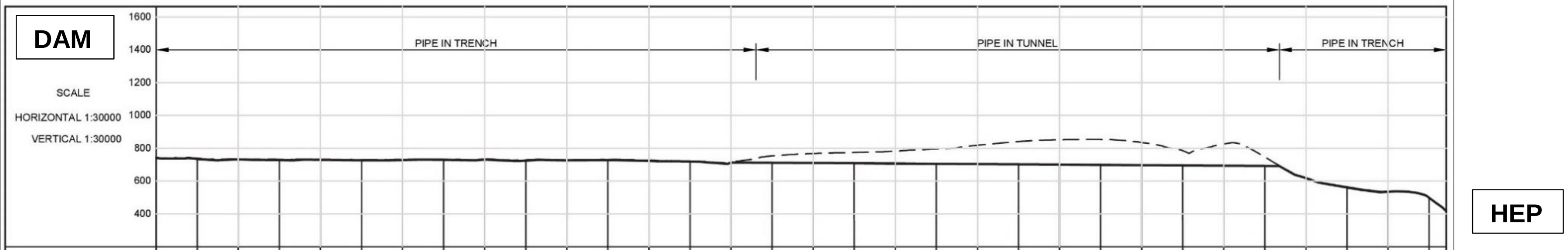
This is discussed and analysed in detail in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15.

Given the positive response from the DWS Directorate, the 37.5 MW installed capacity scheme defined in Table 7-2 is recommended as the preferred scheme, but the implications of a 50 MW installed capacity scheme were also investigated.

7.5.3 *Conduit Flow Capacity Requirements*

Based upon the above water availability, and for the base load scenario, the HEP would be operated on a continuous 24 hours per day basis to achieve a target energy output for each particular month using one, two or all three turbines. Based upon a run using the historical flow series, in most months and years these targets will be met, but in other months the operating rules will require the turbines to be operated for fewer hours per day, or less days per month to match the actual water allocation for that particular month.

These rules are normally based upon status of the dams at the time as well as the actual preceding inflows, taking cognisance of historical flow durations frequency analyses and the EWR rules for that particular month.



CHAINAGE	0	250	500	750	1000	1250	1500	1750	2000	2250	2500	2750	3000	3250	3500	3750	4000	4250	4500	4750	5000	5250	5500	5750	6000	6250	6500	6750	7000	7250	7500	7750	7855
GROUND LEVEL	739.93	733.96	730.65	727.38	727.35	726.32	727.52	727.89	729.44	724.78	725.3	726.71	721.58	717.73	709.58	748.90	763.82	770.40	778.94	791.25	814.15	836.26	847.56	849.32	830.65	780.66	822.51	742.12	618.03	559.96	532.71	496.83	410.96

Figure 7-8: Option 3 Hydropower Conduit Longitudinal Section

As can be seen, the base load scheme would be operated using one, two or three turbines simultaneously. For the worst case scenario, where the Lalini dam water level is at minimum operating level, a maximum of 19.5 m^3/s flow rate would be required to pass through the system to generate the maximum installed 37.5 MW of power.

Based upon the average target hydropower output and the average water level in the dam, the average flow rate in the conduit would be 12.5 m^3/s.

Operating pressures within the conduit range from 5 metres head to 80 metres head in the first section to the end of the tunnel, and from 5 metres head to 340 metres head in the penstock section from the tunnel to the HEP.

If a larger capacity of 50 MW were to be installed, this would require a larger diameter transfer conduit/tunnel as the maximum flow rate would increase to 25 m^3/s.

A further option was investigated whereby the scheme was configured to operate only as a peaking station i.e. using the same daily water availability as was determined by the above modelling, but releasing this same available volume through larger turbines for only a few peak hours per day to take advantage of the higher energy tariffs that would then be paid.

In such a case the maximum installed capacity of hydropower plant would increase to 150 MW (3 x 50 MW units), and the maximum flow rate through the conduit would increase to 75 m^3/s.

Based upon the above range of hydropower options, the acceptable flow velocity, and head loss limitations, the range of conduit diameters investigated was from 2 000 mm to 4 500 mm (the larger diameter being for the peaking option)

Table 7-2: Lalini Main Hydropower Scheme Average Monthly Energy Production

Month	Minimum Target (MW)	Avg HP Output (MW)	Avg Energy Supplied (KWh)
Oct	12.50	18.76	13 959 044
Nov	12.50	23.67	17 043 420
Dec	25.00	22.99	17 102 324
Jan	25.00	21.89	16 283 250
Feb	25.00	23.54	15 963 055
Mar	37.50	24.55	18 268 136
Apr	25.00	22.27	16 035 946
May	12.50	15.69	11 672 893
Jun	12.50	15.83	11 399 591
Jul	12.50	15.95	11 866 003
Aug	12.50	16.04	11 931 220
Sep	12.50	16.46	11 849 343
Total Energy Per Year (kWh)			**173 374 226**
Average Power (MW)		**19.77**	

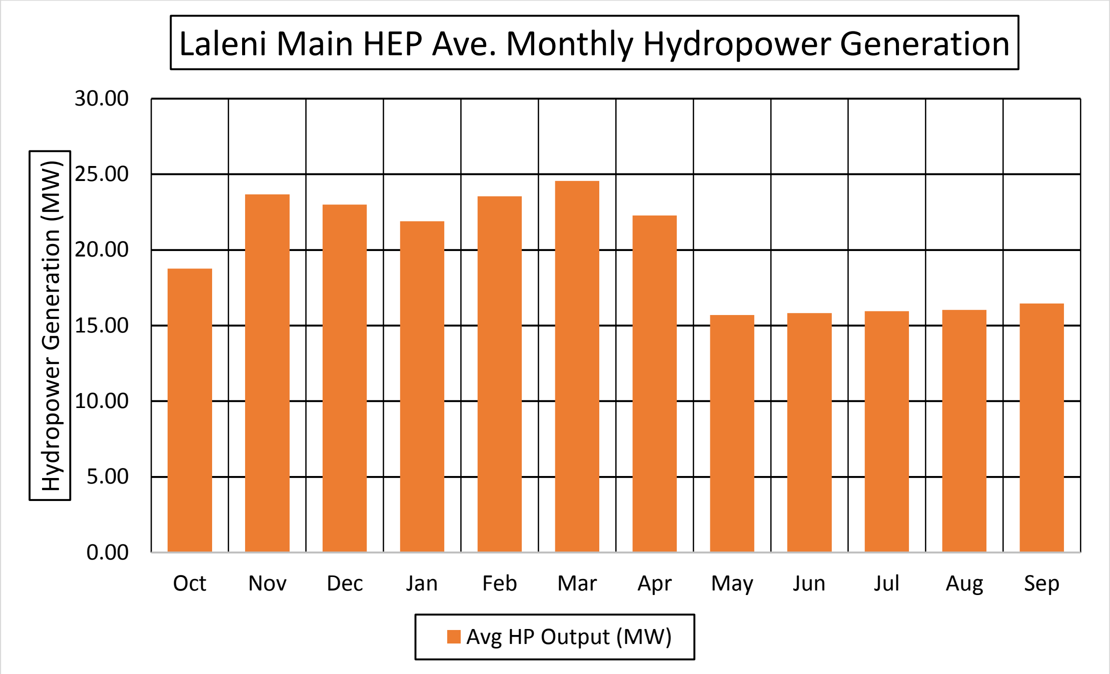


Figure 7-9: Lalini Main HEP Average Monthly Hydropower Generation

7.5.4 Conduit Material

In selecting the preferred conduit material, the availability of pipe diameters and suitability for the local conditions were taken into consideration. Glass reinforced plastic (GRP) pipes are not recommended as they have not had a good track record in the region at these large diameters, and are unsuitable for high pressure penstocks constructed above ground. Plastic pipes (uPVC) are not manufactured at these diameters and higher pressure classes.

HDPE pipes have elsewhere in the world been specially manufactured at diameters of up to 4 metres, but these again would have to be fabricated on site and would only be suitable for the lower pressure ranges. The wall thickness and rib strengthening requirements would be likely to make such a process uneconomical, and no readily available replacement pipe or fittings would be available if required in the future due to the non-standard manufacturing process.

Ductile iron pipes are manufactured up to 2 000 mm diameter at which size they would be limited to 32 bar operating pressure.

Steel pipe can be manufactured at two factories in South Africa for diameters up to 2 500 mm, and at one of those factories, at diameters up to 3 500 mm. Above that larger diameter, plate would have to be imported, and a special manufacturing facility established on site.

Given some of the advantages, including the ease of laying, of ductile iron over steel pipe, one option would be to lay twin ductile iron pipes of an equivalent hydraulic capacity instead of one larger steel pipe. In the case of a 2 500 mm diameter conduit, this would require twin 1 800 mm diameter pipes.

Whether this twinning option is an economic alternative to a single steel pipeline would only be confirmed once a full tender process is undertaken, as the final costs would be influenced by the contractor’s approach and methodology as well as the prevailing iron and steel prices and exchange rates. One advantage of a twin pipe system would be that one pipe could be serviced and cleaned whilst the other continued to operate. However, twin pipes would also require a large tunnel diameter for the same hydraulic capacity.

For this feasibility study analysis, it was decided that analysis would be undertaken based upon a single welded steel pipeline.

Therefore, taking all of the above into consideration for the purpose of this feasibility analysis, the options investigated were based upon using welded steel pipe as the conduit material, and the laying of the conduit below ground up to the tunnel portal, within and through the pre-drilled tunnel section, and then either above ground or in trench from the tunnel exit portal to the HEP plant.

Other options considered included the steepening and extending the tunnel length to exit directly at the HEP location, thus avoiding the steep steel penstock section. One further option was to design part of the tunnel section as a pressure tunnel which would not require the steel pipe in that section. These options are discussed further in the tunnel section below.

7.5.5 Optimum Conduit Diameter for Base Load Case

A discounted cash flow analysis was undertaken which considered the relevant capital, operating and maintenance costs of the conduit and associated works for the base load case operation indicated in Table 7-3, and for a range of steel conduit diameters from 2 000 mm to 3 500 mm. The detail of this is given in the Cost Estimates and Economic Analysis Report.

The costs were discounted back to present values for a typical range of discount rates. As each diameter produces a different annual energy production due to the varying head losses, the expected revenue from energy sales per annum was credited back into the calculation and also discounted back to a present value. The net present value of costs and income was then divided by the present value of the kWh of energy produced to give a unit reference value (URV) of energy produced by each pipeline diameter in Rand/kWh.

Table 7-3 shows a summary of these results, and the 2 500 mm diameter option has the lowest URV for all discount rates.

Another factor is the maximum flow velocity in the pipeline, which for the 2 500 mm diameter pipeline was 3.97 m/s. DWS will allow up to 5 m/s continuously on outlet systems but given that this is raw water which could have an abrasive element, and that smaller pipes have a lower total energy output and higher URV, it was decided to select the 2 500 mm diameter pipe as the preferred conduit size.

Table 7-3: Discounted Cash Flow Analysis to Size Conduit for Base Load

Lalini Dam to HEP Conduit					
NOM. PIPE DIA (mm):>		**2000**	**2500**	**3000**	**3500**
MAX VELOCITY (m/s):		**6.21**	**3.97**	**2.76**	**2.03**
NPV COSTS (Net cost minus Power Sales)	4%	400 259 867	397 413 533	534 867 580	715 607 209
	6%	659 379 777	691 942 489	837 085 458	1 017 978 486
	8%	838 171 137	895 288 740	1 045 430 771	1 225 993 887
	10%	964 493 569	1 039 058 660	1 192 448 411	1 372 378 383
NPV POWER (kWh)	4%	2 890 962 807	3 212 706 923	3 309 189 011	3 344 840 932
	6%	2 289 478 028	2 544 281 058	2 620 689 381	2 648 923 674
	8%	1 865 529 177	2 073 149 639	2 135 409 226	2 158 415 299
	10%	1 558 034 199	1 731 432 602	1 783 429 948	1 802 643 932
Net Cost URV (R/kWh)	4%	0.138	**0.124**	0.162	0.214
	6%	0.288	**0.272**	0.319	0.384
	8%	0.449	**0.432**	0.490	0.568
	10%	0.619	**0.600**	0.669	0.761

This selected pipe size is also coincident with the largest diameter of steel pipe currently produced in the two pipe factories in South Africa.

7.5.6 Conduit Diameter for Peaking Case

As described above, the flow rate for the peaking case is up to 75 m^3/s. Given that this case involves intermittent operation of the scheme for varying hours per day, the same discounting techniques are not necessarily as appropriate for pipe sizing purposes. In this case the maximum head loss and flow velocity are the key factors.

Following the hydraulic analysis to determine head losses and flow velocity, it was recommended that a 4 500 mm diameter pipe size be used, which limits total head losses through the system to 35 m and the maximum flow velocity to 4.7 m/s.

It should be noted that such a pipeline would require a special fabrication plant to be established at the site, and would probably require special importation of steel plate of the thickness required.

7.6 Tunnel Options

The initial approach was to minimise the length of the tunnel section of the overall hydropower conduit in order to reduce costs.

Following the first field reconnaissance mission, a decision needed to be made as to which tunnel profile and alignment the geotechnical investigations should focus.

Generally, shallow tunnel alignments have the advantage if a surge shaft is required. This can however also have the disadvantage of shallow overburden pressure being insufficient to contain water pressures in the tunnel if used as a pressure conduit.

The minimum gradient for tunnel construction is normally set to 0.2% to allow for drainage.

Maximum gradients vary and depend on the construction method. For tunnel boring machine (TBM) construction, the maximum grade is normally about 1%, as defined by train haulage limitations. However, TBM construction is not considered to be a likely solution for this relatively short length of tunnel, unless the successful contractor happened to have a suitable TBM readily available at the time of tendering.

In this case, the most likely solution would be a drill and blast construction method, for which gradients of up to 10% can be considered.

The minimising of the tunnel section length was achieved through adopting a relative shallow grade in the tunnel section of 0.3%, with the tunnel commencing where the pipeline section encounters rising ground level, at an elevation of approximately 715 m.a.s.l. This is the alignment shown in Figure 7-10.

This gradient results in the tunnel exiting the hill at an elevation of 705 m.a.s.l. and a tunnel length of 3 300 m. From this outlet portal, a steel penstock would be constructed down the hillside to the HEP plant, which is located at 445 m.a.s.l.

Once this alignment had been selected, the limited study timescale dictated that the geotechnical investigations should immediately proceed, and part of the available budget for such investigations was allocated for drilling some cores along the tunnel route.

It was recognized that the limited budget allocation would not be enough to undertake fully comprehensive and deep drilling investigations of the tunnel alignment, and the results are therefore only a general indication of the sub-surface geology and rock type. Significant additional investigations would therefore be required in the implementation stage to properly inform the detailed design.

Figure 7-11 shows a plot of the boreholes undertaken during this investigation.

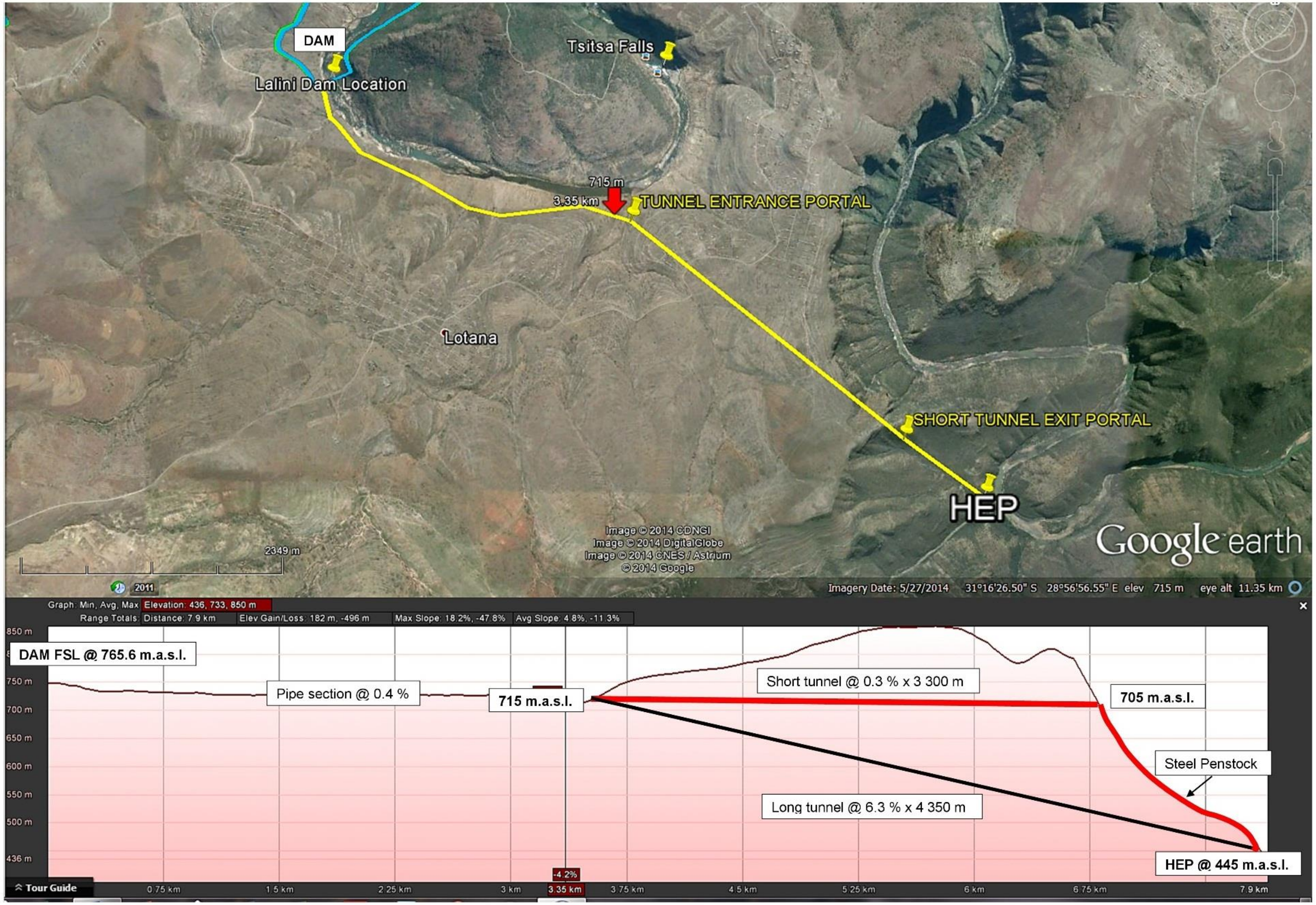


Figure 7-10: Tunnel Horizontal and Vertical Alignment Options

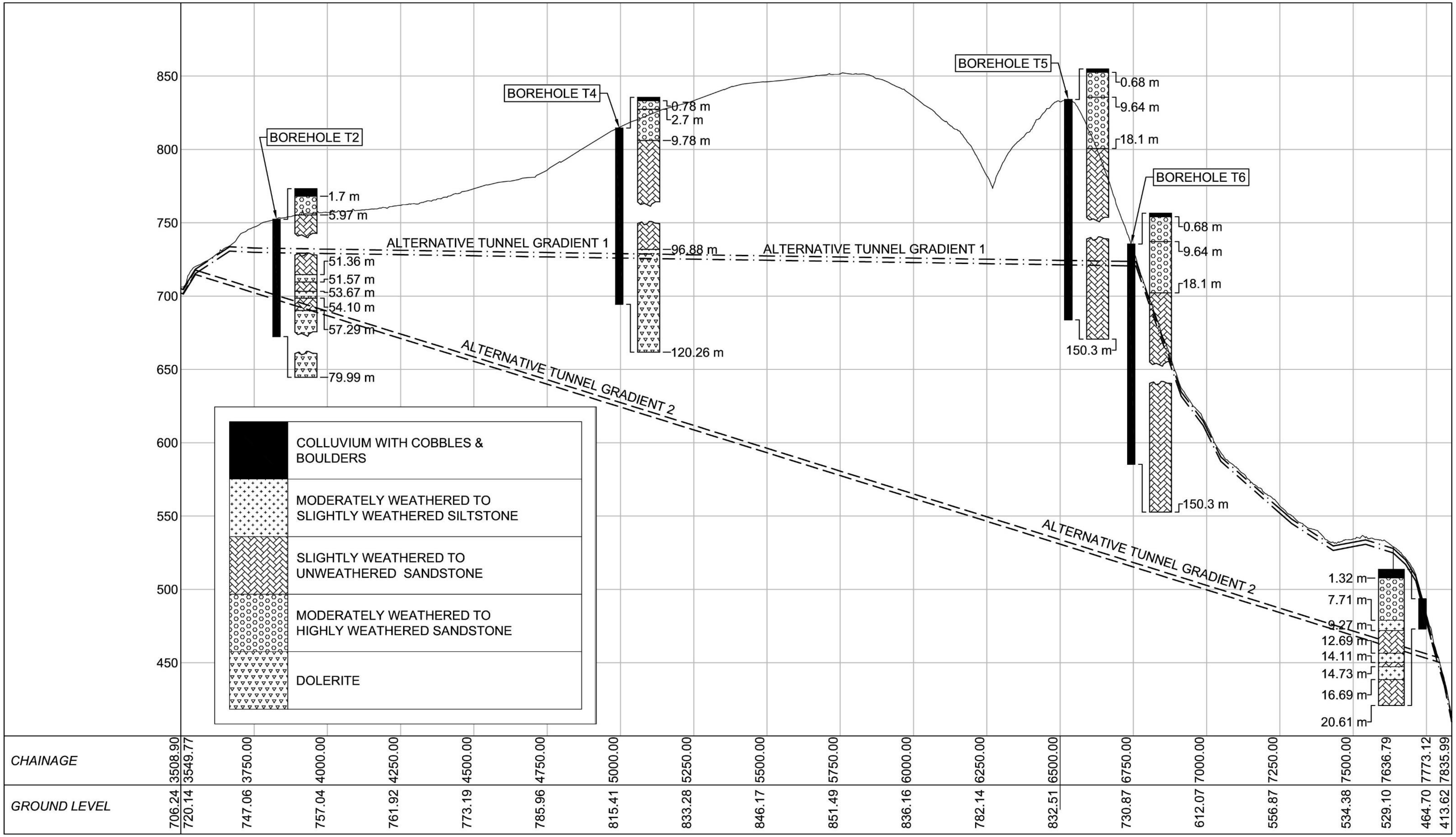


Figure 7-11: Boreholes Drilled along Tunnel Alignment

As shown, the boreholes encountered moderately to highly weathered sandstone at the surface, but soon moved into very competent sandstone, which is the predominant rock throughout. Boreholes T2 and T4 encountered competent dolerite which is a feature of the area. Contact interfaces between the sandstone and dolerite were relatively unaltered and tight. The findings of the geotechnical investigations are given in the Geotechnical Investigations Report No. P WMA 12/T30/00/5212/10.

The overall findings concluded that the main body of the hill along the tunnel alignment would be highly suitable for tunnelling and had a low or non-existent water table. An average of the various tests and classifications showed an RMR of 72, which is Class ii good rock.

7.6.1 Short Tunnel Advantages and Disadvantages

The advantages of a shorter, shallow grade tunnel shown in Figure 7-12 would be that construction could be undertaken from both portals simultaneously, and that water supply and dewatering would be straight forward.

However, an access road and working platforms would need to be constructed to both portals, which is fairly problematical as regards the outlet portal location, with its accessibility challenges. Such an access road would also leave a permanent scar on the steep hillside with a potential for future erosion problems.

A further site reconnaissance visit to specifically investigate tunnelling options and access roads highlighted the difficult prevailing conditions for construction in this proposed penstock location. The steep gradients of up to 35% make conduit construction particularly onerous and this is made worse by the nature of the ground surface which is highly weathered sandstone and “mobile” tallus.

The unsuitability of this slope as regards stability, founding and bedding, and the difficulties in access and handling of large diameter pipes of 13 tonnes each on such gradients, would make this penstock construction very expensive. Whether the penstock was built on plinths or buried underground, this servitude area would continue to be difficult to access and maintain. It could also be a vulnerable section as far as the potential for future erosion and damage to the penstock is concerned. This area has also been identified as a sensitive ecological area of high significance where infrastructure development should be limited as far as possible.

7.6.2 Long Tunnel Advantages and Disadvantages

A second vertical alignment was also considered as shown in Figure 7-13.

This option used the same upstream portal location and elevation, but was graded at 6.3% so that it exited lower down in the valley and close to the HEP location at an elevation of 445 m.a.s.l.

This has its own advantages and disadvantages. The advantage is that it avoids the difficulties described above regarding the penstock construction, and leaves no exposed surface works along its entire route. This also avoids having to construct an access road to the outlet portal on a steep hillside location. Instead the outlet portal construction access road and platform can be shared with that required to construct the HEP plant itself.

The disadvantages would be that the tunnel section of the overall conduit would need to be longer (albeit avoiding the costly and difficult construction of the penstock section), and would have to be constructed only from the lower portal upwards to effect gravity drainage, and removal of excavated materials. Drill and blasting downhill to an elevation of greater than 150 m below the portal has many difficulties, including dewatering challenges.

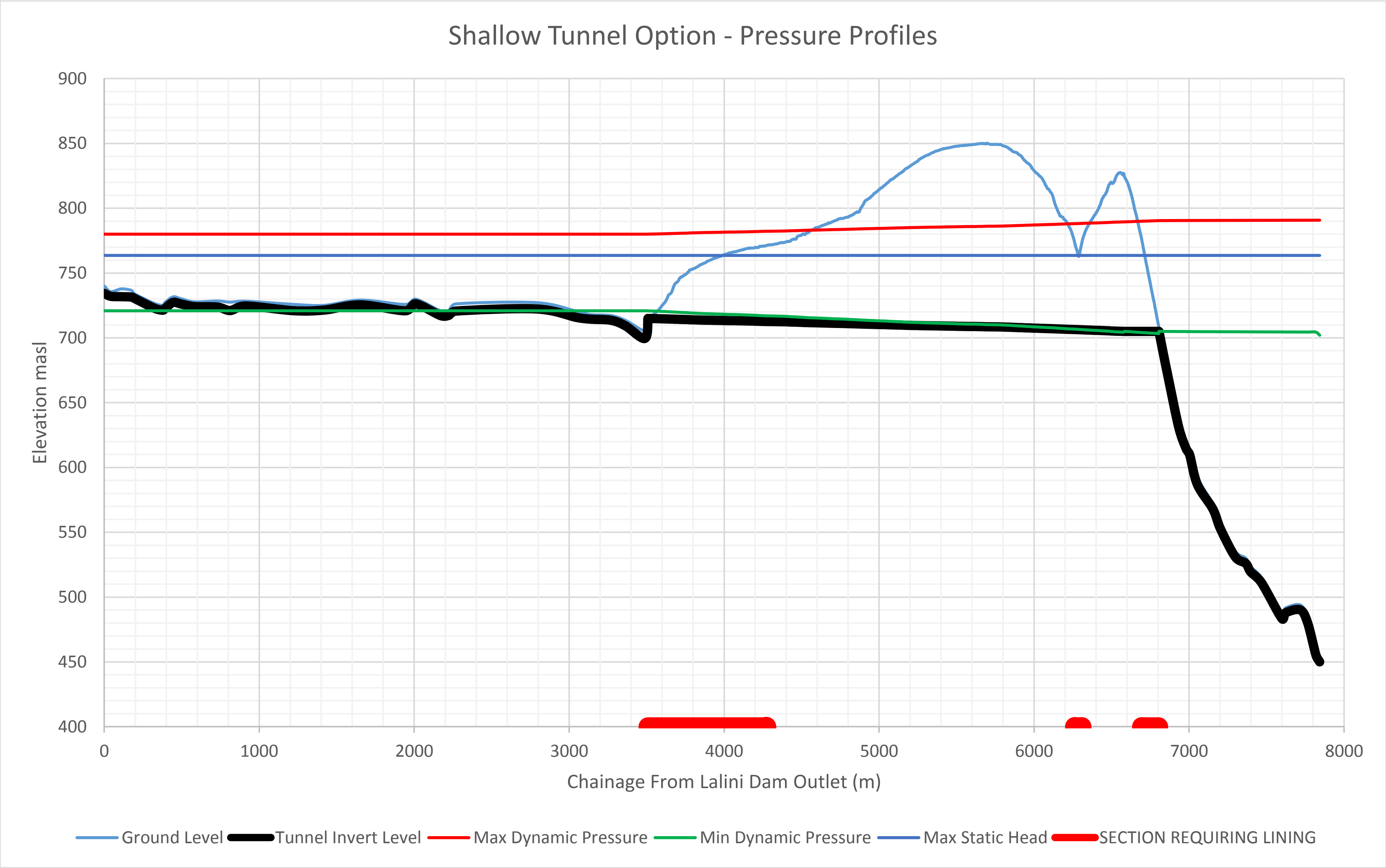


Figure 7-12: Shallow Tunnel Option: Pressure Profiles and Lining Requirements

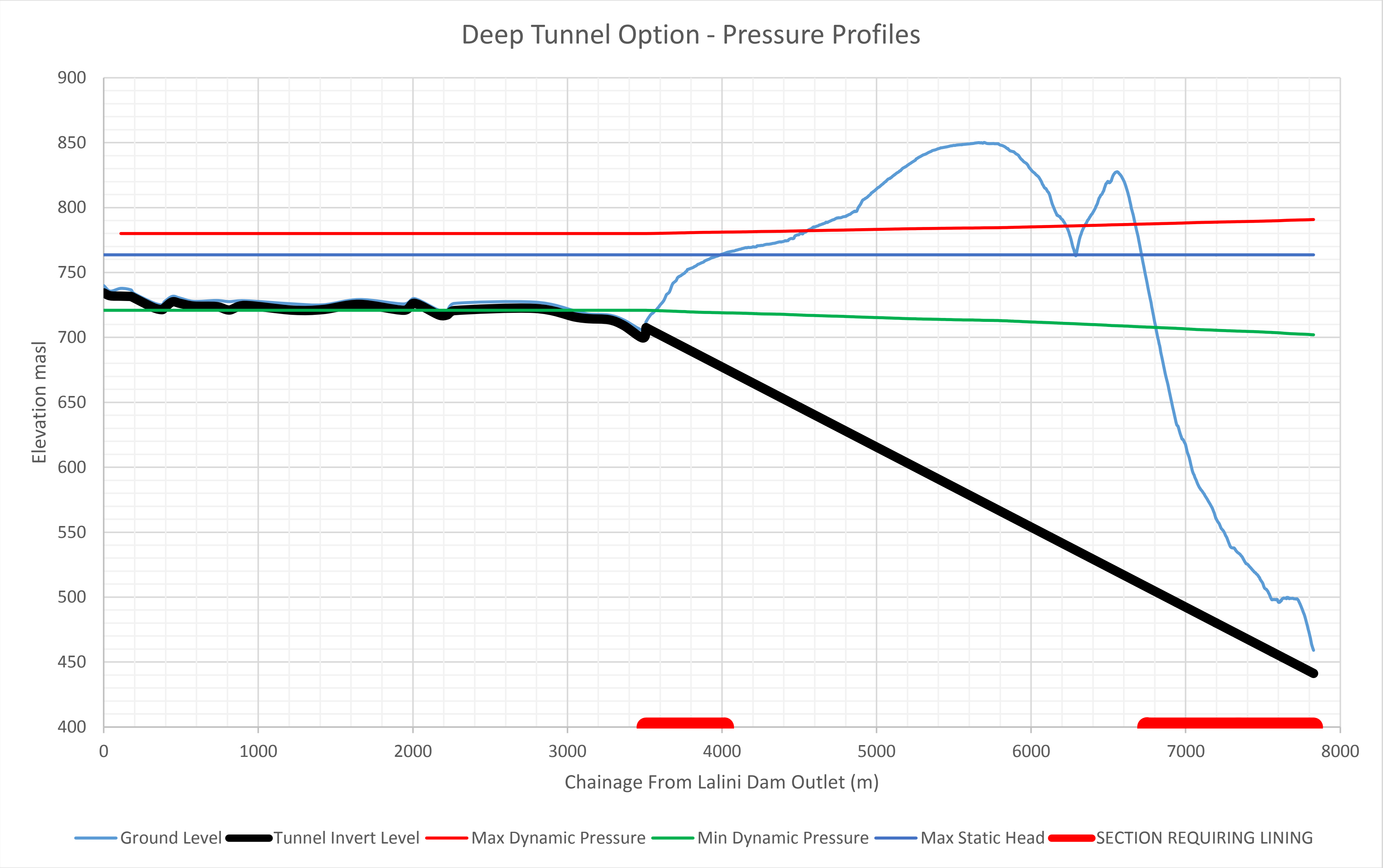


Figure 7-13: Deep Tunnel Option: Pressure Profiles and Lining Requirements

7.6.3 *Pressurized Tunnel Option*

Another option was to design the tunnel section as a pressurized conveyance component rather than an adit through which a steel pipe is laid. Whilst the rock through which the tunnel is to be constructed is envisaged to be very competent, there will be sections at the start and end of the tunnel where there would need to be a transition between the piped section and the tunnel. This transition also coincides with sections where the overburden and rock strength is insufficient to balance the internal hydraulic pressure, and where the tunnel would need to be lined.

Even when surge shafts are installed, all pressure tunnels are subject to hydraulic stressing due to the transient pressure surge effects. If the tunnel was not lined this could inevitably lead to water loss through the opening of cracks and seepage paths created by these internal positive pressures.

Analysis was undertaken to estimate for which sections each tunnel option would be able to be designed as a pressure tunnel.

A surge analysis was undertaken for the expected conduit sizing for the 37.5 MW installed capacity scheme. Unlike pumping stations, turbines do not instantaneously stop or start. The surge analysis thus simulated the sequential opening and closing of the control valves of the three turbines, each over more than 120 seconds, which would be a normal cold start and shut-down process.

The resulting minimum and maximum dynamic pressures along the conduit system have been calculated and are summarized in Table 7-4.

Table 7-4: Summary of Surge Analysis

			PRESSURE ELEVATION IN CONDUIT (m.a.s.l.)					
SURGE CONDITIONS			Tunnel Start (km 3.5)		Tunnel End (Km 6.85)		Turbines (km 7.85)	
Reservoir	**Tunnel**	**Valves**	**Max**	**Min**	**Max**	**Min**	**Max**	**Min**
FSL	Short	Opening	780.1	748.1	797.7	745.4	792.8	736.0
FSL	Short	Closing	776.8	752.1	794.6	751.7	792.1	741.4
FSL	Long	Opening	780.5	747.9			792.5	736.1
FSL	Long	Closing	776.9	752.1			791.8	741.5
MOL	Short	Opening	753.3	720.9	770.4	718.4	765.1	709.0
MOL	Short	Closing	750.1	725.1	767.6	724.7	765.1	714.4
MOL	Long	Opening	753.4	720.9			765.2	709.1
MOL	Long	Closing	750.1	725.1			765.1	714.5

Several criteria must be met if an unlined pressure tunnel section is to be considered suitable, as follows:

1. The crown of the water conduit must always be below the minimum dynamic water head (allowing for transients) to avoid negative pressure.
2. The maximum dynamic water pressure must not be above minimum principal stress Sigma 3 of the rock mass to prevent hydraulic jacking. For this analysis, a density of 26 kN/rn^3 is considered for the vertical stress. A minimum for the ratio of horizontal to vertical stress of 0.5 is assumed on the safe side. (Sigma horizontal (Sigma 3) = 0.5 x Sigma vertical (Gamma x h).

   Also a factor of safety of 1.1 is considered. If the max dynamic water pressure is higher than minimum principal stress, then a 100% watertight lining (steel or glass-fibre lining, concrete lining with membrane, etc.) is mandatorily required, with consequent cost implications.

3. The maximum static water pressure should be below ground water level to prevent water loss. If this is not achieved, then the situation has to be assessed in detail, and grouting might be required for rock mass sealing. Also water lost will increase the local groundwater level, which could cause piping or slope failure.

Analysis was undertaken for both the shallow and deep tunnel options and the pressure envelopes thus derived are shown in Figures 7-12 and 7-13.

In the case of the shallow tunnel, criteria 1 becomes an issue as the minimum hydraulic grade line is coincident with the tunnel alignment.

In both cases, criteria 2 results in the need to line sections of both tunnels as the overburden depth is insufficient to generate sufficient principal stress to resist the maximum dynamic water pressure.

In the case of the shallow tunnel some 927 m of the 3 300 m tunnel length (28%) would need to be fully lined if used as a pressurized conduit. In fact it would be likely that the tunnel would be fully lined from chainage 6 200 m to the portal rather than the two short sections shown. This would increase this requirement from 927 m to 1 310 m (40%).

In the case of the deep tunnel some 1 590 m of the 4 320 m tunnel length (37%) would need to be fully lined if used as a pressurized conduit. Such lining would be undertaken using either in-situ fabricated steel or glass reinforced plastic.

There are advantages and disadvantages to a pressurized tunnel verses a tunnel carrying a pipeline.

a) *Advantages*

1. The pressurized tunnel would have a larger diameter than a conduit with a pipeline laid through it, and would produce less head losses.
2. Construction would be less complex in that the installation of the steel pipeline would not be required. (However, the lining operation could produce a more complex overall construction process).
3. Construction cost could be slightly lower than the tunnel with a pipeline laid through, but this would need to be verified in the detailed design stage, once more geotechnical investigation is undertaken to determine tunnelling conditions and the water-tightness of the rock.

b) *Disadvantages*

1. The unlined section would produce water loss to some degree. This would reduce the hydropower output. This water loss would affect the local groundwater table and could find its way to the surface with unforeseeable consequences.
2. Great care would be required to prevent unexpected or excessive hydraulic transient pressures, which could hydro-fracture the unlined rock section. An expensive surge shaft would likely be required.
3. The flow velocities in the pressurized tunnel option would be significantly lower than the pipe-lined option, which could lead to sediment deposition within the tunnel section. The pipeline section would be self-cleansing and could otherwise be de-silted by periodical pigging if necessary.
4. The unlined tunnel section could be subject to rock degradation and spalling, which debris could pass through to the turbines.
5. Servicing and inspection of the tunnel would only be possible with the system closed down. For the other option, the pipeline could be inspected externally whilst in operation.

6. Significant transition works will be required at the interfaces between the piped conduit and the pressure tunnel section. This is normally in the form of embedded steel lining at the portals which must be pressure tight, and flanged connections to the piped sections. Access hatches would also be required for future inspection and maintenance of the tunnel.

7.6.4 Proposed Tunnel Section Configuration

The scope of geotechnical investigations undertaken at this stage was limited, and it is therefore recommended that significant additional geotechnical investigation drilling be undertaken to better ascertain tunnelling and rock mass conditions along the proposed tunnel alignment. This will be required to inform the detailed design of alternative solutions before a final decision is made.

However, for this feasibility design stage, it is considered preferable to design this section of the conduit as a dry tunnel through which the pipeline is laid continuously to the HEP plant.

The tunnel section would be sized such that there is room to install the steel pipeline on plinths, and to undertake the external butt welding of joints, and making good of the external coating.

It is suggested that the mini-rail system that is normally installed within the tunnel during construction be designed so that it can be used for transporting men, materials and equipment to the working face, removal of muck from the drill and blast operations, as well as carrying each steel pipe length and construction materials for plinths from the entrance portal to its point of installation. The welding of joints would be undertaken progressively from the lower end to the upper end of the tunnel.

Upon completion of the pipeline installation within the tunnel, there would be room alongside the pipeline for future inspection to be undertaken, which would include maintenance of both pipeline and tunnel.

Allowance has been made in the design of this solution for a reinforced and rock bolted shotcrete soffit lining to prevent any spalling of the tunnel roof from damaging the pipeline.

A typical section of the proposed feasibility design of the tunnel is shown in Figure 7-14.

This same configuration would apply to other pipeline diameters that might be considered for the peaking operation options, being 3 000 mm and 4 500 mm diameter respectively. Thus in those cases the tunnel would be proportionately larger, and have the same clearances around the pipeline.

As regards tunnel vertical alignment, it is also proposed for this feasibility design that the deeper, longer alignment be adopted. This this will avoid the need to construct an additional access road to the tunnel outlet portal, and the construction of the penstock section down the steep and potentially vulnerable route to the HEP.

The HEP will require a permanent and high specification access road to be constructed and this can also be used for the construction of the longer tunnel.

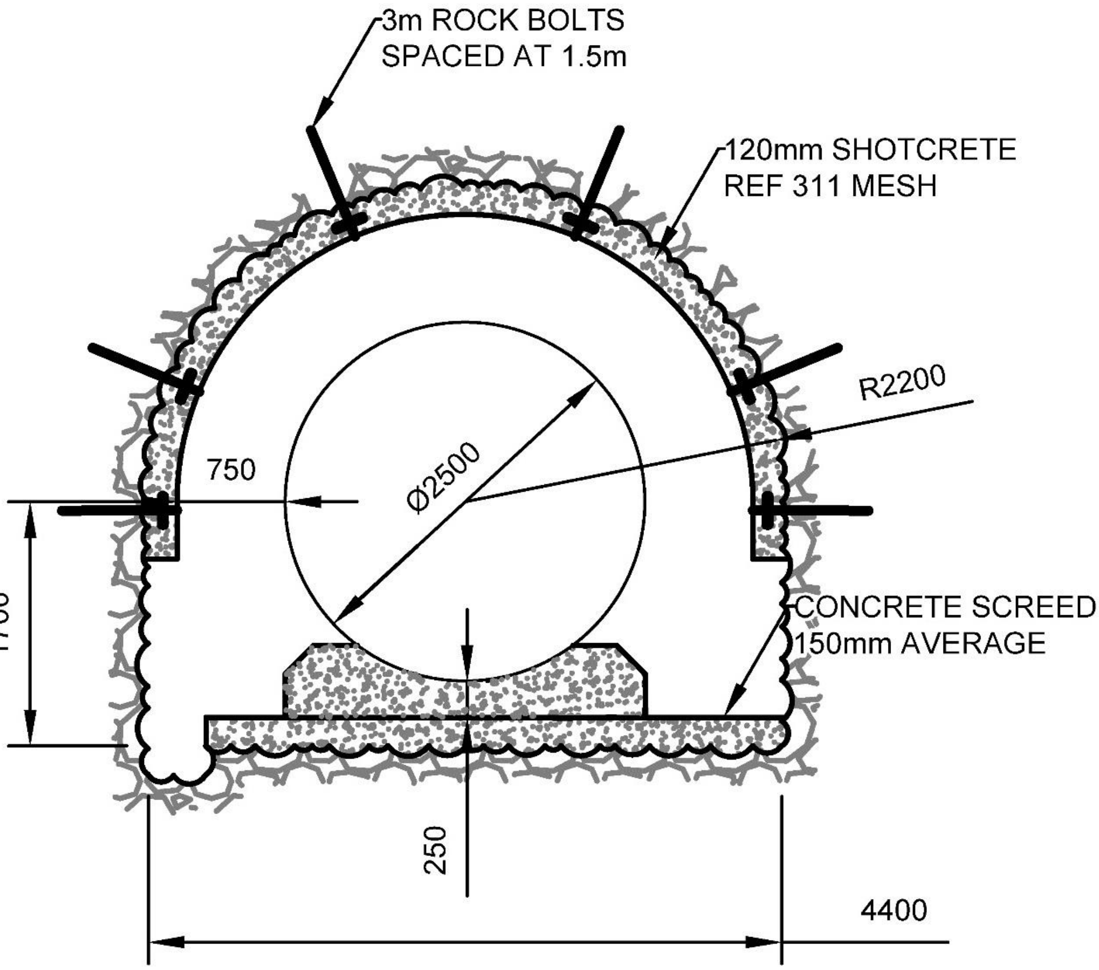


Figure 7-14: Typical Tunnel Section

7.6.5 *Hydropower Plant and Supply Conduit Configuration*

The HEP operational regime rules heavily influence the optimum plant and supply conduit configuration.

Given that the hydropower scheme comprises the conjunctive use of both Ntabelanga and Lalini Dams, the operating rules of both dams as determined by Environmental Water Requirements (EWR) must be considered.

a) Operating Rules: Ntabelanga Dam

This dam release flows down the Tsitsa River into the Lalini Dam and, together with the incremental inflow from the intervening catchment areas, thus supplementing the volume in Lalini Dam that can be utilized for hydropower generation and EWR purposes. In-stream losses are allowed for between the Ntabelanga and Lalini Dams.

The amount of water released downstream from the Ntabelanga Dam would be determined by operating rules which the dam operators will need to follow on a weekly basis. Based upon the recommendations of the EWR studies, the *minimum* amount released is determined by the monthly EWR with the same percentage occurrence as the measured inflow volume, as is given on the EWR flow duration curve for that particular calendar month. Thus the EWR releases will mimic the prevailing rainfall-runoff conditions in the catchment in any particular month, bearing in mind the historical flow patterns that occurred historically over the 90 year simulation period.

The *maximum* that can be released from the Ntabelanga Dam is generally limited to the simulated naturalized monthly flow with the same percentage of occurrence as the prevailing inflow as determined from the flow duration curves for that same calendar month. The exception to this is where the dam spills, and no constraints are applied.

During the hydropower generation model simulations it was noted that in extreme drought periods, the EWR volumes released did not always satisfy the hydropower generation needs to be sustained by the Lalini Dam balancing storage. In such cases it was agreed that up to 7 m^3/s could be released from Ntabelanga Dam downstream to sustain a minimum hydropower generation output and the EWR requirements at Lalini Dam.

Hydropower generation is achieved at Ntabelanga Dam by using the available head of water in the dam and passing the EWR releases through the mini-HEP located just downstream of the dam wall before returning this flow back to the river. This HEP diversion is limited to 16 m^3/s as EWR flows above this have a low recurrence interval, and it was considered not worth sizing the HEP plant and its conduit for a larger flow rate than this.

b) Operating Rules: Lalini Dam

The monthly inflow balancing regime as described for Ntabelanga Dam was modelled in the same way at Lalini Dam. In this case however, there is no potable or irrigation water requirement, but water is instead diverted through a 7.85 km long conduit to the main HEP located in the river gorge downstream of the Tsitsa Falls, and at an elevation of some 300 m below the Lalini Dam site. This arrangement is shown in Figure 7-15. The figure shows two tunnel options of which the deeper, direct option is recommended.

The HEP operational regime options are discussed in detail in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15, and the Hydropower Analysis: Lalini Dam Report No. P WMA 12/T30/00/5212/18.

As with the Ntabelanga Dam, the amount of water released downstream from the Lalini Dam would again be determined by operating rules which the dam operators will need to follow on a weekly basis. Based upon the recommendations of the EWR studies, the *minimum* amount released is determined by the monthly EWR with the same percentage occurrence as the measured inflow volume, as is given on the EWR flow duration curve for that particular calendar month.

In this case the water released from the Ntabelanga Dam would alter the natural Lalini inflow regime, and this will need to be taken into consideration when determining the precedent streamflow conditions in the Lalini catchment when setting the percentage occurrence factor to apply to the monthly flow duration curve, and thus the volume of EWR to be released in any particular month.

Hydropower generation is achieved at the Lalini Dam itself by using the available head of water in the dam and passing the EWR releases through the mini-HEP located just downstream of the dam wall before returning this flow back to the river. This HEP diversion is again limited to 16 m^3/s as EWR flows above this have a low recurrence interval, and it was considered not worth sizing the HEP plant and its conduit for a larger flow rate than this.

The hydropower simulation model always allows for the EWR to be released downstream of the Lalini dam before allowing water to be passed through the main HEP system via the conduit shown in Figure 7-15.

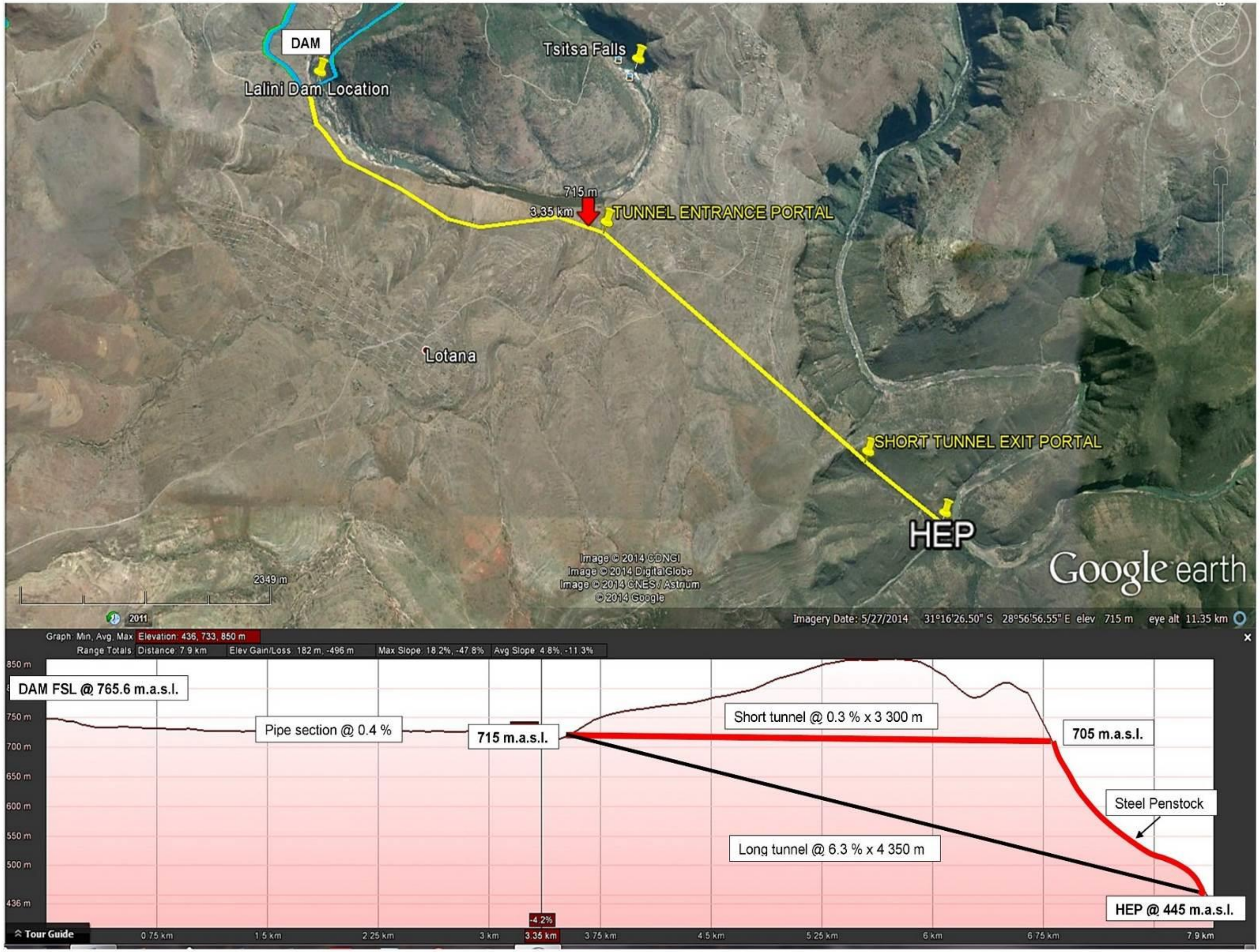


Figure 7-15: Lalini Main HEP System Arrangement

In order to determine how much water is to be passed through the main HEP plant, a target hydropower output was set for each month of the year. The model allows this to be undertaken quickly and iteratively until the maximum average energy output per year is achieved.

From the results that this produced it was noted that for a base load (24/7 operations) main HEP there was no merit in installing plant of capacity greater than 50 MW and, furthermore, this maximum installed capacity was often only fully useable in the one wettest month of the year.

In addition, in the drier months of the year, it was shown that the maximum power output would drop to around 5 to 15 MW, due to the need to limit the flow rate of water returned back into the river when mimicking the naturalized flow regime, as well as times in drought cycles when both Ntabelanga and Lalini Dams would be at their lowest levels.

If the rule of not exceeding the simulated naturalized flow regime for all months and percentage occurrences is strictly adhered to, then the main Lalini HEP scheme would need to be shut down or operated at a very low output level in a significant number of months in the driest years of operation.

This is exemplified in Table 7-5, which shows the percentage occurrences of various naturalised flow rates (expressed in m^3/s) over the 12 calendar months, taken from the monthly flow duration curves.

Table 7-6 shows the recommended minimum EWR releases in each calendar month, based upon the same percentage occurrences as the prevailing inflow conditions in the catchment.

Table 7-5: Simulated Naturalized Flows at Lalini Dam

	%age Occurrence of Naturalized Flow in m^3/s										
MONTH	**0%**	**10%**	**20%**	**30%**	**40%**	**50%**	**60%**	**70%**	**80%**	**90%**	**99%**
Oct	89.98	42.94	27.85	18.44	13.98	11.12	9.52	7.63	5.63	3.76	2.87
Nov	133.46	77.20	47.35	38.34	28.40	21.91	16.37	13.21	10.38	6.78	4.04
Dec	171.33	90.62	66.48	46.83	31.95	22.89	19.07	16.32	10.86	7.77	1.91
Jan	178.63	98.97	65.61	56.75	45.03	34.06	25.45	23.41	15.70	10.93	3.27
Feb	177.76	122.79	94.58	75.57	60.22	47.89	39.18	27.38	19.35	16.24	7.11
Mar	218.40	117.67	80.20	70.21	59.99	53.36	37.29	29.55	24.31	15.11	7.95
Apr	157.53	57.10	46.10	39.52	34.55	28.25	18.40	14.51	10.90	8.16	3.05
May	76.51	25.89	18.07	13.07	10.35	8.77	7.06	5.97	4.88	4.05	3.32
Jun	73.12	19.29	12.67	8.43	6.89	5.24	4.88	4.08	3.72	3.14	2.47
Jul	67.65	17.85	10.29	8.16	5.72	4.76	4.33	3.89	3.33	2.99	2.14
Aug	60.82	22.86	10.98	7.44	6.16	5.14	4.20	3.75	3.05	2.65	2.45
Sep	128.80	28.34	14.70	9.36	7.90	6.09	4.78	3.92	3.38	2.65	2.03
AVE	**127.83**	**60.13**	**41.24**	**32.68**	**25.93**	**20.79**	**15.88**	**12.80**	**9.62**	**7.02**	**3.55**

Table 7-6: Desktop Class BC EWR at Lalini Dam

	%age Occurrence of EWR in m^3/s										
MONTH	**0%**	**10%**	**20%**	**30%**	**40%**	**50%**	**60%**	**70%**	**80%**	**90%**	**99%**
Oct	9.18	9.18	9.07	8.81	8.28	7.37	6.04	4.44	2.96	1.95	1.56
Nov	10.88	10.88	10.76	10.46	9.87	8.81	7.26	5.38	3.60	2.40	1.94
Dec	13.53	13.53	13.42	13.16	12.63	11.66	10.09	7.89	5.39	3.26	1.91
Jan	25.49	25.49	22.81	20.51	18.36	14.54	12.62	9.91	6.80	4.13	2.89
Feb	51.87	51.87	45.40	39.93	35.01	26.30	22.68	17.63	11.86	6.96	4.67
Mar	46.42	46.42	39.95	34.54	29.62	21.66	17.74	13.00	8.53	5.50	4.34
Apr	9.69	9.69	9.58	9.33	8.82	7.93	6.65	5.10	3.66	2.69	2.31
May	6.48	6.48	6.41	6.24	5.90	5.31	4.45	3.43	2.46	1.81	1.57
Jun	3.63	3.63	3.58	3.47	3.25	2.89	2.42	1.93	1.55	1.33	1.26
Jul	3.18	3.18	3.13	3.03	2.83	2.51	2.10	1.68	1.35	1.17	1.10
Aug	2.95	2.95	2.91	2.82	2.64	2.35	1.97	1.57	1.26	1.09	1.03
Sep	7.43	7.43	7.34	7.13	6.72	6.00	4.78	3.70	2.52	1.73	1.43
AVE	**15.90**	**15.90**	**14.53**	**13.28**	**11.99**	**9.78**	**8.23**	**6.30**	**4.33**	**2.83**	**2.17**

Table 7-7 shows the water thus available to be passed through the main Lalini HEP under the same prevailing catchment conditions, being the difference between the naturalised and EWR flow figures.

The cells highlighted in Table 7-7 are those where available average monthly flow would be insufficient to operate the main HEP at its minimum output (one turbine set operating) continuously throughout the month. In the wetter months, this only occurs between 10 and 20% of the years, but in the dry season months this reduced output could occur to a lesser or greater degree up to 60% of the years.

The flow rate required to operate a single 12.5 MW turbine unit continuously is some 6 m^3/s. The operational regime proposed is to therefore make use of the available balancing capacity in the dams to pass a minimum of 6 m^3/s through the main Lalini HEP turbines in the particularly low flow dry season months in order to ensure that a minimum of 12.5 MW can always be produced by the main HEP at all times.

Table 7-8 (based on the 37.5 MW installed capacity option) shows the impact of strictly limiting the main HEP flow throughput to the naturalized flow regime, and it is evident that the power outputs in dry season months could be low for a significant proportion of the years of simulation.

The highlighted cells in Table 7-9 show the quantum of water that would be required to be released through the main HEP extra over the naturalized flow regime values, and the percentage occurrence of when this would be required (e.g. 80% actually means this would only be required 20% of the time).

As can be seen this additional release amount averages less than 3 m^3/s, but in some drought years could be up to the maximum 6 m^3/s, albeit that this would be a rare occurrence.

Table 7-7: Flow Available for Hydropower Generation

	%age Occurrence of Flow Available for Hydropower Generation (m^3/s)										
MONTH	0%	10%	20%	30%	40%	50%	60%	70%	80%	90%	99%
Oct	80.80	33.76	18.78	9.63	5.70	3.75	3.48	3.19	2.67	1.81	1.30
Nov	122.58	66.32	36.59	27.88	18.53	13.10	9.11	7.84	6.78	4.38	2.10
Dec	157.79	77.09	53.07	33.68	19.32	11.22	8.98	8.43	5.47	4.51	0.00
Jan	153.14	73.48	42.81	36.25	26.67	19.52	12.83	13.50	8.90	6.80	0.38
Feb	125.89	70.92	49.19	35.64	25.20	21.59	16.50	9.76	7.49	9.29	2.44
Mar	171.97	71.25	40.26	35.67	30.37	31.70	19.55	16.55	15.78	9.61	3.61
Apr	147.84	47.41	36.51	30.19	25.73	20.31	11.76	9.40	7.24	5.47	0.73
May	70.03	19.40	11.66	6.83	4.45	3.46	2.61	2.54	2.42	2.24	1.76
Jun	69.49	15.66	9.08	4.96	3.65	2.35	2.46	2.15	2.17	1.81	1.22
Jul	64.47	14.67	7.16	5.13	2.89	2.25	2.23	2.21	1.97	1.82	1.04
Aug	57.87	19.91	8.07	4.63	3.52	2.79	2.23	2.18	1.78	1.57	1.42
Sep	121.37	20.91	7.36	2.22	1.18	0.09	0.00	0.22	0.85	0.92	0.60
AVE	111.94	44.23	26.71	19.39	13.93	11.01	7.65	6.50	5.29	4.18	1.38

Table 7-8: Main HEP Power Output without Supplementary Release through HEP

	%age Occurrence of HEP Output (MW) - No Supplementary Release										
MONTH	0%	10%	20%	30%	40%	50%	60%	70%	80%	90%	99%
Oct	37.5	37.5	37.5	18.6	11.0	7.2	6.7	6.1	5.1	3.5	2.5
Nov	37.5	37.5	37.5	37.5	35.7	25.2	17.5	15.1	13.1	8.4	4.0
Dec	37.5	37.5	37.5	37.5	37.5	21.6	17.3	16.2	10.5	8.7	0.0
Jan	37.5	37.5	37.5	37.5	37.5	37.5	24.7	26.0	17.1	13.1	0.7
Feb	37.5	37.5	37.5	37.5	37.5	37.5	31.8	18.8	14.4	17.9	4.7
Mar	37.5	37.5	37.5	37.5	37.5	37.5	37.5	31.9	30.4	18.5	7.0
Apr	37.5	37.5	37.5	37.5	37.5	37.5	22.7	18.1	13.9	10.5	1.4
May	37.5	37.5	22.5	13.2	8.6	6.7	5.0	4.9	4.7	4.3	3.4
Jun	37.5	30.2	17.5	9.6	7.0	4.5	4.7	4.1	4.2	3.5	2.3
Jul	37.5	28.3	13.8	9.9	5.6	4.3	4.3	4.3	3.8	3.5	2.0
Aug	37.5	37.5	15.5	8.9	6.8	5.4	4.3	4.2	3.4	3.0	2.7
Sep	37.5	37.5	14.2	4.3	2.3	0.2	0.0	0.4	1.6	1.8	1.2
AVE	37.52	36.14	28.84	24.12	22.04	18.77	14.72	12.51	10.20	8.06	2.67

As shown in Table 7-10, the benefits of this additional release allowance within the EWR rules are obvious, in that on average, some 10% more power can be generated by the same HEP configuration than if the additional release is not allowed.

This situation was presented to the team undertaking the Lalini EWR study and the consensus was that such releases would not significantly change the ecological regime of the river below the HEP outlet, and therefore could be allowed.

Following review and discussion of the EWR Report, the DWS RDM office has approved the operational regime whereby an additional 6 m^3/s over naturalized flow can be passed through the HEP turbines and released back to the river as and when required in any month.

Table 7-9: Water Released Through HEP Extra Over Naturalized Flow to Maintain 12.5 MW

	% age Occurrence of Water Released Over Naturalized Flow (m^3/s) to Maintain 12.5 MW Output										
MONTH	0%	10%	20%	30%	40%	50%	60%	70%	80%	90%	99%
Oct	0.00	0.00	0.00	0.00	0.30	2.25	2.52	2.81	3.33	4.19	4.70
Nov	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.62	3.90
Dec	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.53	1.49	6.00
Jan	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.62
Feb	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.56
Mar	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.39
Apr	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.53	5.27
May	0.00	0.00	0.00	0.00	1.55	2.54	3.39	3.46	3.58	3.76	4.24
Jun	0.00	0.00	0.00	1.04	2.35	3.65	3.54	3.85	3.83	4.19	4.78
Jul	0.00	0.00	0.00	0.87	3.11	3.75	3.77	3.79	4.03	4.18	4.96
Aug	0.00	0.00	0.00	1.37	2.48	3.21	3.77	3.82	4.22	4.43	4.58
Sep	0.00	0.00	0.00	3.78	4.82	5.91	6.00	5.78	5.15	5.08	5.40
AVE	0.00	0.00	0.00	0.59	1.22	1.78	1.92	1.96	2.06	2.46	4.62

Table 7-10: Main HEP Power Output with Supplementary Release through HEP (MW)

	%age Occurrence of HEP Output (MW) - With Supplementary Release										
MONTH	0%	10%	20%	30%	40%	50%	60%	70%	80%	90%	99%
Oct	37.5	37.5	37.5	19.3	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Nov	37.5	37.5	37.5	37.5	37.1	26.2	18.2	15.7	13.6	12.5	12.5
Dec	37.5	37.5	37.5	37.5	37.5	22.4	18.0	16.9	12.5	12.5	12.5
Jan	37.5	37.5	37.5	37.5	37.5	37.5	25.7	27.0	17.8	13.6	12.5
Feb	37.5	37.5	37.5	37.5	37.5	37.5	33.0	19.5	15.0	18.6	12.5
Mar	37.5	37.5	37.5	37.5	37.5	37.5	37.5	33.1	31.6	19.2	12.5
Apr	37.5	37.5	37.5	37.5	37.5	37.5	23.5	18.8	14.5	12.5	12.5
May	37.5	37.5	23.3	13.7	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Jun	37.5	31.3	18.2	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Jul	37.5	29.3	14.3	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Aug	37.5	37.5	16.1	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
Sep	37.5	37.5	14.7	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5
AVE	37.52	36.32	29.11	25.67	24.97	22.81	19.24	17.16	14.99	13.66	12.50

In terms of the 37.5 MW scheme described above, a 2 500 mm diameter conduit would be required to provide the optimum transfer capacity.

A 50 MW scheme would require the conduit size to be increased to 3 000 mm diameter, and this scenario was also investigated and costed for the purposes of completeness.

A further scenario is also investigated whereby the scheme is operated as a peaking station only. In such a case, some 150 MW of power generation would be installed and operated for a limited number of hours per day to focus only on earning the highest tariff rates. In such a case, the conduit size would need to be 4 500 mm diameter.

Costing and economic analysis have been undertaken for these scenarios, and the recommended solution is the 37.5 MW installed capacity and a 2 500 mm diameter conduit.

7.7 Regulation of Flow below HEP Outlet

When operated as a base load (24/7) station, there would be no need to regulate the recombined EWR and HEP discharges downstream of the HEP plant outlet, as these would fall within the accepted operating rules determined following the Reserve Determination and EWR studies.

Should the base load (37.5 MW) station be operated as a peaking station in the winter/dry season months, then a typical scenario would be that the full installed capacity turbines were operated over (say) 8 peak hours per day instead of 12.5 MW over 24 hours, thus using the same daily volume of water available.

In order to ensure that the recombined flows are balanced, regulated, and normalized back to a 24 hour regime, a regulating dam and storage facility would need to be constructed in-stream with a minimum storage capacity of 16 hours of the daily HEP flow under the prevailing conditions. In this case, this would require a minimum balancing dam capacity of 375 000 m^3.

Should a full-time peaking station be installed (up to 150 MW), then this requirement increases significantly as the peaking operations would be concentrated to 3 to 5 hours per day, and the balancing storage requirement would rise to as high as 2 million m^3.

For the former base load option, this balancing storage would extend approximately 500 m downstream of the HEP discharge location, and for the latter peaking option this body of balancing storage could extend as far as 1 500 m downstream and require a dam wall height of 15 m or more.

Such in-stream balancing storage would have its own impact on the environment by drowning the river bed flora and fauna at that location and significantly changing its natural state.

It would also be very difficult to adequately regulate outflow rates from this storage.

The storage would also act as a sediment trap and would rapidly lose its capacity to regulate flow.

In conclusion, it is considered to be highly unlikely that such a balancing regime would be practical or environmentally acceptable, and this further supports the conclusion that the most likely solution is the 37.5 MW installed capacity and a 2 500 mm diameter conduit, operated as a base load station.

This would still allow for the HEP station to be operated as a peaking station in the winter months in years when the flow regime is not in a drought condition.

7.8 Main Hydropower Plant Configuration

7.8.1 Electro-Mechanical Equipment

Internationally-renowned hydropower plant manufacturers from Europe were consulted to determine suitable hydropower generating plant types, design details, performance, costs, installation requirements and general arrangements.

For the 37.5 MW and 50 MW plant options, and the likely monthly generating regime, it was recommended that three or four (net) 12.5 MW units would be best suited to match the head verses flow regime. The basis of feasibility design presented herein is for the 37.5 MW solution.

The turbines proposed are of the vertical Pelton type with 6 jet nozzles. Depending upon the eventual procurement process and manufacturer selected, the number and configuration of jet nozzles could vary.

The proposed arrangement is overhung, i.e. the turbine runner is mounted directly onto an extended and reinforced generator shaft. All remaining (small) axial thrust and radial loadings on the turbine runner created by rotational speed, jet impact and weight are therefore taken by a suitably designed generator shaft/bearing system.

The main cooling of the generator is by water cooling and therefore requires a two circle cooling system.

Typical arrangements and photos of plants of a similar capacity are given in Figures 7-16 to 7-22. Please note these are generic examples and not specific to this project.

The specification of these turbines was based upon the following:

Headwater level	740.00	m.a.s.l.
Tailwater level	420.00	m.a.s.l.
Gross Head	320.00	m
Rated flow per turbine	6.25	m^3/s
Total losses (4 turbine operation)	45	m
Rated net head (4 turbine operation)	275	m
Rated output at 1 turbine shaft (4 turbine operation)	15.24	kW
Rated output at grid connection	12.50	kW
Turbine speed	500	min-1

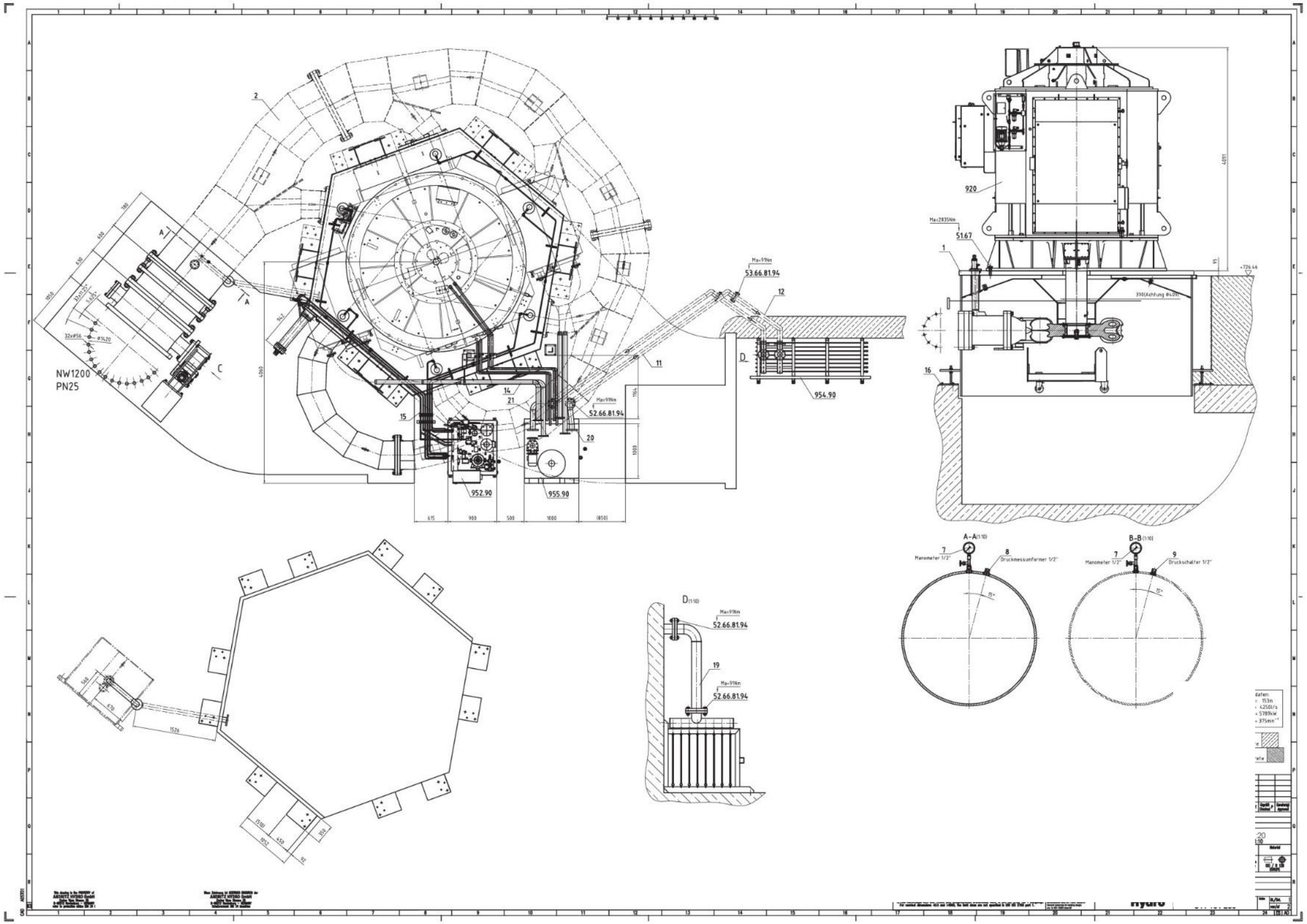


Figure 7-16: Installation Arrangement of a Similar Pelton Wheel Turbine

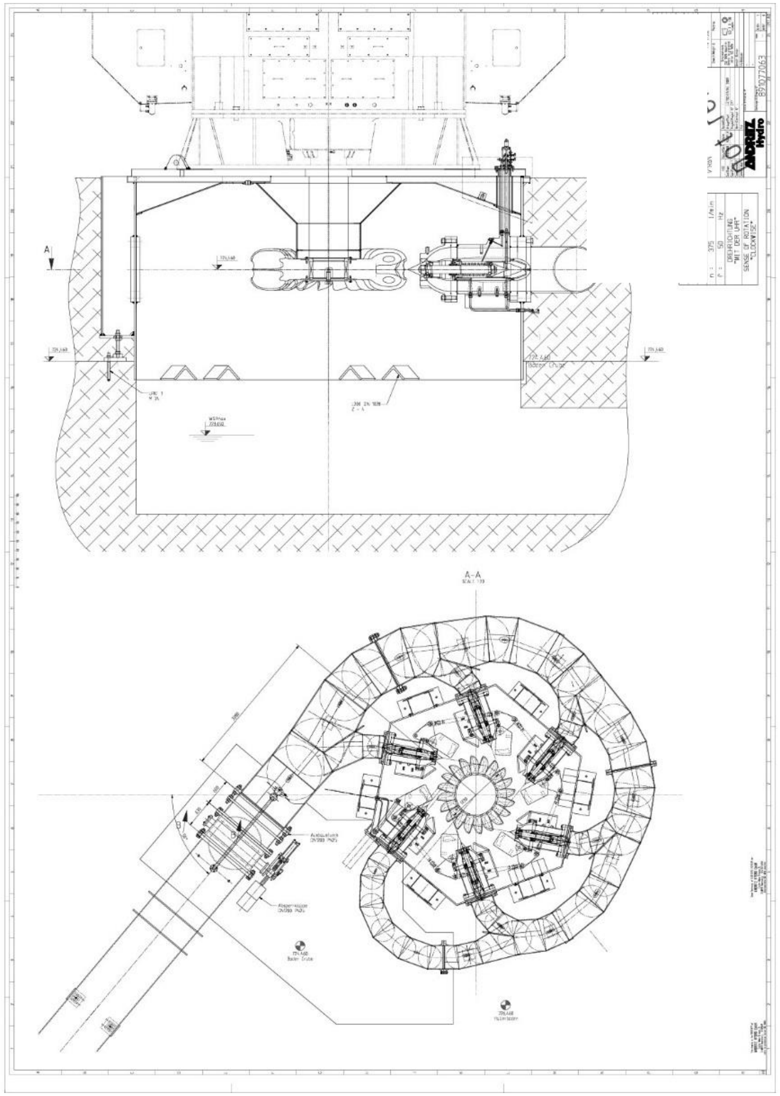


Figure 7-17: Detail of Pelton Runner and Jet Arrangement

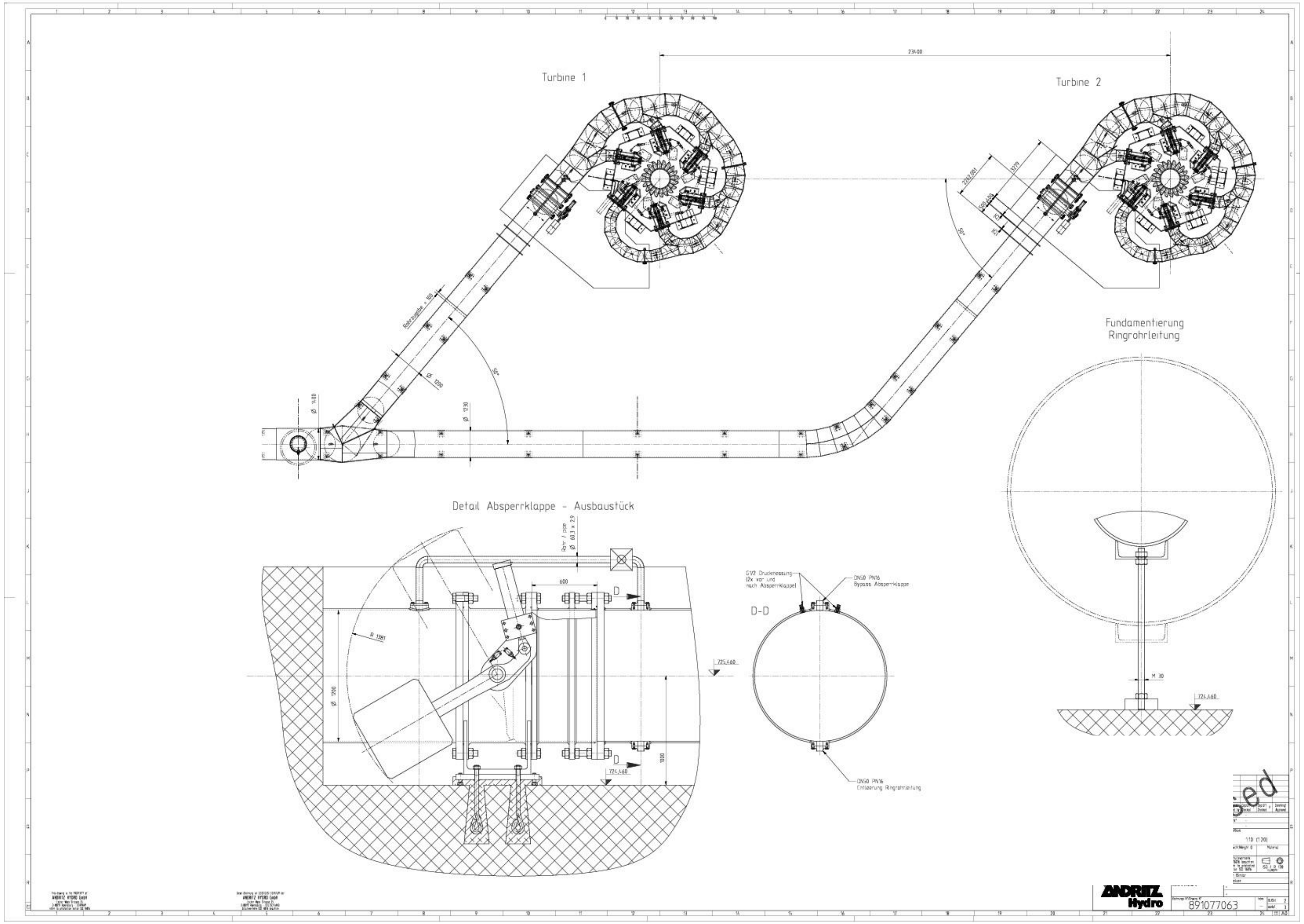


Figure 7-18: Typical Installation of Adjacent Turbines and Main Control Valve

Figure 7-19: Photo of Similar Sized Pelton Wheel Generator Installation

Figure 7-20: Photos of Installation Process

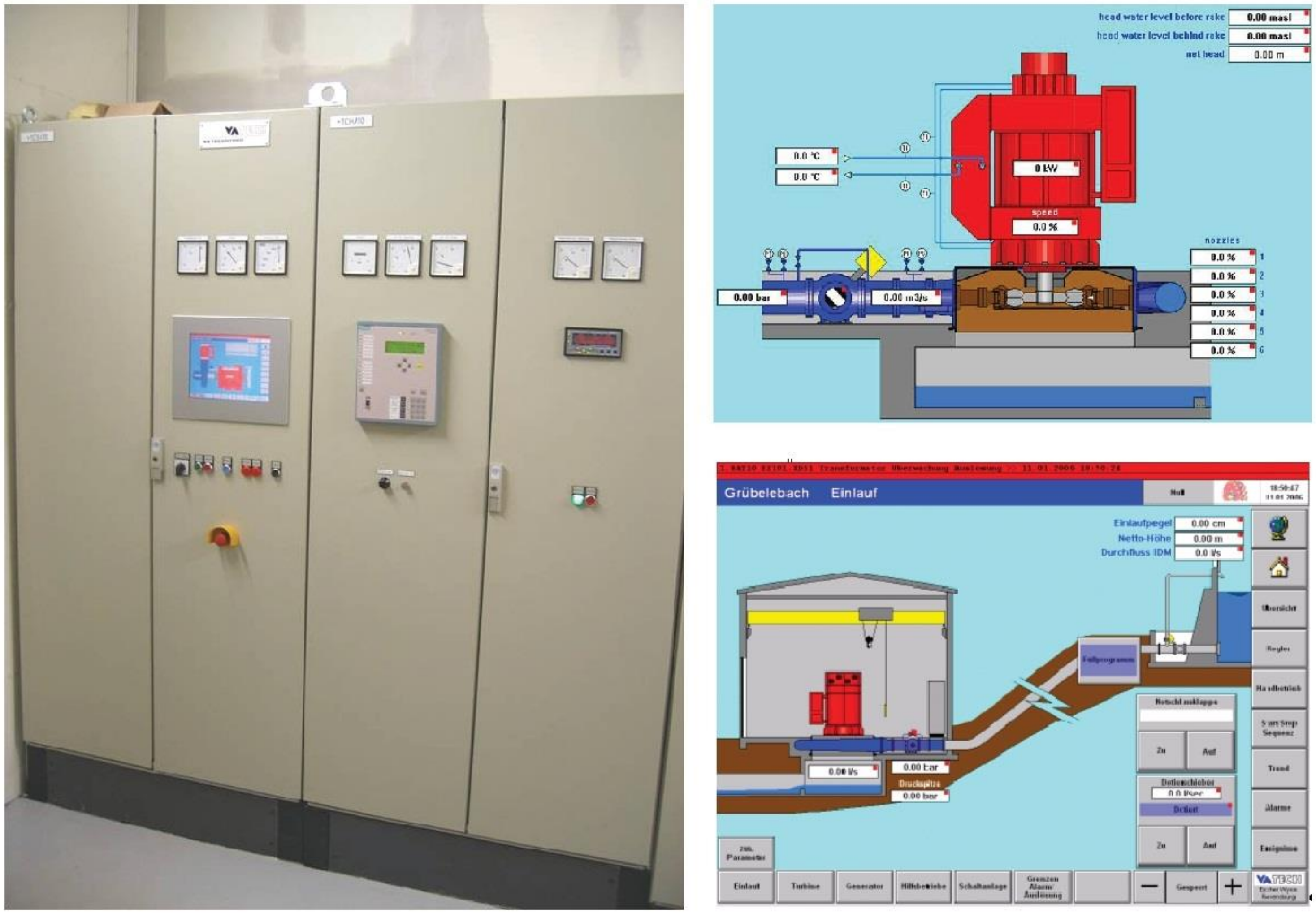


Figure 7-21: Example of Control Panel and Mimic Diagram Display

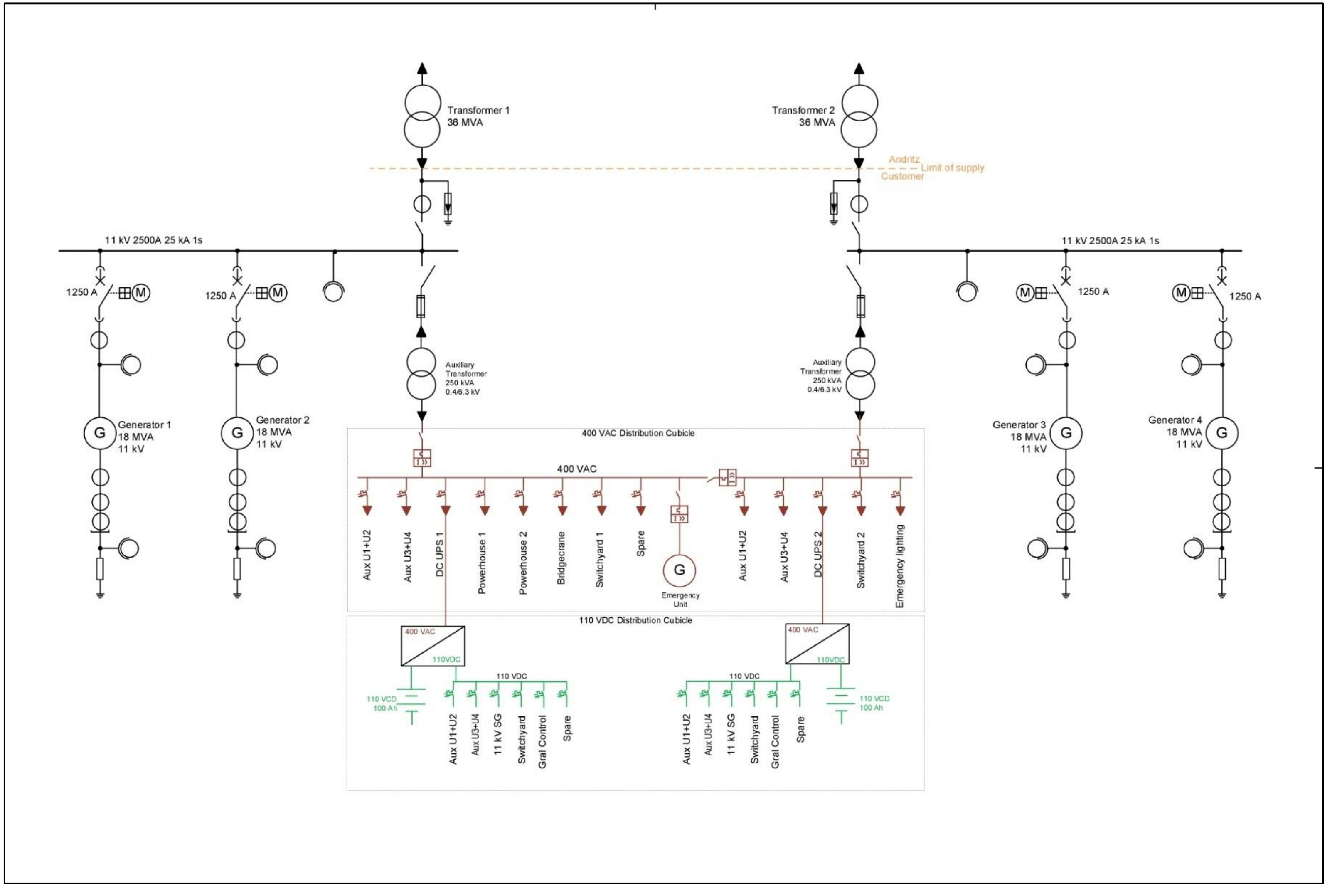


Figure 7-22: Typical Single Line Diagram for a Similar 4 Turbine Installation

7.8.2 Hydropower Plant Structure

The structure to house the HEP is designed to meet the functionality requirements of the plant as well as the construction and installation sequencing required for this type of turbine.

A two-stage basement concrete placement is required, and cut-outs in the basement are required to allow operational valves and turbine jet volute casings to be accessed and maintained.

Channels are also included below the Pelton wheel runner to carry the water away from the plant once the jet energy has been absorbed.

Each of these channels must be able to carry a minimum of 6.5 m^3/s. Upon leaving the structure basement, the flow is discharged down the bank of the river via a stepped energy dissipating cascade system founded on good rock and constructed using reinforced concrete and gabion systems.

Specific spacing of each generator is important to avoid interference with each other with respect to both vibration and high voltage current.

This results in a long and narrow building layout as shown in Figure 7-23. This figure is for a 3 x 12.5 MW turbine solution. If an additional turbine is to be installed, then the building would be proportionately longer.

This building would require adequate lighting, heating, and ventilation and will have a sound-proofed control room at one end.

The generator is the heaviest single component of the generating set, and each would have a weight of some 75 tonnes, with each turbine weighing some 35 tonnes.

The building would be equipped with a suitable overhead crane, and has access doors between each generator set so that transport vehicles can reverse into the building for delivery and replacement of these components.

The HEP building is positioned adjacent to the tunnel exit portal so that the pipeline penstock exiting the tunnel can be connected to the HEP inlet pipework below the hard-standing area.

This site layout and cross-section is shown on Figures 7-24 and 7-25.

This shows a diagram of the earthworks and hard-standing areas required between the tunnel and HEP building, as well as the discharge cascades returning hydropower flow back to the river.

This hard-standing platform and access road thereto would be required as a first priority so that the tunnel and HEP building construction can be undertaken.

This will also require a power supply and water supply to be brought to the location for construction and long-term usage.

The water supply would be developed by a package plant abstracting from the river, and the power supply could share the same power line as would eventually be used to evacuate energy from the HEP into the grid. However, the means of implementing this power supply aspect would be at the discretion of ESKOM.

It is proposed that operators of the HEP would be housed in the same staff housing compound as is to be developed for the Lalini Dam, and would commute via the access road each day.

A small ablution and mess block should be provided at the HEP building.

As shown on the layout diagram, a separate transformer compound is located next to the control room end of the HEP building.

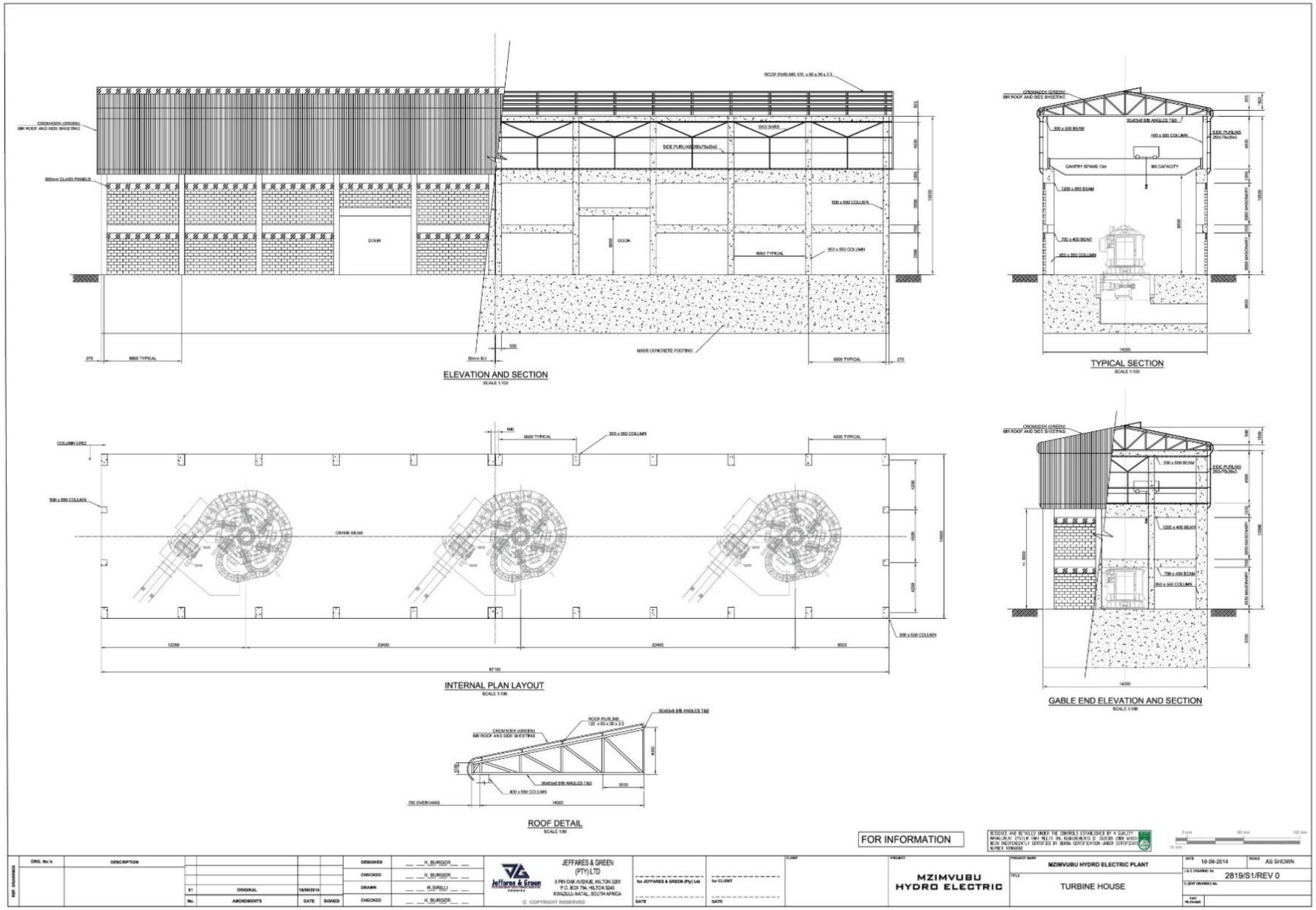


Figure 7-23: Hydroelectric Power Plant Building (3 Turbine Option)

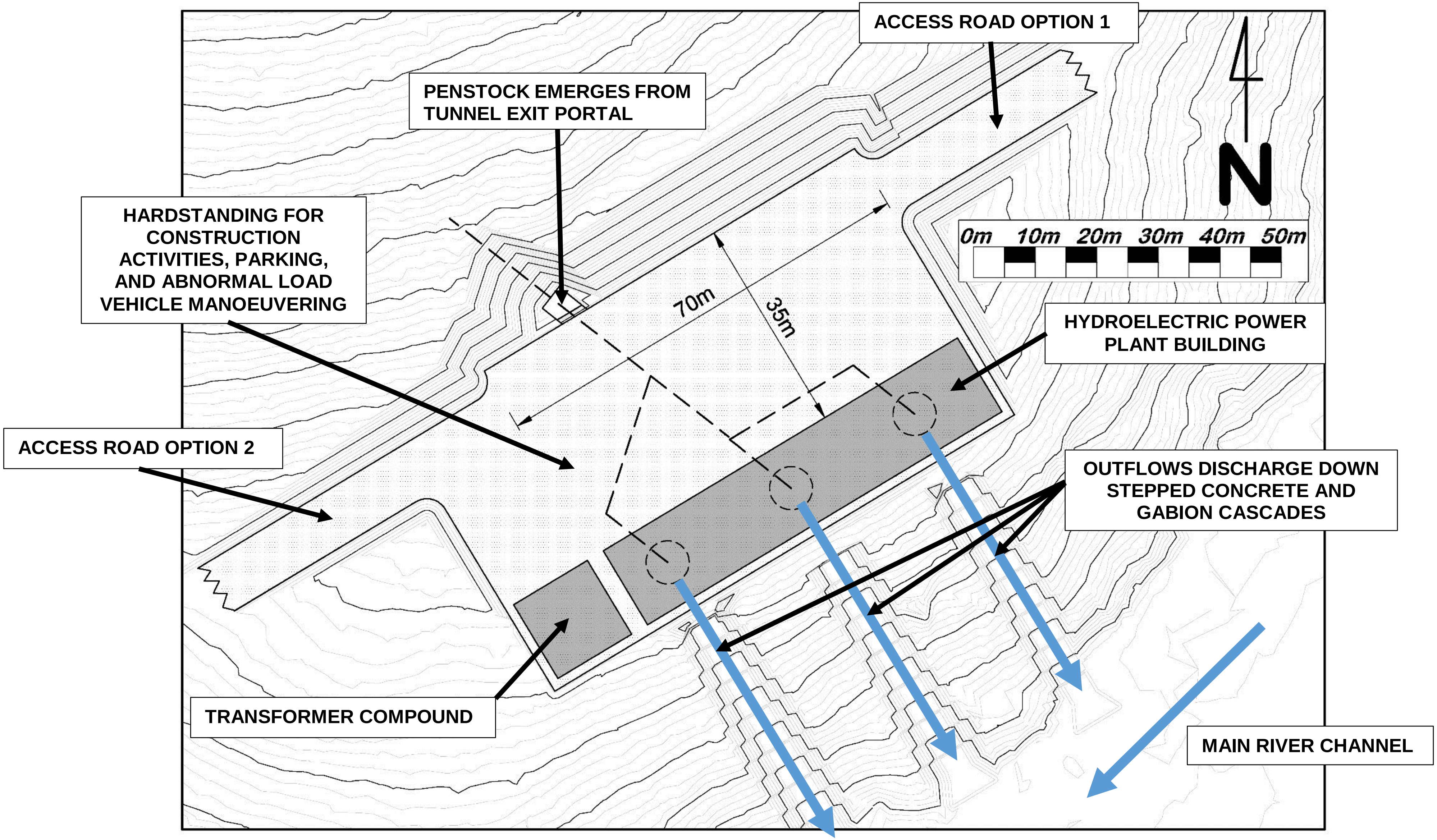


Figure 7-24: Lalini Main Hydropower Plant Site Layout

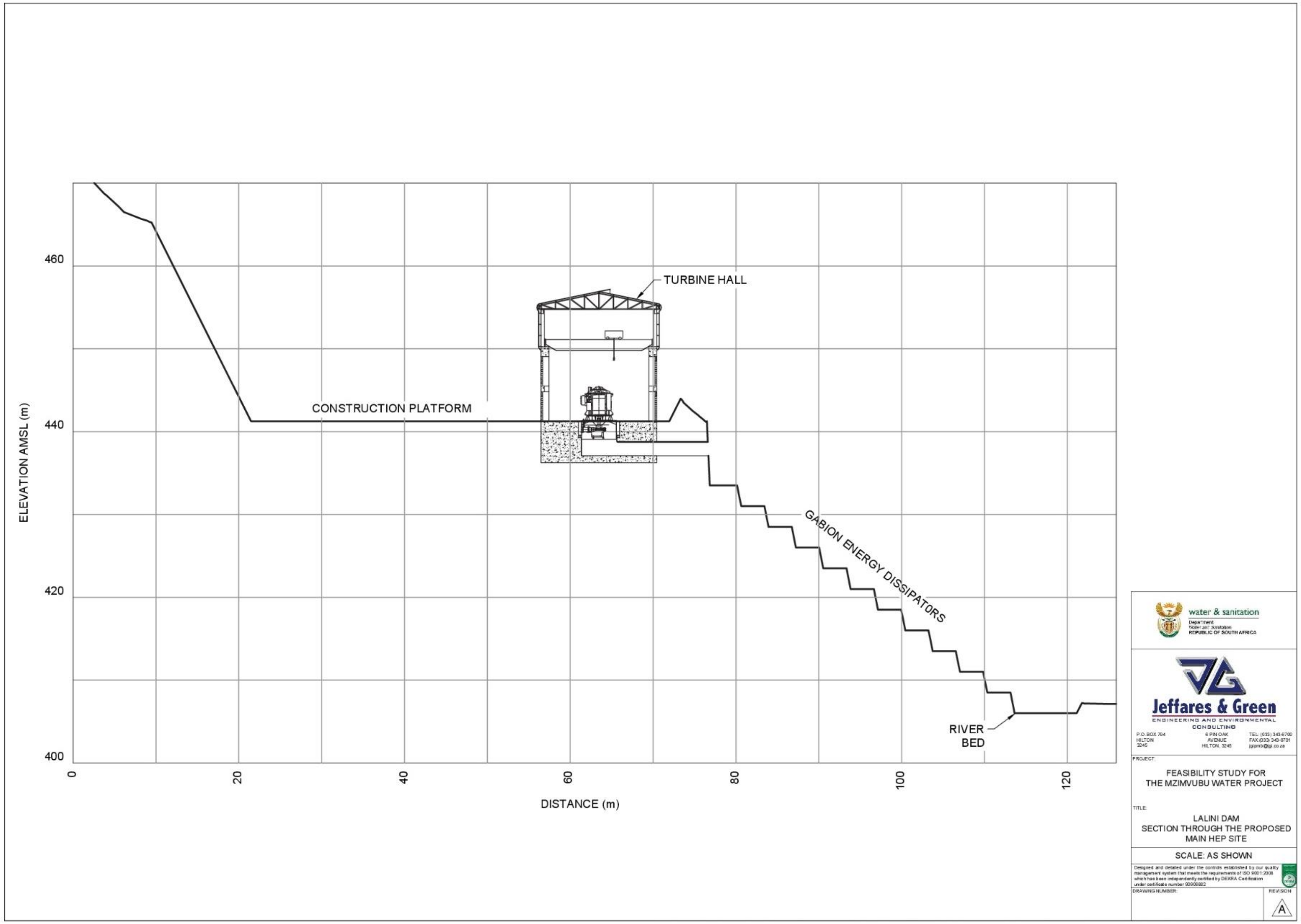


Figure 7-25: Turbine House and Outlet Works Cross-section

7.9 Lalini Dam Mini-Hydropower Plant

As with the Ntabelanga Dam, the environmental water requirements (EWR) released from the Lalini Dam into the river above Tsitsa Falls creates an opportunity for some additional hydropower to be generated at this location.

The Lalini Dam: Hydropower Analysis Report No. P WMA 12/T30/00/5212/18 describes the conjunctive scheme hydropower modelling simulations undertaken and indicates that up to 5 MW can be generated in the wetter months, with seasonal availability of EWR determining outputs that can be achieved in other seasons.

The results of the analysis for the 0.28 MAR Lalini Dam are as shown in Table 7-5 and Figure 7-26.

Table 7-11: Recommended Lalini Dam HEP Outputs

Month	Minimum Target (MW)	Avg HP Output (MW)	Avg Energy Supplied (KWh)
Oct	2.00	1.41	1 047 895
Nov	3.00	1.74	1 251 338
Dec	3.00	2.34	1 742 819
Jan	4.00	3.10	2 303 120
Feb	5.00	3.90	2 644 895
Mar	5.00	3.91	2 910 565
Apr	5.00	1.74	1 249 716
May	4.00	1.22	905 288
Jun	3.00	0.66	476 106
Jul	1.00	0.59	440 637
Aug	1.00	0.54	401 078
Sep	1.00	0.81	585 678
Total Energy Per Year (kWh)			**15 959 136**
Average Power (MW)		**1.83**	

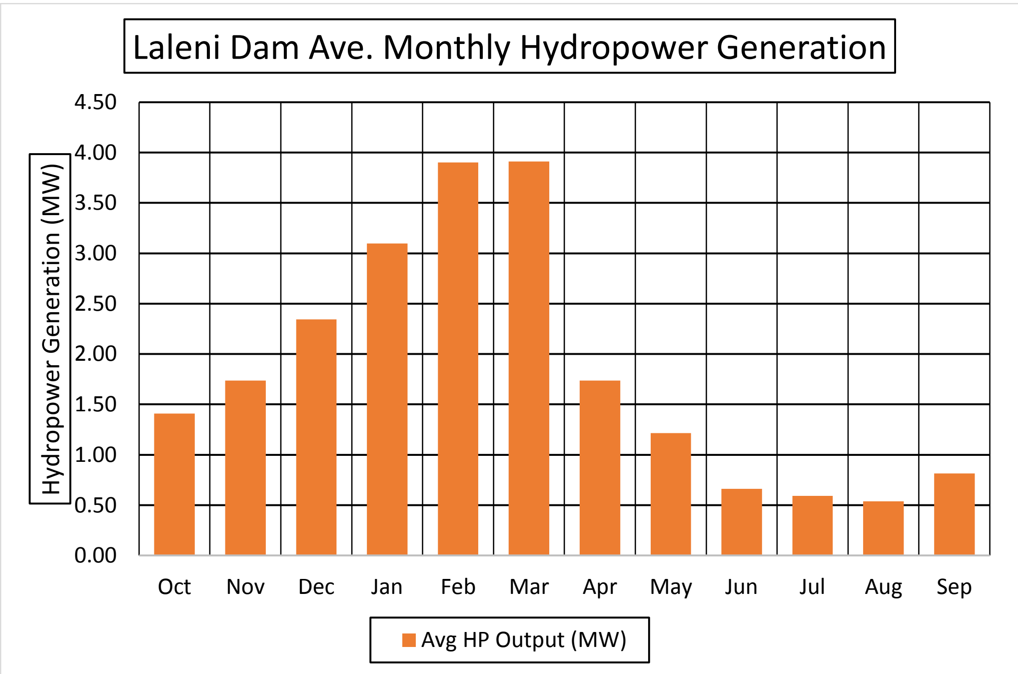


Figure 7-26: Lalini Dam HEP Average Monthly Hydropower Generation

Thus the hydropower plant configuration has been based upon a target operating range of between 1 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 (blackstart 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 tachometer for speed control, 2 PT100 sensors (1 per bearing) to check bearing temperature and also 2 vibration sensors (1 per bearing). Typical pump-turbine units suggested were:

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

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

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

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

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

Table 7-12: Lalini Mini-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 7-27 shows a proposed layout of the hydropower turbine house together with the inlet and outlet pipework arrangements.

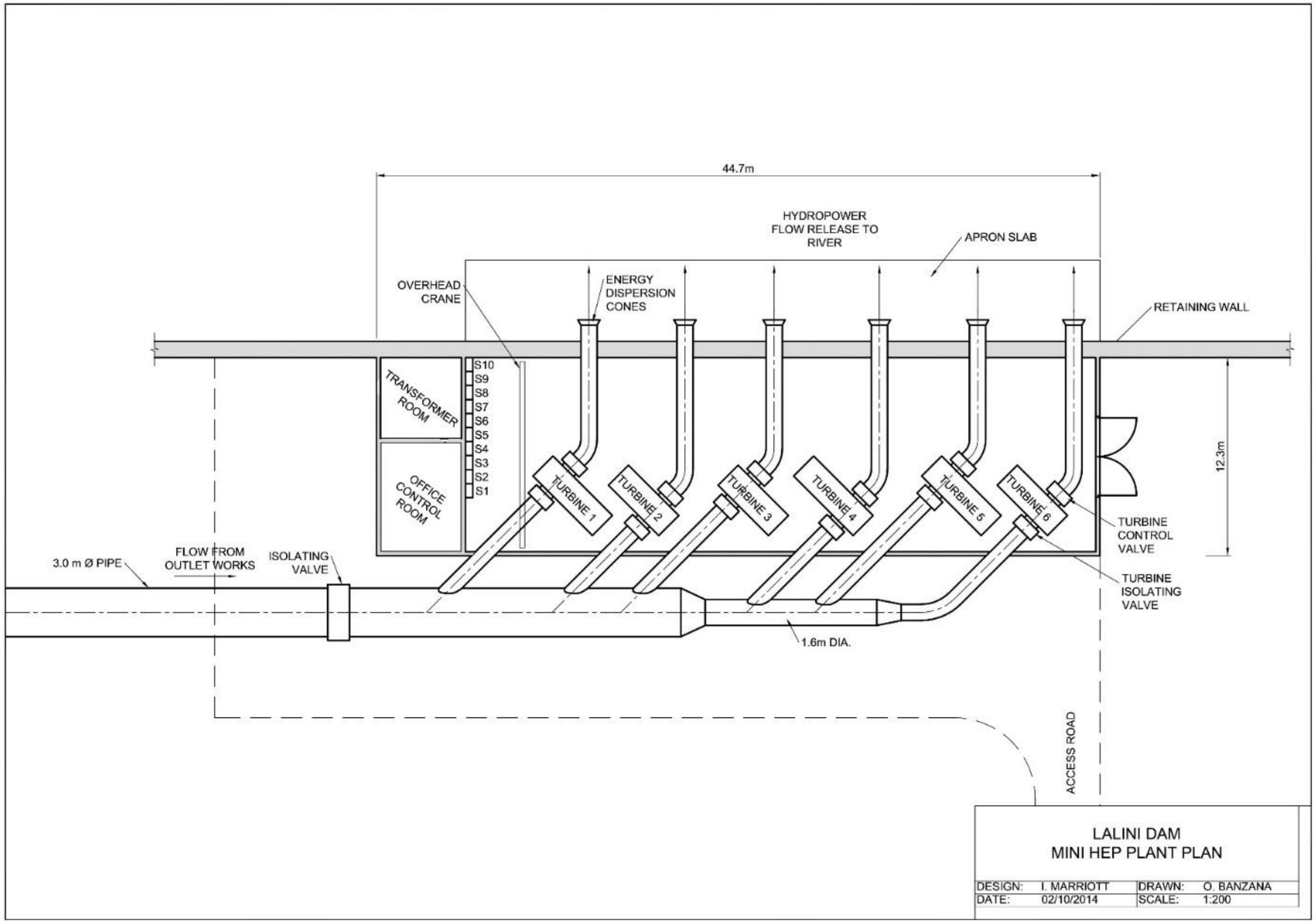


Figure 7-27: Proposed Layout of Lalini Dam HEP and EWR Discharge Point

When the hydropower plant is not in use, release of water for EWR purposes can still be made via a sleeve valve in the main dam outlet 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 Lalini Dam are further discussed in detail in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15.

8. ASSOCIATED INFRASTRUCTURE

8.1 Construction and Permanent Access Roads

Some major road works will be required for the construction and long-term operation of the scheme.

In general, road designs, realignments and upgrades have been designed in accordance with the South African Technical Recommendation for Highways (TRH) standards for such work as detailed in the following documents;

1. TRH 4 : Structural design of Flexible Pavements
2. TRH 17: Geometric Design of Rural Roads
3. TRH 20: The Structural Design Construction and Rehabilitation of Unpaved Roads

8.1.1 Main Access Road

Figure 8-1 shows the existing District Road DR 08170 linking the N2 national road near to the Tsolo to Maclear road junction with the villages of Lotana and Lalini in the vicinity of the dam and hydropower infrastructure locations.

This existing gravel road also services the settlements of Madadeni, Gwali, Upper Lotana, Cingcosdwadeni, Ngcolorha, Manzimabi, Mahoyana, and Mbutho.

This 17.4 km “Main Access Road” provides the best access to the dam and tunnel construction sites from the main road and does not have any major bridge crossings to contend with. Some donga crossing would need to be widened and upgraded to carry heavy loads.

Figures 8-2 to 8-5 show typical sections of this existing road.

In addition to construction traffic, this road would be the main route used for the delivery of the heavy electromechanical components of the HEP, which will require abnormal load vehicles able to transport loads of up to 100 tonnes.

Thus it is proposed that this road be upgraded geometrically and structurally to cater for heavy construction traffic and abnormal vehicles that are anticipated to be used in the construction activities. This district road would, however, remain a gravel surfaced road. Provision has been made in the costing to refurbish the upper base courses to a high standard gravel road once construction has been completed in order to ensure that the road is handed back to the Provincial Roads Department in an acceptable state.

From this main access road, several new roads will need to be constructed for both construction and permanent access purposes. These are shown on Figure 8-6.

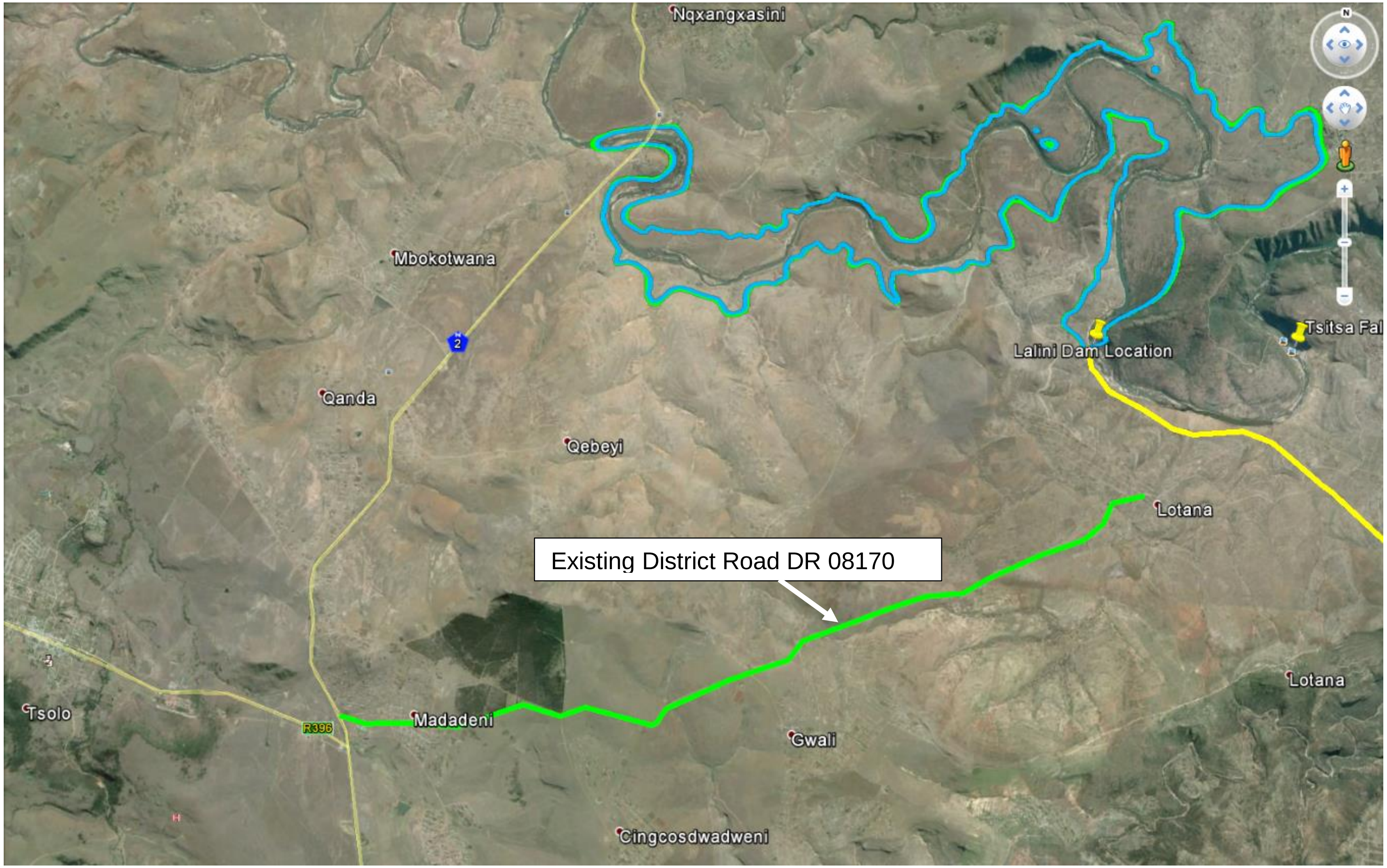


Figure 8-1: Main Access Road to Infrastructure Construction Locations

Figure 8-2: Main Access Road: Section Close to N2 Junction

Figure 8-3: Main Access Road: 4 km from N2 Junction

Figure 8-4: Main Access Road: 3 km from Lotana

Figure 8-5: Main Access Road Approaching Lotana

8.1.2 Dam and Pipeline Access Roads

The 4.2 km roads shown in blue will be new roads. These roads will be initially established as gravel haul roads for use by normal construction vehicles. However as this will be the main permanent access route to the Lalini Dam and mini-hydropower plant, the road would be upgraded to a double sealed surface, once main construction activities have ceased. This would require the following layer works.

Table 8-1: Layer Works for Dam and Pipeline Access Roads

	Layer Description
	Double Seal Surface Treatments
	150 mm G2
	200 mm Stabilised Sub-base Course –C4
	150 mm G7
	150 mm G9

8.1.3 Tunnel Entrance Portal Access Road

This 1.3 km road shown in dark green will be a new road to the upper entrance to the tunnel. The road would be constructed as a gravel haul road for use by normal construction vehicles. It will mainly be used during the construction of the tunnel portal section, and during the delivery and installation of the pipeline section within the tunnel. As frequent access to the tunnel in the future would not be required, this could remain a gravel road.
However, as this section of road is relatively short it is recommended that this also be upgraded to a double sealed surface, once main construction activities have ceased, with the same layer works shown in Table 8-1.

8.1.4 Access to the Main HEP and Tunnel Exit Portal

The access road to the main HEP building and outlet portal of the tunnel is the highest priority road. This road has exacting requirements in terms of gradients and load carrying capacity, and yet has to traverse the most difficult terrain on the whole project.

This road will be used as the main construction haul link for the tunnel and HEP building construction. It will also be the route along which the abnormal loads travel when delivering the hydropower electro-mechanical and transformer components, and for servicing and replacement of such plant in the future.

Two options were investigated, and these are shown as HEP Access Road Option 1 (red) and HEP Access Road Option 2 (light green) in Figure 8-6.

An assessment of the estimated number of heavy vehicles that will operate over this section of road was undertaken in order to inform pavement design. During the peak of construction, the number of heavy vehicles was estimated to be approximately 20 per day per lane.

Given the abnormal loads that this road will also have to carry, a sensitivity analysis was undertaken. Various load equivalency factors were used ranging between 1 and 15, with 15 being established from a paper written by M De Beer, I M Sallie, Y van Rensburg and M Kemp, titled “Load Equivalency Factors (LEFs) for Abnormal Vehicles (AVs) and Mobile Cranes in South Africa based on the Mechanistic-Empirical (M-E) Design Methodology”, 2009.

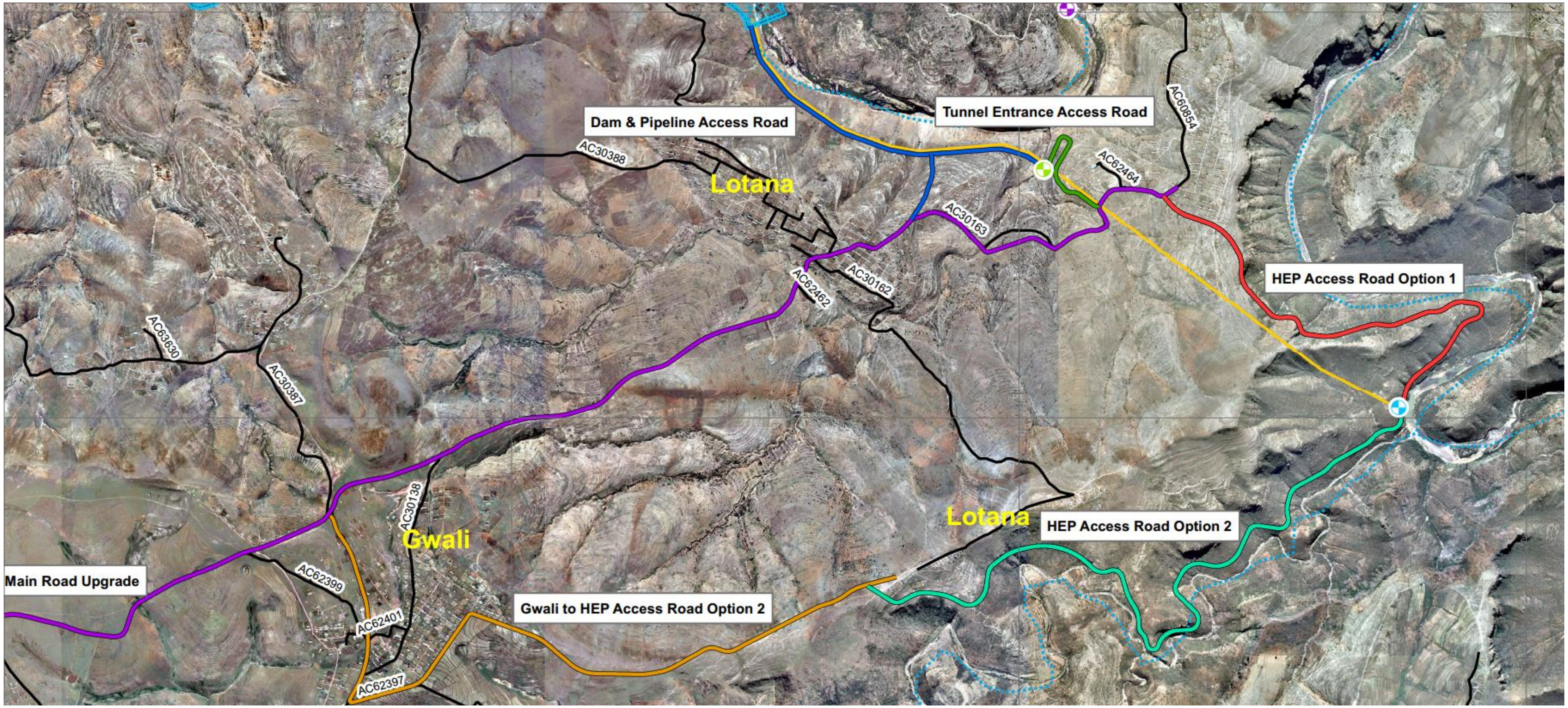


Figure 8-6: Main Access Road and Other Roads to Construction Sites

The resulting pavement class, as described in TRH 4, is an ES 1 pavement. The pavement structure is presented below in Table 8-2.

Table 8-2: Layer Works for HEP Access Road

	Layer Description
	40 Asphalt
	125 mm G2
	150 mm C4
	150 mm G7
	150 mm G9

The above high grade specification and the difficult terrain results in an expensive road, but one which would require only low maintenance inputs in the longer term. Another option would be to design the road as an all-weather gravel road which would be used during construction and then surface layers reprocessed to bring the road back up to full all-weather standards. This would reduce capital costs by approximately 5%, but on-going regular maintenance would be required during the lifetime of the road.

It is therefore recommended that a high specification asphalt road be designed and constructed at the start of the project to provide reliable access to the HEP during construction and operation. For the purposes of this feasibility study, the design is based upon the layer works in Table 8-2.

The abnormal load vehicles that will be required to transport the electro-mechanical plant components would be a multiple axle flatbed horse and trailer arrangement, which have specific requirements in terms of maximum gradients and turning circles.

A typical abnormal load transporter arrangement is shown in Figure 8-7. These requirements determine the geometric standards that must be applied in the road design. These standards are summarized in Table 8-3.

Option 1 as indicated on Figure 8-6 and the layout plans gains access via the upgraded main access road described in 8.1.1 above. The access road follows the spur towards the lower end of the tunnel and traverses the mountain side to the HEP outlet works. The terrain that this road traverses results in large side-cuts and large side-fills that will require substantial retaining walls to support the road layer works. The design of this road therefore needs special attention to ensure that the road has the least impact on and blends into this sensitive environment as far as possible.

The road geometry achieves a maximum of 10% gradient throughout the alignment and provision will have to be made for abnormal vehicle lay-bys so as to achieve safe operations of these vehicles (brake cooling periods etc.). This access road has been planned to cater for abnormal vehicles required to transport the 75 tonne turbine components for the HEP.

Option 1 provides serious challenges in that it requires large cuts and fills to be constructed at significant costs. Therefore **Option 2** was also investigated. **Option 2** follows the valley wall of a south west tributary of the Tsitsa River flowing from Gwali to the HEP location.

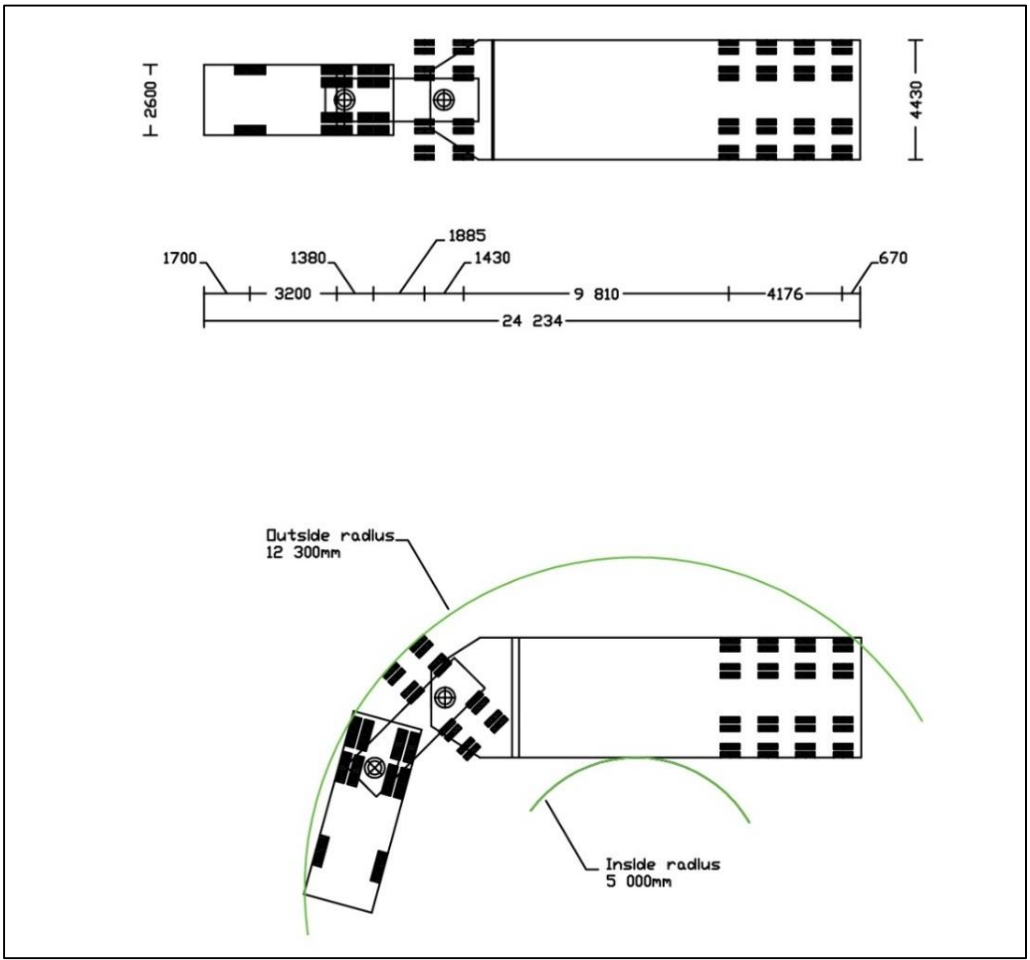


Figure 8-7: Typical Abnormal Vehicle Properties

Table 8-3: Summary of Geometric Standards

GEOMETRIC STANDARDS	
Project Design Speed	20-40 km/hr
Max Length of Horizontal Curve	1000m
Min K Value for Crest Vertical Curve	1
Min K Value for Sag Vertical Curve	3
Min Length of Vertical Curve	60m
Topography	Mountainous
Max Gradient	10%
Min carriageway Width (Surfaced)	7.5m
Normal Camber	2.5%

An indication of typical terrain to be traversed by Option 1 is given in Figures 8-8 and 8-9 and for Option 2 in Figure 8-10.

Figure 8-8: HEP Access Road Option 1: Typical Terrain from Lotana Plateau

Figure 8-9: HEP Access Road Option 1: Typical Terrain along Spur towards HEP

Figure 8-10: Option 2: Typical Terrain from the Gwali Direction

The geometric design criteria for **Option 2** were the same as for **Option 1**, and it was easier to achieve vertical alignment grades ranging between 1.5% and 10%, with the requirement of retaining walls reduced proportionally to that of Option 1.

Whilst this access road provides more suitable operational conditions for the abnormal vehicles, it would be, at 8.1 km long, significantly more expensive to construct than Option 1, which is 5.3 km long.

In addition, Option 2 also requires the upgrading of a further 8.2 km of the existing roads from the main access road at Gwali to the start of the new Option 2 HEP Access Road. Technically Option 2 will be easier to construct, but it will be significantly longer and more expensive, and will also impact a larger area of sensitive vegetation.

8.1.5 Gwali to HEP Option 2 Existing Road Upgrade

This 8.2 km long section of road would need to be upgraded if Option 2 were to be adopted. The geometric standards and layer works would be the same as for the Main Access Road.

At this feasibility design level of study, Option 1 has been adopted as being the preferred option, but it is recommended that further detailed investigation and optimisation of the HEP Access Road route be undertaken at the detailed design stage. This optimisation should take all relevant factors into consideration, such as technical aspects, construction difficulty, cost and permanent impact on the environment.

8.2 Roads and Bridges: Upgrades and Realignment

Other major road works will be required to undertake the realignment of infrastructure that will become inundated once the Lalini Dam has been commissioned. The layouts of these roads are shown on Figure 8-11.

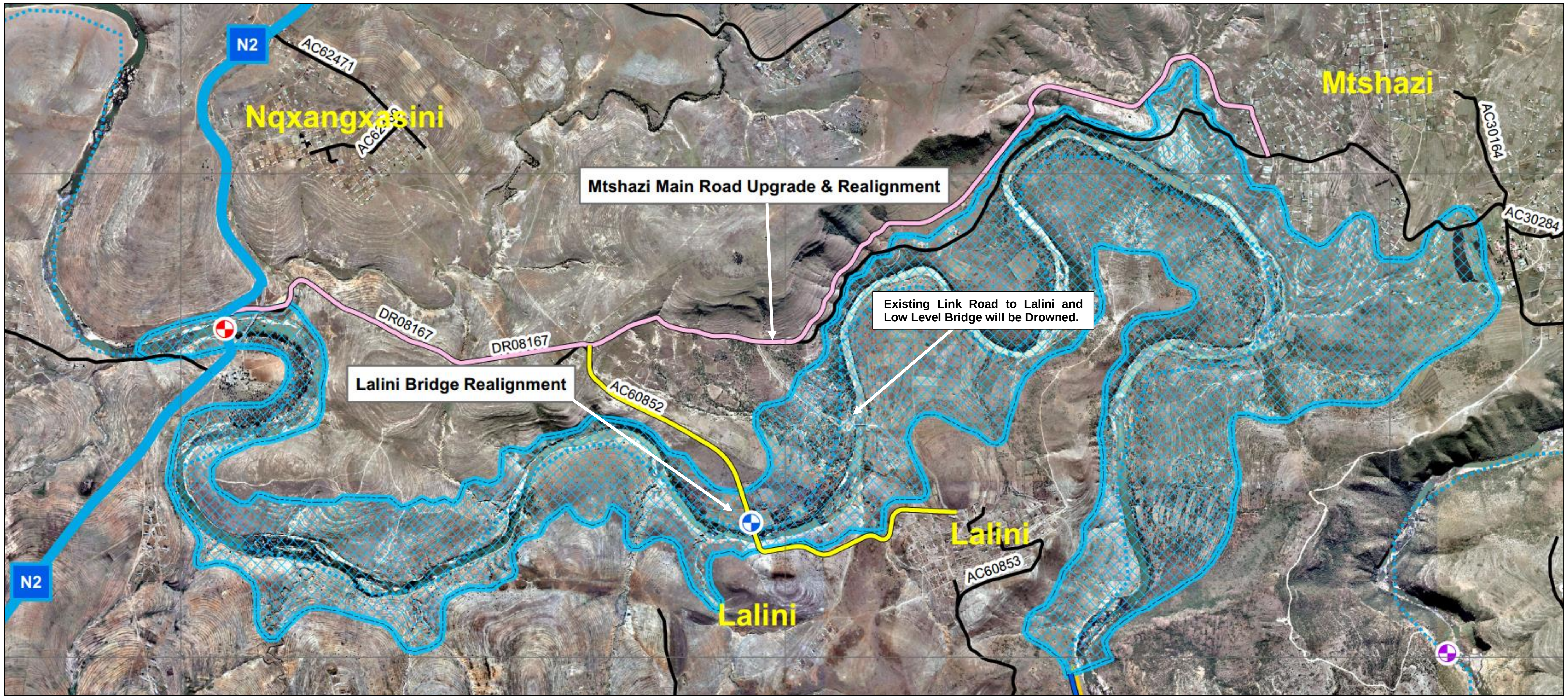


Figure 8-11: Roads and Bridges to be Permanently Upgraded and Realigned Before/During Construction

8.2.1 *Mtshazi Main Road*

The impoundment of Lalini Dam will inundate some existing roads as well as drowning an existing river crossing vehicular bridge. The latter connects the village of Lalini with the settlements of Mtshazi, Shawbury, and the main N2 national road to Qumbu and Mthatha.

District Road DR 08167 shown in pink is a tarred road, is the main access from these villages to the N2, and is also a main tourist route for visitors to the Thina and Tsitsa Falls.

This 10.4 km road is currently in a pot-holed state, and some 40% of the existing route will need to be realigned to ensure that it passes outside of the future inundated area.

The proposed pavement design for district road DR 08167 is presented in Table 8-4. This pavement design should be optimised at the detail design stage when detailed traffic counts will be undertaken, as well as in-situ sampling of the existing road pavements.

Table 8-4: Layer Works for District Road DR 08167

	Layer Description
	Double Seal Surface Treatments
	150mm G2
	200mm Stabilised Sub-base Course –C4
	150mm G7
	150mm G9

A summary of the design criteria used in the road design is provided in Table 8-5.

Table 8-5: Summary of Geometric Standards for DR 08167

GEOMETRIC STANDARDS	
Project Design Speed	60 km/h
Min Horizontal Curve Radius for a max super-elevation of 6%	135 m
Max Length of Horizontal Curve	1 000 m
Min K Value for Crest Vertical Curve	16
Min K Value for Sag Vertical Curve	16
Min Length of Vertical Curve	100 m
Topography	Rolling
Max Gradient	7%
Min carriageway Width (Surfaced)	7.5 m
Normal Camber	2.5%

8.2.2 *Lalini Bridge Relocation*

The existing link road from the above Mtshazi road to Lalini village crosses the Tsitsa River via a low level single track vehicular bridge, which was constructed by SANRAL. This carries both vehicular and pedestrian traffic and is the main route for Lalini residents to travel to Mtshazi, Shawbury and the main N2 national road.

This existing bridge and road will be permanently drowned by the impoundment of Lalini Dam.

Alternative routes were sought to replace this route, which included a new road from Lalini along the south bank of the river and connecting to the N2. Unfortunately this would increase the travelling distance for journeys from Lalini to Mtshazi and Shawbury by 15 km. This would be highly unacceptable for pedestrians which include children going to school. If this option were adopted, then a high level footbridge would also be required to cater for the pedestrian users. This option would however still not be an acceptable solution as far as additional travel distance and time required by the vehicular road users.

The EIA study team were consulted and it was suggested that in such circumstances the solution should follow the principles of a “like-for-like” replacement. In order to meet the SANRAL standards, the bridge deck soffit would be required to be at an elevation providing 1.4 m freeboard above the 1 in 100 year flood level. This results in a bridge deck length of 450 m.

The alignment of the new link road and bridge is shown in yellow on Figure 8-11.

A general arrangement of the proposed bridge is given in Figure 8-12.

A multi-purpose bridge was therefore designed which has a single track vehicular way and a barrier-protected pedestrian walkway. Given the long length of the bridge, the vehicular carriageway has two widened waiting bays for vehicles to pass each other. The bridge must meet SANRAL design standards.

The 4.4 km new link road connecting the bridge to the existing Mtshazi road and to the existing main road into Lalini, would be designed to the same standards and have the same layer works as for the district road DR 08167 above, and would therefore be a tarred surface road.

8.3 Camps and Permanent Staff Accommodation

Several construction contracts are likely to be awarded to undertake the various components of this project. Depending upon the timing of the various contracts and the approach and methodology of the contractors, the construction of the works will provide construction work opportunities for between 400 and 1 100 people for varying periods[7]. Most of these jobs will be filled with labour commuting or being transported from local communities. This includes the small villages close to the works as well as from the urban areas such as Qumbu, 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.

The estimated operational staff levels of the Lalini Dam and HEP are as given in Table 8-6.

These are considered to be the maximum number required, and these numbers may reduce depending upon who operates the dam and HEP and the calibre of staff assigned to these operations.

[7] *Figures based upon the average employment created during the Lesotho Metolong Dam Project construction period.*

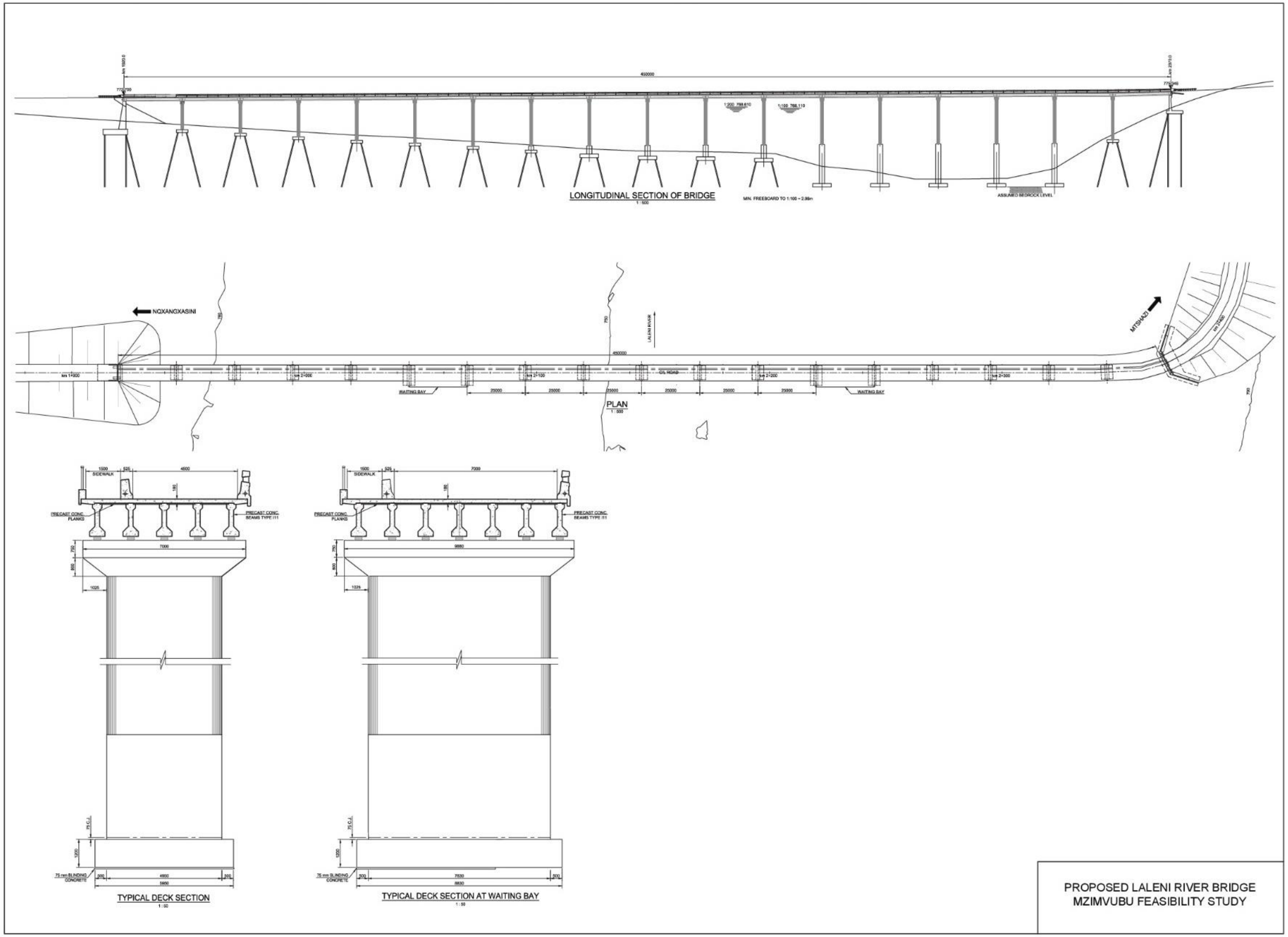


Figure 8-12: Proposed Lalini Bridge over Inundated River Section

Table 8-6: Estimated Staff Requirements at Lalini Dam and Hydropower Plant

LALINI DAM				
Position	**Haygrade**[8]	**Day Shift**	**Night Shift**	**Total Shifts/Day**
Senior Water Control Officer	G	1	1	2
General Worker	A	4	2	6
Totals		**5**	**3**	**8**
LALINI HYDROPOWER PLANTS (BOTH)				
Position	**Haygrade**	**Day Shift**	**Night Shift**	**Total Shifts/Day**
Certified Engineer (also covers dam)	L	1		1
Senior Plant Superintendent	J	1		1
Artisan Electrician	H	1	1	2
Artisan Millwright / Fitter & Turner	H	1		1
Artisan Aid	C	4	2	6
Totals		**8**	**3**	**11**

A proposed site infrastructure layout is given on Figure 8-13.

Given the permanent road network that will be established to access all of the Lalini infrastructure components, it is proposed that a staff housing estate is constructed as shown at a suitable location within short commuting distance to both the dam and HEP.

Allowance will also be made for a guest house to accommodate official visitors such as head office management, and the occasional VIP.

Provision has therefore been made for a housing estate containing some 16 stands on which one-, two- and three-bedroomed staff houses can be built. These will also have fitted kitchens, bathrooms, lounge and dining rooms, and will have mains electricity, water, and waterborne sanitation. If more housing is eventually required, there is sufficient land available for this purpose within the boundary shown.

Allowance has been made in the project budget for construction of 4 x 1-bedroom, 10 x 2-bedroom, and 2 x 3-bedroom houses. These requirements can be reviewed during the design stage.

Electricity will be via the ESKOM connection to the project site described in section 8.4. Water supply will be from a small package plant drawing from the river downstream of the dam, using the proposed new flow gauging station as an abstraction weir. A wastewater treatment facility will also be built, with its discharge of treated effluent either directly to the river or via a tributary which flows into the river. The housing complex will also have street lighting, tarred roads and surface water drainage.

[8] *The Hay system of job evaluation is a point factor method of job evaluation that measures three factors common to all jobs – know-how, problem solving and accountability. The classification system focuses on internal job relationships and maintaining internal equity.*

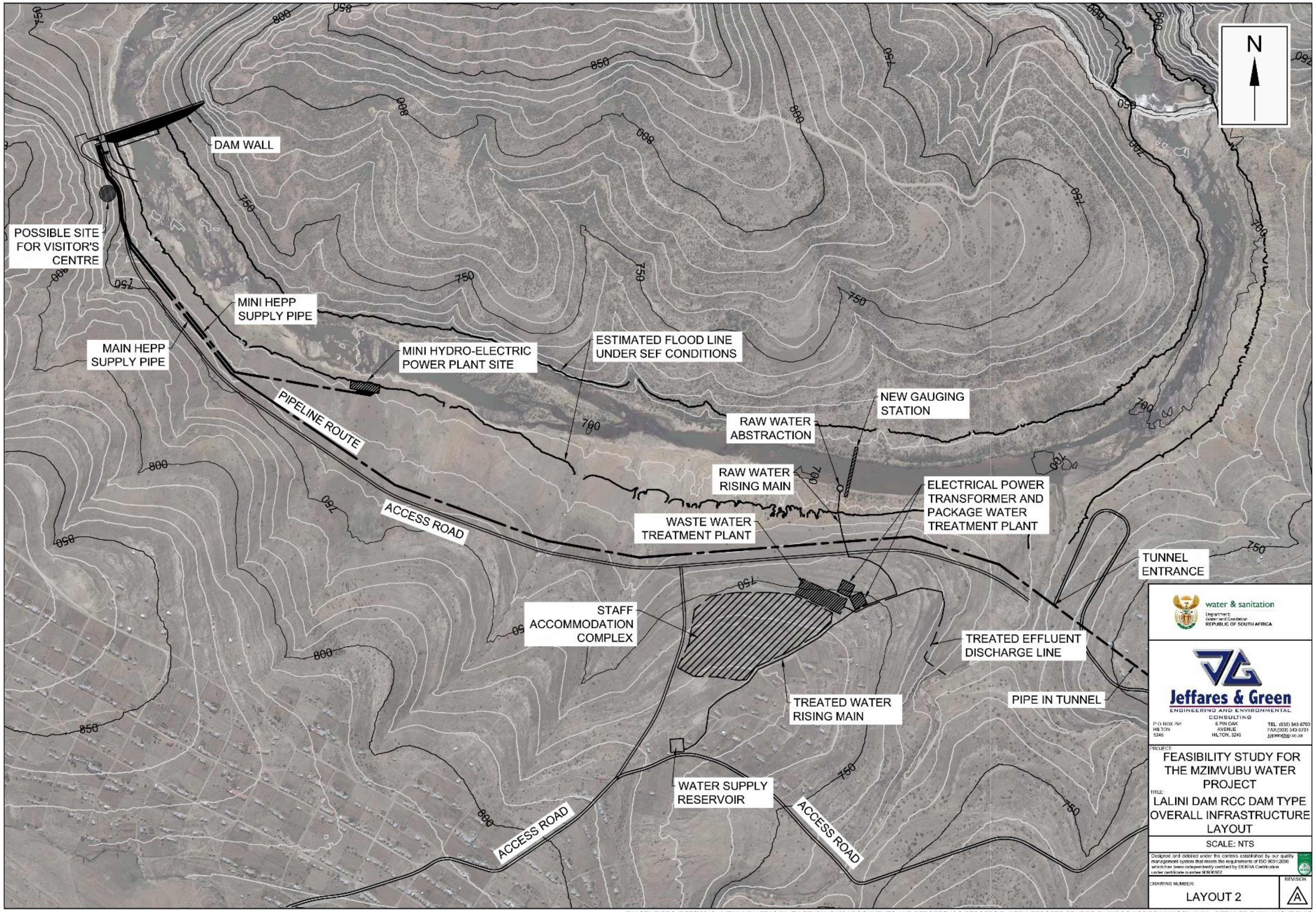


Figure 8-13: Proposed Lalini Site Infrastructure Layout

8.4 Power Supplies and Grid Connections

Table 8-7 summarizes the expected power load requirements during the construction and operation of the scheme as well as the grid access connection capacities required to deliver the generated hydropower into the local grid system

The connections required for loads 1 and 2 would be used both for the works construction and longer term to operate the works. This would also include the supply of power to the housing estate, offices, water supply and wastewater treatment plant.

Discussions are underway with ESKOM to confirm these requirements and to also agree how and to which part of their grid energy generated by the HEP's would be evacuated. This will confirm final power line capacities, routes and budgets required.

These discussions have resulted in suggestions from ESKOM that the main grid connection to the Lalini scheme would be via a 132 kV line to the existing 132 kV grid system. This is as indicated on Figure 8-14.

This line should be constructed to ESKOM's approved standards as advance works for the project, rather than ESKOM themselves undertaking the construction. The reason for this is that the construction power supply is required to be in place before any construction can start and ESKOM stated that they would need up to three years to implement if they were tasked with this component of the scheme.

This 132 kV line would therefore initially provide a power supply to the Lalini scheme, but would later be switched and synchronized so that the net surplus power generated by the Lalini HEPs could be fed back into the national grid to facilitate revenue generation.

Within the Lalini scheme itself, a further 22 kV power line will need to be constructed from the Lalini main HEP transformer/switching compound to provide power to the dam, tunnel and infrastructure works, which later can be used to evacuate the surplus power generated at the Lalini mini-HEP back into the national grid. This 22 kV line should also be expediently constructed under the advance works rather than be assigned to ESKOM to implement.

The proposed alignments of the 132 kV and 22 kV lines are as indicated in Figure 8-14, and these maximize the usage of existing and proposed road corridors which can serve as joint servitudes, thus minimizing the land requirements. These alignments must be optimized during the detailed design stage.

An amendment to the environmental authorisation or a new EIA will be required if these routes need to be revised from those included in the EIA study.

Table 8-7: Power Requirements for Scheme

						Load Locations	
Ref. No.	**Use description**	**Eskom infrastructure required from:**	**Capacity**	**Required for construction**	**Required for permanent use**	**Latitude**	**Longitude**
New Loads Required on ESKOM grid							
1	Power supply for Lalini tunnel and HEP	Year 2018	5 MW	Yes	Yes*	31°17'53.54"S	28°59'10.76"E
2	Power supply for Lalini dam and associated works	Year 2018	10 MW	Yes	Yes*	31°15'54.61"S	28°55'05.82"E
Hydropower Plants to Feed into ESKOM grid						**HEP Plant Locations**	
3	Lalini mini-hydropower plant	Year 2021	Seasonal output of 1 MW to 5 MW	No	Yes	31°15'58.25"S	28°55'08.37"E
4	Lalini Main Hydro Power Plant	Year 2021	Seasonal output of 12.5 MW to 37.5 MW.	No	Yes	31°17'55.04"S	28°59'10.67"E

** Permanent use would be at a much lower power requirement for operations, housing, water supply, wastewater treatment, HEP black-start, lighting, valves, and control systems, etc.*

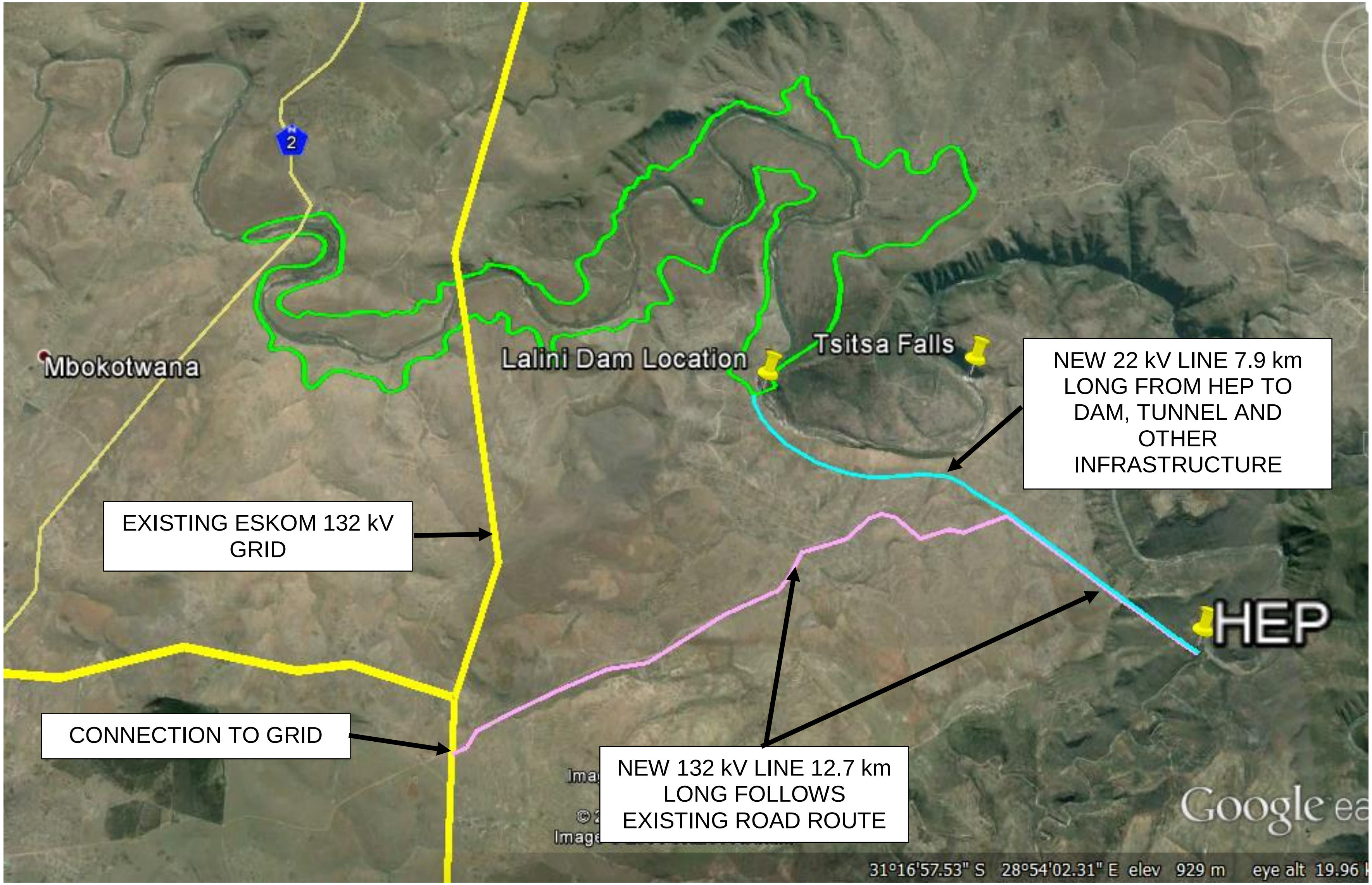


Figure 8-14: Proposed 132 kV and 22 kV Power Line Alignments

8.5 Water Supply

The villages of Lalini and Lotana both have existing water supplies but it is not certain whether these would have sufficient capacity to meet the short and longer-term requirements at the Lalini Dam and staff accommodation complex. Eventually, the Lalini and Lotana villages' water supplies would be an integrated part of the bulk potable water supplies developed under, and supplied from, the proposed Ntabelanga scheme.

For the construction works, a separate water supply should therefore be developed to supply potable water to the client's and engineer's site offices and temporary accommodation during the construction period, and for the permanent accommodation village and administration offices in the longer term. This will typically have a capacity of 150 m^3/day, and it is usual for this facility to be a modular package plant.

It is recommended that this plant not be sized to cater for the dam, tunnel, and other works construction as this would normally be the contractor's responsibility.

The plant is located such that water is pumped from a river intake just upstream of the proposed new gauging weir (see Figure 8-13) to the treatment plant adjacent to the accommodation village. The treated water is lifted into an elevated storage tank (24 hours storage) serving the accommodation site by a gravity reticulation system. These elevated tanks will later be used as the permanent treated water storage supplying the operations buildings and housing, and their 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. The location of the operations/administration centre could be within the accommodation village or it could be located as a combined facility at the visitor's/information centre nearer the dam. If the latter is decided during the detailed design stage, then a water supply system, electricity, telecoms, and wastewater treatment for that centre will need to be provided in addition to those facilities at the accommodation village.

Consideration could possibly be made to size these works such that the water supply requirements to the whole of Lotana could be met, and it is recommended that this aspect be investigated at detailed design stage. This will also depend upon the plans and timing of the District Municipality to supply this area with water from the main Ntabelanga scheme, and its scheduled completion date.

8.6 Wastewater Treatment Plant

A wastewater treatment plant will be required to treat effluents produced by the Lalini Dam operations centre and housing complex. This would 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, before discharging treated effluents back to the river to approved quality standards.

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.

At the main HEP site, the ablution facilities could discharge to a septic tank system as usage will be of low volume.

8.7 Telecommunications

Whilst the cellular network in the region has reasonably good coverage, adequate communication systems will need to be assured 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.

8.8 Visitor's Information Centre

The Lalini Dam and its body of water, and the hydropower plants, 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 information centre can form the focus of such an initiative by providing visitors with a view of the works and information on the project, including the cultural and tourism activities in the area. A location for this centre is suggested above on Figure 8-13. It is recommended that such a building be of interesting architecture in keeping with the local culture and terrain.

Consideration could also be made to combine this building for both visitors and as the administration and operations centre. If this building could be completed early enough as a part of the advance infrastructure, then it could be used as the Client and Resident Engineers offices during construction, as was the case at Katse Dam.

8.9 Compensation and Mitigation Works

The EIA PSP has identified other mitigations, offsets, and compensation works that could require engineering inputs and construction activities.

These include, *inter alia*,

- relocation of homesteads affected by the scheme;
- lost livelihood compensation;
- a water and sanitation health (WASH) awareness programme;
- land acquisition and offsets;
- wetland offsets;
- flora and fauna relocation and rescue;
- fish/eel ladders; and
- other mitigations, such as the improvement of schools, clinics, police stations.

Preliminary budgets have been provided in the cost estimates for these other potential works, the final requirements and implementation of which should be further considered in the detailed design stage.

9. COST ESTIMATE

9.1 Capital Cost

The cost estimate for the Lalini Dam and its associated infrastructure, including the two hydropower plants and associated infrastructure, is given in Table 9-1.

Full details of these cost build-ups, cashflow projections and escalation calculations are given in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15.

Table 9-1: Lalini Capital Cost Estimates

Main HEP Installed Capacity Option:>	**37.5 MW**	**50 MW**	**150 MW**
Component	**Capital Cost R'million**		
Lalini Dam (0.28 x MAR Capacity)	601.64	601.64	601.64
Associated Works	127.01	127.01	127.01
Mini-Hydropower Plant (5 MW)			
Building Structure incl O/H Crane	11.55	11.55	11.55
Turbines & Generators Electro-Mech	37.00	37.00	37.00
Transformer Station	2.00	2.00	2.00
Power lines (22 kV) to Grid (say 8 km)	6.00	6.00	6.00
Access Roads			
Lalini Main Road Upgrade	52.31	52.31	52.31
Tunnel Entrance Access Road	11.20	11.20	11.20
Dam & Pipeline Access Road	15.43	15.43	15.43
HEP Access Road Option 1	173.02	173.02	173.02
Roads and Bridges Realignments			
Mtshazi Main Road Upgrade & Realignment	87.36	87.36	87.36
Lalini Bridge Realignment	103.70	103.70	103.70
Hydropower Water Delivery Conduit	2 500 mm dia.	3 000 mm dia.	4 500 mm dia.
Longer tunnel option	687.07	860.88	1 320.68
Main Hydropower Plant			
Building Structure incl O/H Crane	28.80	38.40	42.24
Turbines & Generators Electro-Mech	119.59	163.27	907.50
Switching and Transformer Station	3.00	5.00	incl
Earthworks	7.50	10.00	10.00
Power Lines to Grid 12.7 km (132 kV)	17.50	17.50	17.50
Sub-Total Cost Estimates	***2 091.69***	***2 323.28***	***3 526.14***
Contingencies (10%)	*209.17*	*232.33*	*352.61*
Engineering and EIA Mitigations (12%)	*276.10*	*306.67*	*465.45*
Escalation (averages 18%)	*463.85*	*515.21*	*781.96*
VAT (14%)	*425.71*	*472.85*	*717.66*
Grand Total (R'million)	**3 466.53**	**3 850.34**	**5 843.83**

Costs have been presented for the two base load options described above as well as the peaking station option. As described in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15, the levelized cost of power produced by the two base load options are identical, and the cost of power produced by the peaking option is very much higher, but it also has significant increased cost and environmental impact implications.

The recommended scheme is therefore the 37.5 MW base load installation.

9.2 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. More details are given in the Cost Estimates and Economic Analysis Report No. P WMA 12/T30/00/5212/15.

Maintenance costs per annum are based upon the percentages of capital cost recommended in the DWS Water Supply Planning and Design Guidelines.

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

The following are estimates of these annual operating and maintenance costs, but these should be treated with caution pending decisions being made on the eventual institutional arrangements:

Operation and Maintenance Costs:	R20.83 million/a
Staffing costs:	R 6.80 million/a
Power costs:	R 3.00 million/a

These costs are taken into account in the financing options detailed in the Legal, Institutional and Financing Arrangements Report No. P WMA 12/T30/00/5212/16.

9.3 Project Implementation Programming

The implementation of the Lalini hydropower scheme is a key component of the conjunctive scheme which generates significant revenue to subsidise all of the power costs and more on the Ntabelanga water supply and irrigation component of the conjunctive scheme. This brings the unit cost of water produced down to a viable and sustainable level.

In order that these benefits are realized timeously, it is recommended that this component be implemented simultaneously with the Ntabelanga components so that there is no lag in the revenue stream that produces such cross-subsidization. Budget and cash flow constraints may however require that the implementation programme for the project be extended.

A draft implementation programme is included in Appendix C. This is under review by the DWS and will be regularly updated.

9.3.1 Priority Infrastructure

The following are considered to be associated works components that should be constructed as a priority, and should therefore be part of an advance infrastructure contract which is completed before the main works construction commences:

- Main access roads, especially those to the dam, and to the tunnel exit portal and main HEP plant;
- Power supplies for construction; and
- Telecommunications.

Additional optional components are:

- Staff accommodation, if it is to be used by DWS and engineer's team during construction, but do not allow contractor to use;
- Temporary water supply and wastewater treatment works, if staff accommodation is built, and
- Visitor's information centre and admin/operations centre, which could be DWS and engineer's site offices.

Most of the above works will require an environmental authorization, and are therefore included in the EIA authorization process.

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 undertaking this programme, which has commenced well ahead of the commissioning of the Ntabelanga and Lalini dams.

10. REFERENCES

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2. Van den Ber, 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.

APPENDIX A

LALINI DAM: DESIGN FLOOD NOTE

Pietermaritzburg

6 Pin Oak Avenue
Hilton
3201

PO Box 794
Hilton
3245
South Africa

Telephone
+27 (0)33 343 6700

Facsimile
+27 (0)33 343 6701

E-mail
jgipmb@jgi.co.za

Web
www.jgi.co.za

Jeffares & Green
ENGINEERING & ENVIRONMENTAL CONSULTING

9 September 2014

Department of Water and Sanitation (DWS)
Directorate: Integrated Water Resources Planning

Your Ref.:Mzimvubu
Our Ref.: 2819

ATTENTION: MR MENARD MUGUMO

Dear Sir,

SELECTION OF DESIGN FLOOD FIGURES FOR THE LALINI DAM

As part of the Mzimvubu Water Project, the Lalini Dam site was selected as the preferred site at which to construct a storage structure for the purpose of hydropower production. As part of the dam design process it is necessary to estimate the appropriate design flood values in accordance with the Dam Safety Guidelines for South Africa published by the South African National Committee on Large Dams (SANCOLD) in terms of Dam Safety in Relation to Floods (1991). This correspondence serves as a design note and presents the design requirements based on the Dam Safety Guidelines and the methodologies used to determine the peak discharge values corresponding to the Recommended Design Flood (RDF), the Safety Evaluation Flood (SEF) and other design floods for intermediate return periods.

The proposed Lalini Dam is located approximately 37 km north-east of Mthatha and approximately 13 km south-south-east of Qumbu in the Eastern Cape Province of South Africa. The dam is situated on the Tsitsa River, which in turn is fed by the Inxu, Mooi and Pot Rivers. Four streamflow gauges, located in the area of the proposed dam, have been used in this study. These gauges included:

- Gauge T3H005 located at the outlet of Quaternary Catchment T34H on the Tina River,
- Gauge T3H006 located at the outlet of Quaternary T35K on the Tsitsa River,
- Gauge T3H007 located at the outlet of Quaternary Catchment T33G on the Mzimvubu River, and
- Gauge T3H009 located at the outlet of Quaternary Catchment T35C on the Mooi River at Maclear.

The catchment specific characteristics (related to the design flood calculations) of the proposed Lalini Dam includes a contributing catchment area of 4 422 km^2, a length of longest water course of 187.05 km, an average channel slope of 0.0053 m/m and a distance of 117.22 km to the catchment centroid. Based on the length of longest water course and average catchment slope, a time of concentration of 28.01 hours was calculated using the US Bureau of Reclamation equation (1965).

Design rainfall, required as input into deterministic methods of peak discharge estimation, was determined using the following methodology:

- A catchment centroid for the whole Lalini Dam catchment area was identified, and 5 625 1'×1' gridded design rainfall points were abstracted around the catchment centroid and averaged for each design rainfall return period and duration combination, i.e. all design rainfall at each 1'x1' grid point selected was averaged.
- Design rainfall for each of the above mentioned points was extracted using the Design Rainfall Utility developed by Smithers and Schulze (2003).
- This method was selected as it provides the most comprehensive coverage of the catchment area and has been found to give consistent and reliable estimates of design rainfall in South Africa (Smithers and Schulze, 2003; Gericke and du Plessis, 2011).

By way of example, **Table 1** presents the calculated design rainfall depths for the durations applicable to this study, i.e. those used in the Rational and Unit Hydrograph methods.

Table 1 Design Rainfall Results Comparison for the Lalini Dam Catchment

LALINI DAM CATCHMENT - DESIGN RAINFALL DEPTHS (mm)								
Duration (d)	**Duration (min)**	**1:2**	**1:5**	**1:10**	**1:20**	**1:50**	**1:100**	**1:200**
Averaged 5 625 1'×1' Gridded Points								
9	540	42.2	57.1	67.8	78.8	94.2	106.8	120.2
17	1 020	49.5	66.9	79.4	92.3	110.4	125.1	140.8
28	1 680	61.0	82.5	98.1	114.2	136.9	155.3	175.1
31	1 860	64.6	87.4	103.9	120.9	144.9	164.4	185.4
34	2 040	65.9	89.2	106.0	123.5	147.9	167.8	189.2
43	2 580	70.0	94.7	112.5	131.0	157.0	178.1	200.8

The Guidelines on Dam Safety in Relation to Floods (SANCOLD; 1991) were used to determine the return period requirements for the RDF and the SEF. Although a formal classification process has not been undertaken with the Dam Safety Office of DWS, at this stage of the project it is to be expected that, based on the Dam Safety Guidelines, the Lalini Dam will be classed as a Category III Dam. The RDF and SEF are therefore equal to the 1:200 year design flood event and the Regional Maximum Flood (RMF) plus a K-Factor category, determined from Kovacs (1988), respectively.

Methods Used

A variety of methods to estimate the design flood values for this study were used as per the methods outlined in “Department of Water Affairs Flood Frequency Estimation Methods” (Van Der Spuy and Rademeyer, 2012), with the results being validated using the Utility Program for Drainage (UPD), wherever possible. These methods were as follows:

- Statistical Methods
  - Probability Distribution Fitting to Observed Streamflow Data (Using data from streamflow gauges T3H005, T3H006, T3H007 and T3H009)
  - Regional Flood Frequency Analysis (RFFA) based on two different methods, namely:
    - Haile (2011) and
    - JPV (Görgens, 2007)
- Deterministic Methods
  - Synthetic Unit Hydrograph (SUH)
  - Rational Method
- Empirical Methods
  - Catchment Parameter Method (CAPA)
  - HRU 1/71
  - Midgely and Pitman Method (MIPI)
  - Regional Maximum Flood (TR 137)

The design peak discharge results obtained using the above methods are summarised in **Table 2**.

Table 2 Peak Discharge Calculations Results for the Lalini Dam

Return Period	1:2	1:5	1:10	1:20	1:50	1:100	1:200	RMF	SEF
Unit Hydrograph (28 hr)	462	762	979	1 266	1 644	2 009	2 459		
Rational Method	512	759	1 059	1 269	1 870	2 543	2 867		
MIPI			1 141	1 547	2 147	2 708	3 800		
HRU 1/71	377	942	1 390	1 861	2 614	3 297	3 817*		
CAPA		936	1 323	1 801	2 576	3 170	3 708*		
Statistical Analysis			966	1 224	1 574	1 845	2 134		
RMF	1 546	1 952	2 328	2 776	3 493	4 211	4 969	7 116	8 705
Haile RFFA	163	330	470	627	864	1 071	1 300		
JPV (Veld Zones, GEV)	273	694	1 023	1 384	1 927	2 397	2 928		
JPV (RMF K Regions, GEV)	470	1 316	1 971	2 683	3 741	4 650	5 668		

NOTE: * Denotes extrapolated results

Recommended Design Flood

As presented in **Table 2**, the 1:200 year peak discharge values range from 1 300 m^3/s (using Haile's RFFA Method) to 6 668 m^3/s (based on the JPV [RMF K Regions, GEV] method). The peak discharge values obtained using gauged streamflow data (Index Flood method) are significantly lower than expected (Jeffares & Green, 2013). Typically, regionalised statistical methods are the preferred means in determining peak discharge values used for design purposes. However, it was found that the gauged streamflow data from which the Annual Maximum Series (AMS) was extracted, contained significant levels of missing data as well as instances where flood events exceeded gauge rating tables. It is postulated that due to the level of missing and/or capped AMS data, the resultant probability distributions provide low estimates of design peak discharge values, and therefore are viewed with caution in this study. The JPV method with Veld Zone and RMF K region regionalisation with the GEV distribution are included in **Table 2**. Smithers *et al.* (2014) have found the JPV method with Veld Zone regionalisation and the GEV distribution was the best regional flood frequency for for KwaZulu-Natal.

The difference in the 1:200 year peak discharge results obtained from deterministic methods, namely the Unit Hydrograph and Rational Methods, were found to be notable. This is, however, to be expected due to the fact that the Lalini Dam contributing catchment area is bigger than that for which the Rational Method was developed (normally used for catchment areas of less than 15 km^2). Slight changes in the input variables such as the areal reduction factor or the catchment C Factor can result in significant changes in the resultant peak discharge value. The Unit Hydrograph Method, on the other hand, was developed for catchment areas ranging between 15 and 5 000 km^2. The results of this method were found to be less conservative than those obtained from the Rational Method, but larger than the Statistical Analysis Results, which were considered to be under-estimating the peak discharge at the site.

The 1:200 year peak discharge results obtained from the empirical methods range from 3 708 m^3/s (CAPA Method) to 4 969 m^3/s (RMF Method). The RMF results presented in **Table 2** are considered conservative due to the fact that they are developed using an upper envelope flood method. Unfortunately, calculated 1:200 year return period peak discharges were not available using the CAPA and HRU 1/71 methods, due to the fact that the K_t (HRU 1/71 method) and K_p (CAPA method) values are only available for return periods up to the 1:100 year return period flood event. However, by using a Natural Log-based regression to extrapolate the 1:200 year return period flood peaks for these two methods (individual extrapolation relationships for each method), an extrapolation to the 200 year return period event was made. This comparison showed that the MIPI, CAPA and HR1/71 methods all produce very similar results, about the 3 800 m^3/s mark.

Based on the results obtained from all of the methods considered, it is proposed that a RDF peak discharge value of **3 500 m^3/s** is recommended for the proposed Lalini Dam study. This RDF value is less than the 1:200 year return period peak discharge value estimated using the RMF method (upper envelope method). However, it is greater than the estimated 1:200 year return period peak discharge values obtained using deterministic and RFFA (JPV-GEV-Veld Zones) methods.

The results obtained from the Statistical method, based on gauged streamflow data, were not considered in determining the final RDF peak, due to the high level of missing data associated with the streamflow gauges. It was concluded that the missing and capped AMS data from the gauges resulted in low estimated peak discharge values.

Safety Evaluation Flood

The RMF was calculated using the original Francou-Rodier equation (Rodier and Roche, 1984). As per the Dam Safety Guidelines (SANCOLD, 1991) for a Category III Dam, the SEF was determined by adding a K-Factor category to the adopted Kovacs K-Factor (i.e., $RMF_{+\Delta}$), based on the TR 137 Flood Region Map derived by Kovacs (1988). The Lalini Dam catchment does not lie entirely within one Kovacs K-Factor region, thus an area weighting approach was adopted to calculate the RMF. 1 372 km^2 was delineated in a region with a K-Factor of *5.2*, while 3 050 km^2 was delineated in a region with a K-Factor of *5.0*. The RMF for the Lalini Catchment, based on a catchment area of 4 422 km^2, was calculated to be 7 116 m^3/s. The $RMF_{+\Delta}$, based on increased Kovacs K-Factors of *5.4* and *5.2*, respectively, was determined to be 8 705 m^3/s.

Depending on the results of the dam type analysis, if a roller compacted concrete (RCC) dam is selected where overtopping of wall under extreme flood conditions may not cause the dam to fail, the dam safety office may consider using a routed SEF value, or using RMF (**7 116 m^3/s**) as the SEF, rather than increasing it to $RMF_{+\Delta}$. The routed $RMF_{+\Delta}$ value, based on level pool routing of a 247.5 million m^3 (0.3 MAR) impounded area (with a spillway invert level of 766.63 m AMSL), a spillway length of 320 m and a spillway coefficient of discharge of *2.0*, attenuated the peak discharge to **7 027 m^3/s**. The Dam basin morphology above the spillway invert level has large accumulated increases in volume per unit increase in height. Thus, one could expect that a relatively large reduction in flood peak.

Based on discussions with DWS regarding the above results, it is recommended that the SEF for the Lalini Dam be selected as **7 100 m^3/s** due to the link identified between the 1:10 000 year return period flood peak determined by DWS and the RMF presented in this document.

Peak Discharge for River Diversion during Construction

It is proposed that the 1:20 year peak discharge figure be used to size the capacity of the diversion works during construction. Therefore, it was pertinent to go through a selection process for the 1:20 year peak discharge similar to that described previously for the selection of the RDF.

Emphasis was placed on the empirical methods, the Unit Hydrograph Method and JPV (GEV- Veld Zones) for the selection of this design peak discharge. The Rational Method was excluded due to its lack of suitability for a catchment of this size and the use of observed data was discarded due to the high level of missing data associated with the streamflow gauges.

The JPV, MIPI, CAPA and HRU 1/71 methods all produced similar results between 1 470 m^3/s (MIPI) and 1 861 m^3/s (HRU 1/71). Finally, the scaled 1:20 year return period RMF_{20} value was the largest at 2 776 m^3/s. Therefore, it is proposed that a figure of **1 500 m^3/s** be adopted as the 1:20 year peak discharge to deal with during construction and for sizing of the necessary diversion works.

Finally, **Table 3** presents the selected peak discharge values for a range of return periods, should they be required at any stage.

Table 3 Lalini Dam Design Flood Estimation Results

Return Period	1:2	1:5	1:10	1:20	1:50	1:100	1:200	RMF	SEF
Peak Discharge (m^3/s)	350	750	1 100	1 500	2 000	2 700	**3 500**	7 116	**7 100**

Yours sincerely

RYAN GRAY
Associate/ Hydrologist

PROF. JEFF SMITHERS
Senior Engineer

For: **JEFFARES AND GREEN PTY LTD**

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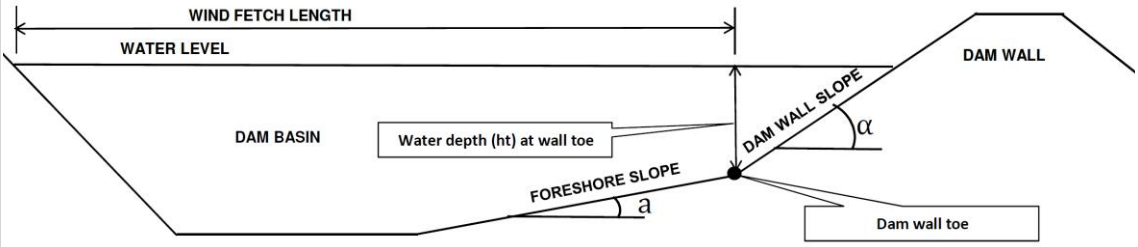
APPENDIX B

LALINI DAM: FREEBOARD CALCULATIONS

Spreadsheet for calculation of the following based on The Rock Manual (2007):

1. Wave height and period resulting from defined wind blowing over a defined fetch of a dam basin.
2. Wave run-up on a defined sloped face of a dam wall due to the wind generated wave calculated under 1.
3. Reduction factors of wave run-up due to: oblique wave attack, wave breaking on shallow foreshore and bermed slope.



1. Wave height and period resulting from defined wind blowing over a defined fetch of a dam basin

	Input: (all blue highligted cells are input cells ; rest are locked)			
	Mean water depth (h) over chosen wind fetch line [m]	20.0		
	Chosen wind fetch, F [m]	4 722		
	Dam wall slope [verical/horizintal e.g. 1/3=0.333 = tanα]	0		
	Return period (1 : x years)	**1:25 yrs**	**1:50 yrs**	**1:100 yrs**
Refer: WRCF § 2.3.1	Mean hourly wind speed U [m/s]	20.0	21.8	23.0

Calculation of Hs and Tp according to (a) Saville, (b) Doneland and (c) Young & Verhagen

1(a)	**Saville method (SMB)**				
	TRM Chapter 4 ; Page 369	g*Hs/U10^2=0.283tanh(0.0125(g*F/U10^2)^0.42)	(4.78)		
	TRM Chapter4 ; Page 369	g*Ts/U10=7.54tanh(0.077(g*F/U10²)^0.25)	(4.79)		
			1:25 yrs	**1:50 yrs**	**1:100 yrs**
	TRM Chapter4 ; Page 369	**Hs** [m]	1.06	1.17	1.25
	TRM Chapter4 ; Page 369	**Tp** [s] - Based on Ts ≈ 0.93Tp	4.09	4.28	4.40
1(b)	**Donelan method**	[*Wind direction assumed = wave direction = maximum straight line fetch.* Thus θ = φw and (θ - φw) = 0].			
	TRM Page 372	g*Hs/(U10*cos(θ-φw))^2=0,00366*(g*F/(U10*cos(θ-φw))^2)0.38	(4.86)		
	TRM Page 373	g*Tp/(U10*cos(θ-φw))=0.542*(g*F/(U10cos(θ-φw))^2)^0.23	(4.87)		
			1:25 yrs	**1:50 yrs**	**1:100 yrs**
		Hs [m]	0.91	1.01	1.08
		Tp [s]	3.30	3.45	3.55
1(c)	**Young and Verhagen method**				
	TRM Page 373	g*Hs/U10^2=0.241(tanhA1*tanh(B1/tanhA1))^0.87	(4.92)		
		A1=0.493(gh/U10²)^0.75			
		B1=0.00313(g*F/U10²)^0.57			
	TRM Page 373	g*Tp/U10=7.519(tanhA2*tanh(B2/tanhA2))^0.37	(4.93)		
	[NB: Erratum in TRM Eq (4.93):	A2=0.331(g*h/U10²)^1.01			
	"U10²" should read "U10"].	B2=0.0005215(g*F/U10²)^0.73			
			1:25 yrs	**1:50 yrs**	**1:100 yrs**
		A1=	0.289	0.254	0.234
		B1=	0.047	0.043	0.040
		A2=	0.161	0.135	0.122
		B2=	0.017	0.015	0.014
		Hs [m]	0.68	0.74	0.78
		Tp [s]	3.37	3.51	3.59

Selected values of Hs and Tp [The values of the formula which gives the maximum wave height is automatically selected]

Hs [m] *(selected from 1(a), 1(b) or 1(c))*	**1.06**	**1.17**	**1.25**
Tp [s] *(corresponding Tp of Hs selected)*	**4.09**	**4.28**	**4.40**

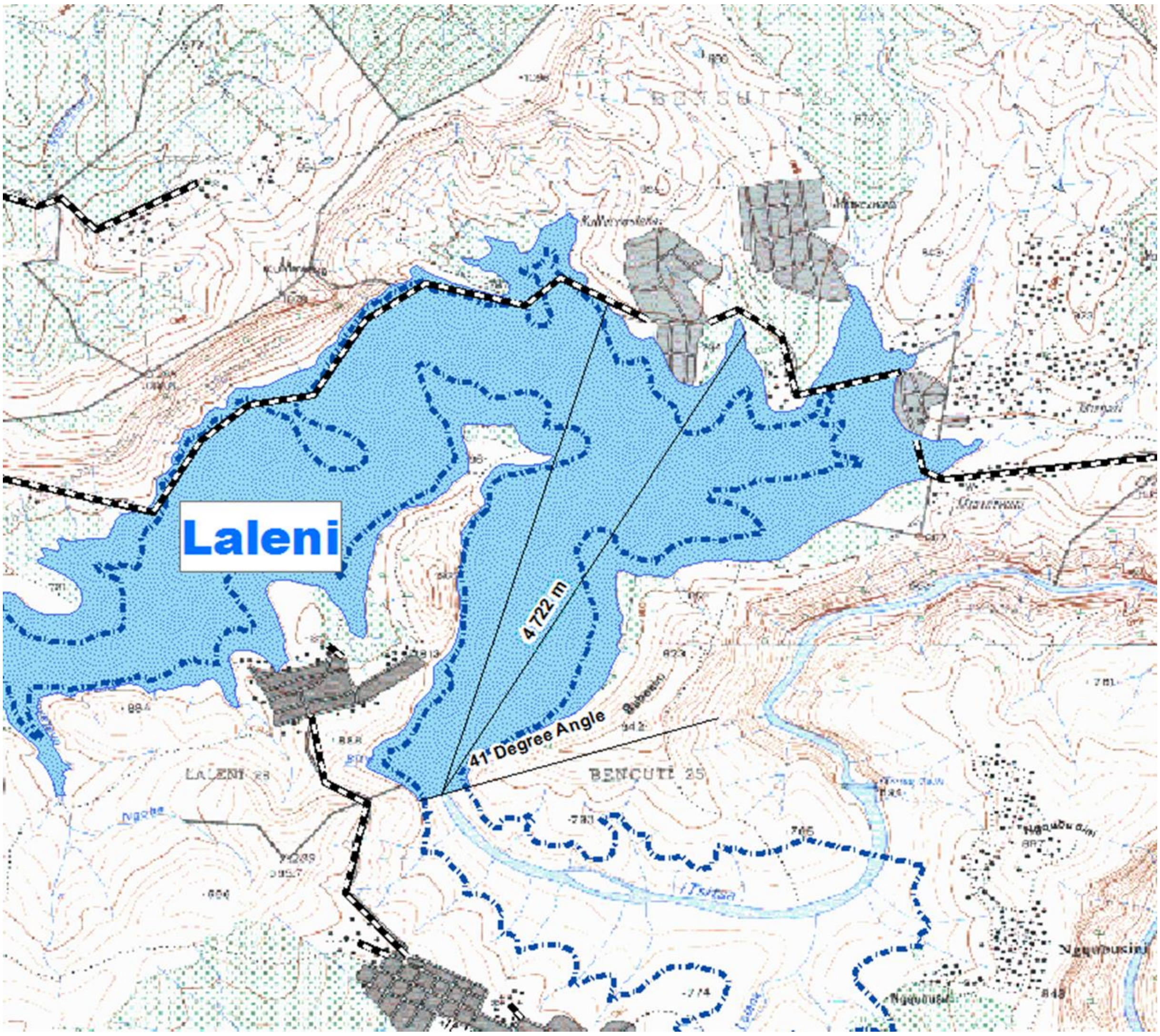
2.	**Calculation of wave run-upon a defined sloped face of a dam wall due to the wind generated wave calculated under 1. for (a) smooth slopes and (b) different types of rough slopes**				
			1:25 yrs	**1:50 yrs**	**1:100 yrs**
	WRCF § 2.5.1	$H_{2\%}/Hs \approx 1.4$			
		H2%=[m]	**1.48**	**1.64**	**1.74**
		$T_{m-1,0} = Tp/1.1$	3.72	3.89	4.00
		Tm = 0.79Tp	**3.23**	**3.38**	**3.47**
	WRCF § 2.6	$\xi = \tan\alpha/(Hs/(1.56T^2))$^0.5			
	TRM Page488	$\xi_p = \tan\alpha/(Hs/(1.56(Tp)^2))$^0.5	0.00	0.00	0.00
		$\xi_{m-1,0} = \tan\alpha/(Hs/(1.56(T_{m-1,0})^2))$^0.5	0.00	0.00	0.00
		$\xi_m = \tan\alpha/(Hs/(1.56(Tm)^2))$^0.5	0.00	0.00	0.00
2(a)	**Run-up for smooth slopes (e.g. concrete and asphalt slab and grass); Ahrens (1981), Allsop (2985), TAW (2002)**				
2(a) i	**Ahrens (1981)**				
	TRM Page492	R2% = Hs(Aξp + B) [m]	(5.7) Eq. 5.8 & 5.9		
		R2% [m]	0.0	0.0	0.0
2(a) ii	**Allsop et al (1985)**				
	TRM Page 492	R2% = Hs(Aξp + B) [m]	(5.7) Eq. 5.8 & 5.9		
		R2% [m]	Out of range	Out of range	Out of range
2(a) iii	**TWA (2002a)**				
	TRM Page 493	R2% = Hs(Aξm-1,0 + B) [m]	(5.7) Eq. 5.8 & 5.9		
		R2% Mean	0.0	0.0	0.0
		R2% Mean + standard deviation	0.0	0.0	0.0
2(b)	**Run-up for rough slopes - reduction factor applied to smooth slope formulae;Ahrens (1981), Allsop (2985), TAW (2002)**				
2(b) i	**Ahrens (1981)**				
	TRM Page492	R2% = γf Hs(Aξp + B) [m] (γf from Table 5.2 TRM Page 494)	(5.7) Eq. 5.8 & 5.9		
		Pitched stone slope	0.00	0.00	0.00
		Armour stone - single layer on impermeable base	0.00	0.00	0.00
		Armour stone - double layer on impermeable base	0.00	0.00	0.00
2(b) ii	**Allsop et al (1985)**		**1:25 yrs**	**1:50 yrs**	**1:100 yrs**
	TRM Page 492	$R_{2\%} = \gamma_f Hs(A\xi_p + B)$ [m]	Eq 5.7		
		$R_{2\%} = \gamma_f Hs(A\xi_p + B)$ [m]	Out of range	Out of range	Out of range
		Pitched stone slope	Out of range	Out of range	Out of range
		Armour stone - single layer on impermeable base	Out of range	Out of range	Out of range
		Armour stone - double layer on impermeable base	Out of range	Out of range	Out of range
2(b) iii	**TWA (2002a)**				
	TRM Page 493	$R_{2\%} = \gamma_f Hs(A\xi_{m-1,0} + B)$ [m] (γf from Table 5.2 TRM Page 494)	Eq. 5.8 & 5.9		
		Pitched stone slope : Mean	0.00	0.00	0.00
		: Mean + standard deviation	0.00	0.00	0.00
		Armour stone - single layer on impermeable base : Mean	0.00	0.00	0.00
		: Mean + standard deviation	0.00	0.00	0.00
		Armour stone - double layer on impermeable base : Mean	0.00	0.00	0.00
		: Mean + standard deviation	0.00	0.00	0.00
2(b) iv	**Rough slopes - explicit formula**				
	V/d Meer & Stam (1992)	Based on *mean* of tests on double layer rock armour on *impermeable & permeable* cores			
	TRM Page 494; Eq. 5.10 & 5.11:	$R_{2\%} = Hs(0.96\xi_m)$ for $\xi_m < 1.5$ and $R2\% = Hs(1.17\xi_m$^0.46) for $\xi_m > 1.5$	0.00	0.00	0.00
3.	**Reduction factors of wave run-up due to: (a) oblique wave attack, (b) wave breaking on shallow foreshore and (c) bermed slope**				
3(a)	**Reduction factor due to oblique wave attack**				
	TRM Page 496 : Eq. 5.13 :	$\gamma_\beta = 1 - 0.0022\lvert\beta\rvert$ for $0^o \le \lvert\beta\rvert \le 80^o$ wave direction measured from direction of normal to slope (i.e. normal direction=0º)			
		Input wave approach direction :	59.0	Degrees [°]	
		Reduction factor due to obique wave attack, γ_β =	0.87		
3(b)	**Reduction factor due wave breaking on *shallow* foreshore (in case where h_t<3*Hs) - refer definition sketch above**				
	Battjes & Groenendijk (2000) :				
		Input foreshore slope (in direction of nearshore wave apporoach direction) **:**	0.000	(Vertical/horizontal=tan a)	
	TRM Page 359 : Box 4.4	**Input water depth at toe (h_t) of wall** (refer definition sketch above):	2.00	m	
	Page 359 ; Eq's 4.58 and 4.59:	$Htr/Hrms = [(0.35+5.8\tan(a))h_t]/[(0.6725+0.2025(Hs/h_t))Hs]$	0.848	0.756	0.704
	Page 360 ; Eq 4.60:	$H_{2\%}$ on shallow foreshore in m = $(H_{2\%}/H_{rms})_{Table\ 4.10}H_{rms}$	1.29	1.44	1.54
	TRM Page 496 : Eq. 5.14:	Reduction factor due to shallow foreshore, $\gamma_h = (H_{2\%}/Hs)/1.4$	0.87	0.88	0.88
3(c)	**Reduction factor due to bermed dam wall slope (i.e. not a straight line wall slope)**				
	TRM Page 497 :	Methodology to determine the reduction of wave run-up due to a bermed slope can be obtained from the Rock Manual p497			

Reference abreviations :

WRCF=WRC Report: Guidelines on Freeboard for Dams (2011)
TAW=Technical Report Wave Run-up and Wave Overtopping at Dikes (2002)
TRM=The Rock Manual (2007)

	Wind Set Up	
Pair	Air density	1.2
Pw	density of water	1000
CD	air/water drag coeffieicent (0.008 to 0.003; assume 0.005 after Kamphuis (2002)	0.005
U 10	mean one hourly wind speed	23
H	Average water depth	20
F	Two times the effective fetch used for wave height computations	9444
g	gravitational Acceloration	9.8
	Wind Set Up Total	**0.076467**

	Seiches and Surges	
	Literature suggests between 0.5 m and 1.0 m for medium to large dams respectively	
	Selected Freeboard for Seiches and Surges	**0.75**



Fetch Diagram

Landslide Factors

H/d	**Dimensionless wave height**		Radius of slide	Propogation direction
d	water depth	20	15	45
Vs	Slide Volume falling into reservoir	5000		
b	slide width	30		
Ps/Pw	Density ratio of the slide material to water	1.8		
#	Impact angle	30	Travel distance	
d/x	Dimensionless travel distaance	0.028571	700	
	Total	**0.135253**		

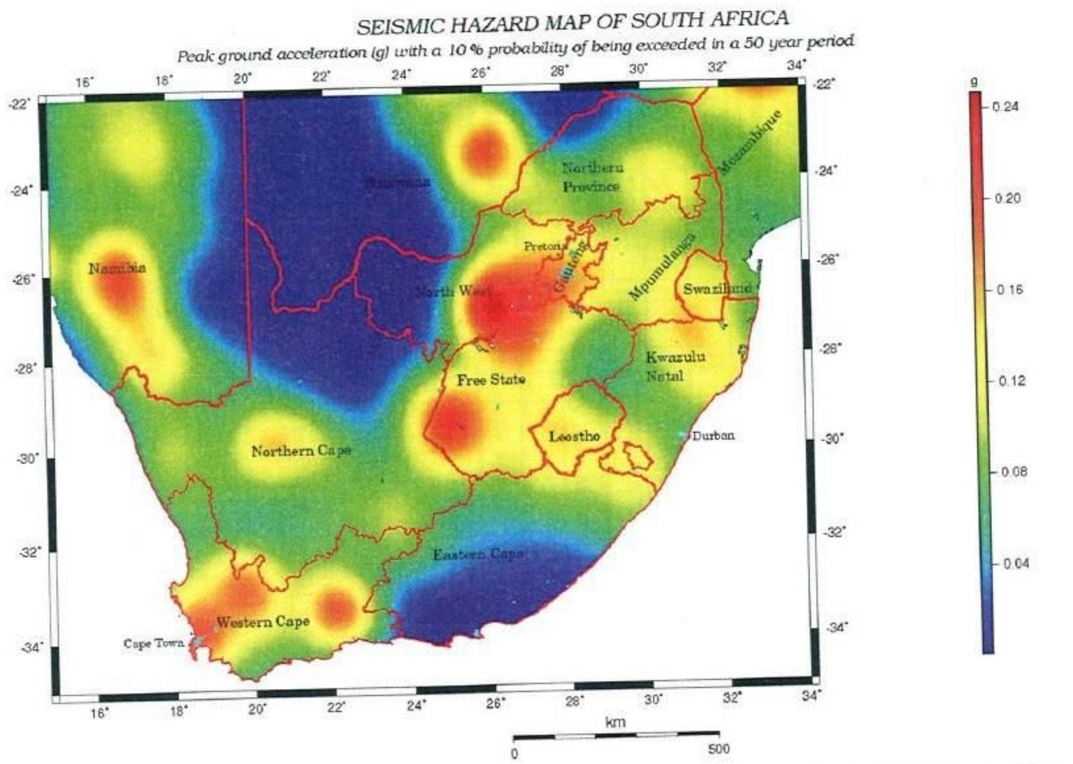


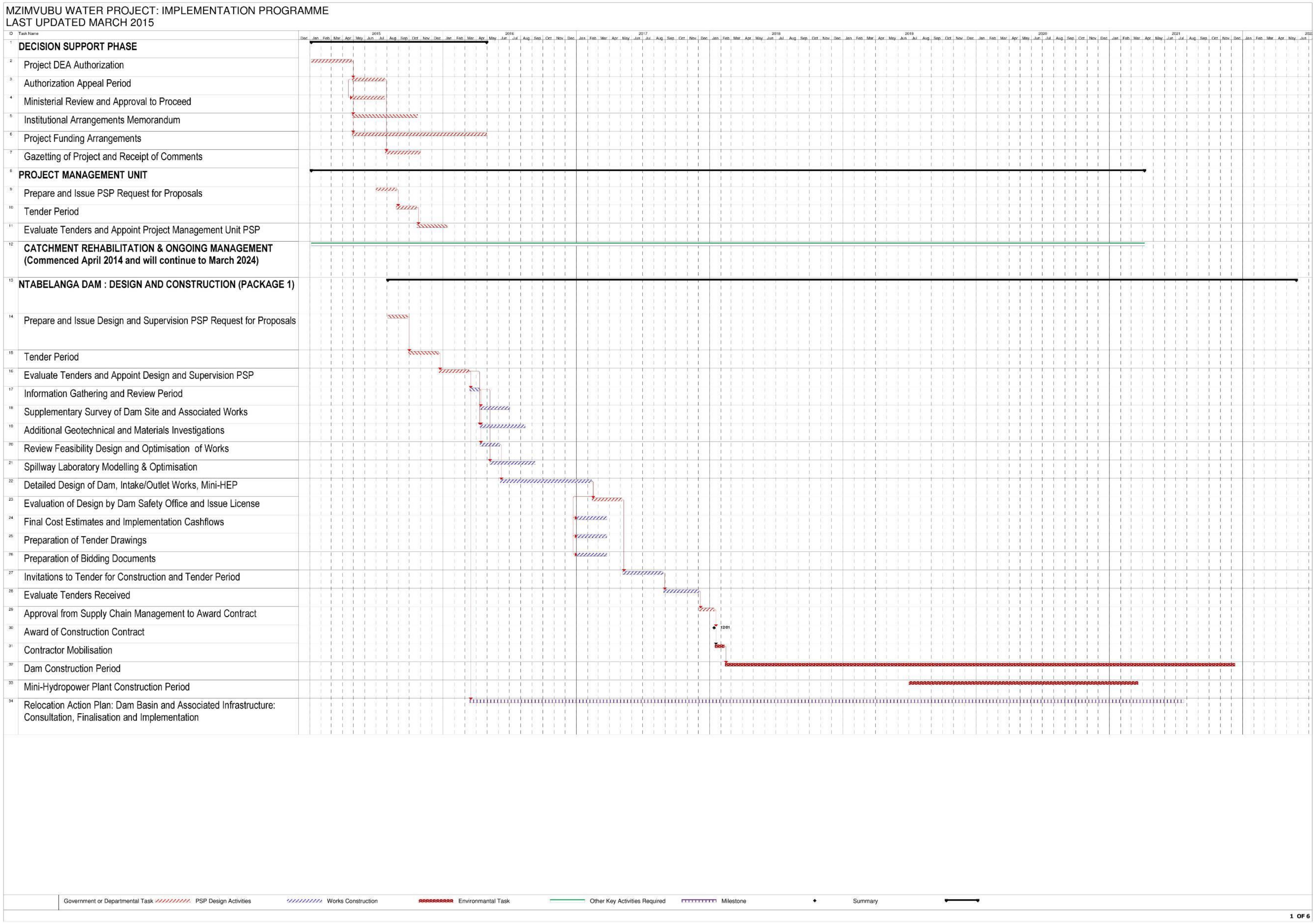
Figure 2.8-1 **Seismic hazard map from Council for Geoscience (2003) data showing peak ground acceleration as a ratio of acceleration) with a 10 % probability of being exceeded in a 1:50 year return period (Kijko *et al.*, 2003)**

SUMMARY ALLOWANCES (m)

Wave Height	0.958
Wind Set Up	0.076
Seiches & Surges	0.750
Earthquakes	0.302
Landslide	0.135
Total	**2.222**

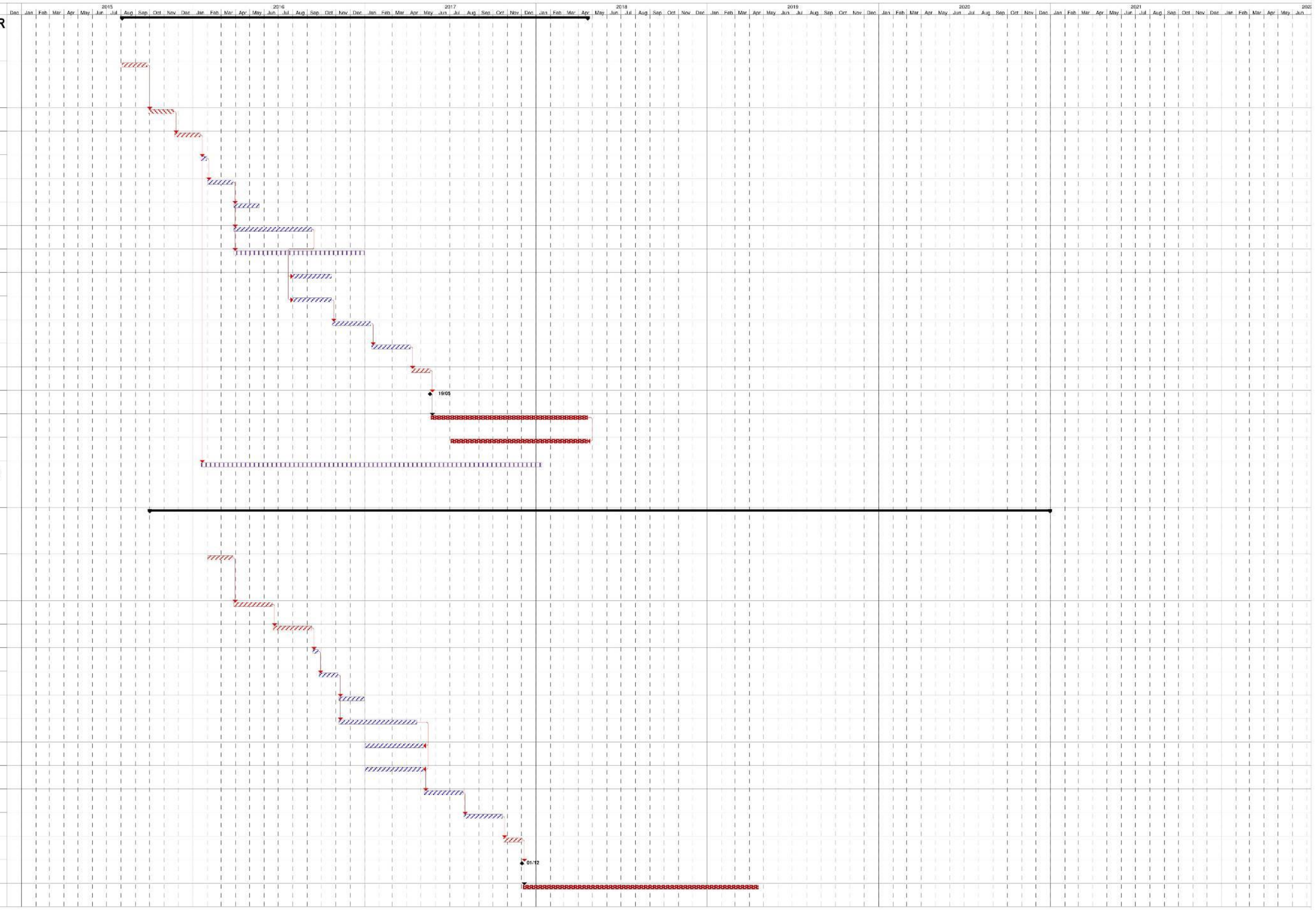
APPENDIX C

DRAFT IMPLEMENTATION PROGRAMME


MZIMVUBU WATER PROJECT: IMPLEMENTATION PROGRAMME
LAST UPDATED MARCH 2015
Task Name
2015
2016
2017
2018
2019
2020
2021
1 DECISION SUPPORT PHASE
2 Project DEA Authorization
3 Authorization Appeal Period
4 Ministerial Review and Approval to Proceed
5 Institutional Arrangements Memorandum
6 Project Funding Arrangements
7 Gazetting of Project and Receipt of Comments
8 PROJECT MANAGEMENT UNIT
9 Prepare and Issue PSP Request for Proposals
10 Tender Period
11 Evaluate Tenders and Appoint Project Management Unit PSP
12 CATCHMENT REHABILITATION & ONGOING MANAGEMENT (Commenced April 2014 and will continue to March 2024)
13 NTABELANGA DAM : DESIGN AND CONSTRUCTION (PACKAGE 1)
14 Prepare and Issue Design and Supervision PSP Request for Proposals
15 Tender Period
16 Evaluate Tenders and Appoint Design and Supervision PSP
17 Information Gathering and Review Period
18 Supplementary Survey of Dam Site and Associated Works
19 Additional Geotechnical and Materials Investigations
20 Review Feasibility Design and Optimisation of Works
21 Spillway Laboratory Modelling & Optimisation
22 Detailed Design of Dam, Intake/Outlet Works, Mini-HEP
23 Evaluation of Design by Dam Safety Office and Issue License
24 Final Cost Estimates and Implementation Cashflows
25 Preparation of Tender Drawings
26 Preparation of Bidding Documents
27 Invitations to Tender for Construction and Tender Period
28 Evaluate Tenders Received
29 Approval from Supply Chain Management to Award Contract
30 Award of Construction Contract
12/01
31 Contractor Mobilisation
32 Dam Construction Period
33 Mini-Hydropower Plant Construction Period
34 Relocation Action Plan: Dam Basin and Associated Infrastructure: Consultation, Finalisation and Implementation
Government or Departmental Task
PSP Design Activities
Works Construction
Environmental Task
Other Key Activities Required
Milestone
Summary
1 OF 6

MZIMVUBU WATER PROJECT: IMPLEMENTATION PROGRAMME
LAST UPDATED MARCH 2015

ID	Task Name
35	**NTABELANGA ADVANCE ACCESS ROADS, BRIDGE AND POWER SUPPLIES (PACKAGE 2)**
36	Prepare and Issue Design and Supervision PSP Request for Proposals
37	Tender Period
38	Evaluate Tenders and Appoint Design and Supervision PSP
39	Information Gathering and Review Period
40	Supplementary Survey of Road and Power Lines
41	Additional Geotechnical and Materials Investigations
42	Detailed Design of Roads, Bridges and Power lines
43	Application to ESKOM for Power Supply
44	Preparation of Tender Drawings
45	Preparation of Bidding Documents
46	Invitations to Tender for Construction and Tender Period
47	Evaluate Tenders Received
48	Approval from Supply Chain Management to Award Contract
49	Award of Construction Contract
50	Construction of Access Roads and Bridge
51	Construction of Power Lines
52	Relocation Action Plan: Roads and Power Line Routes: Consultation, Finalisation and Implementation
53	**NTABELANGA OPERATIONS ACCOMMODATION VILLAGE AND VISITORS CENTRE (PACKAGE 3)**
54	Prepare and Issue Design and Supervision PSP Request for Proposals
55	Tender Period
56	Evaluate Tenders and Appoint Design and Supervision PSP
57	Information Gathering and Review Period
58	Supplementary Survey of Works Areas
59	Detailed geotechnical and materials investigations
60	Detailed design of works
61	Preparation of Tender Drawings
62	Preparation of Bidding Documents
63	Invitations to Tender for Construction and Tender Period
64	Evaluate Tenders Received
65	Approval from Supply Chain Management to Award Contract
66	Award of Construction Contract
67	Construction of Works

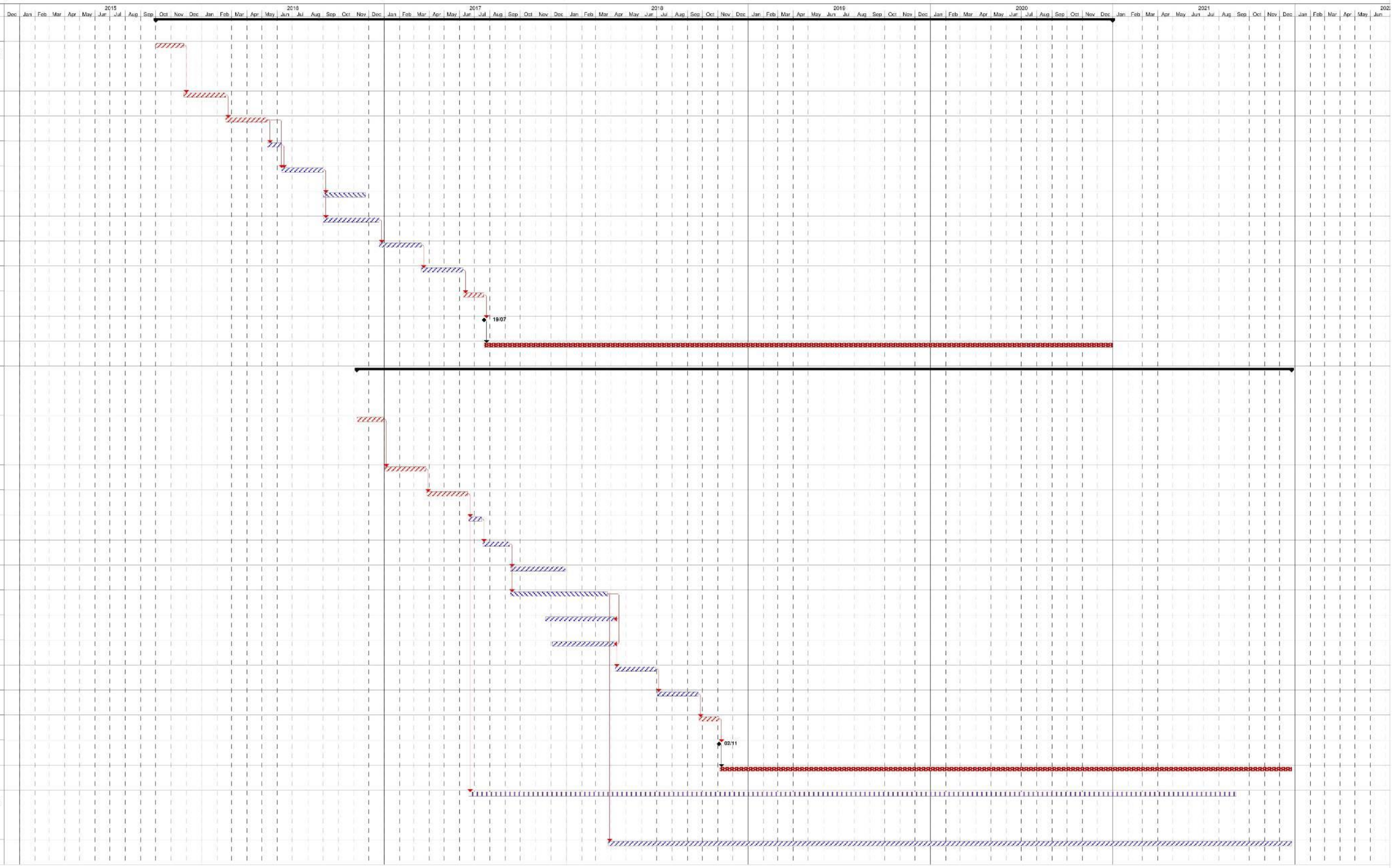


Government or Departmental Task | PSP Design Activities | Works Construction | Environmental Task | Other Key Activities Required | Milestone | Summary

2 OF 6

MZIMVUBU WATER PROJECT: IMPLEMENTATION PROGRAMME
LAST UPDATED MARCH 2015

ID	Task Name
68	**NTABELANGA WTW : DESIGN AND BUILD (PACKAGE 4)**
69	Prepare and Issue Design Management PSP Request for Proposals
70	Tender Period
71	Evaluate Tenders and Appoint Design Management PSP
72	Information Gathering and Review Period
73	Additional Geotechnical and Materials Investigations
74	Final Cost Estimates and Implementation Cashflows
75	Preparation of Bidding Documents
76	Invitations to Tender for Construction and Tender Period
77	Evaluate Tenders Received
78	Approval from Supply Chain Management to Award Contract
79	Award of Design and Build Contract
80	WTW Design, Construction and Commissioning Period
81	**NTABELANGA DAM PRIMARY AND SECONDARY BULK POTABLE WATER SYSTEM (PACKAGE 5)**
82	Prepare and Issue Design and Supervision PSP Request for Proposals
83	Tender Period
84	Evaluate Tenders and Appoint Design and Supervision PSP
85	Information Gathering and Review Period
86	Supplementary Survey of Works Areas
87	Additional Geotechnical and Materials Investigations
88	Detailed design of works
89	Preparation of Tender Drawings
90	Preparation of Bidding Documents
91	Invitations to Tender for Construction and Tender Period
92	Evaluate Tenders Received
93	Approval from Supply Chain Management to Award Contract
94	Award of Construction Contract
95	Construction of Works
96	Relocation Action Plan: Bulk Potable Water Pipelines and Reservoirs: Consultation, Finalisation and Implementation
97	**IMPLEMENTATION OF TERTIARY PIPELINES BY DMs**

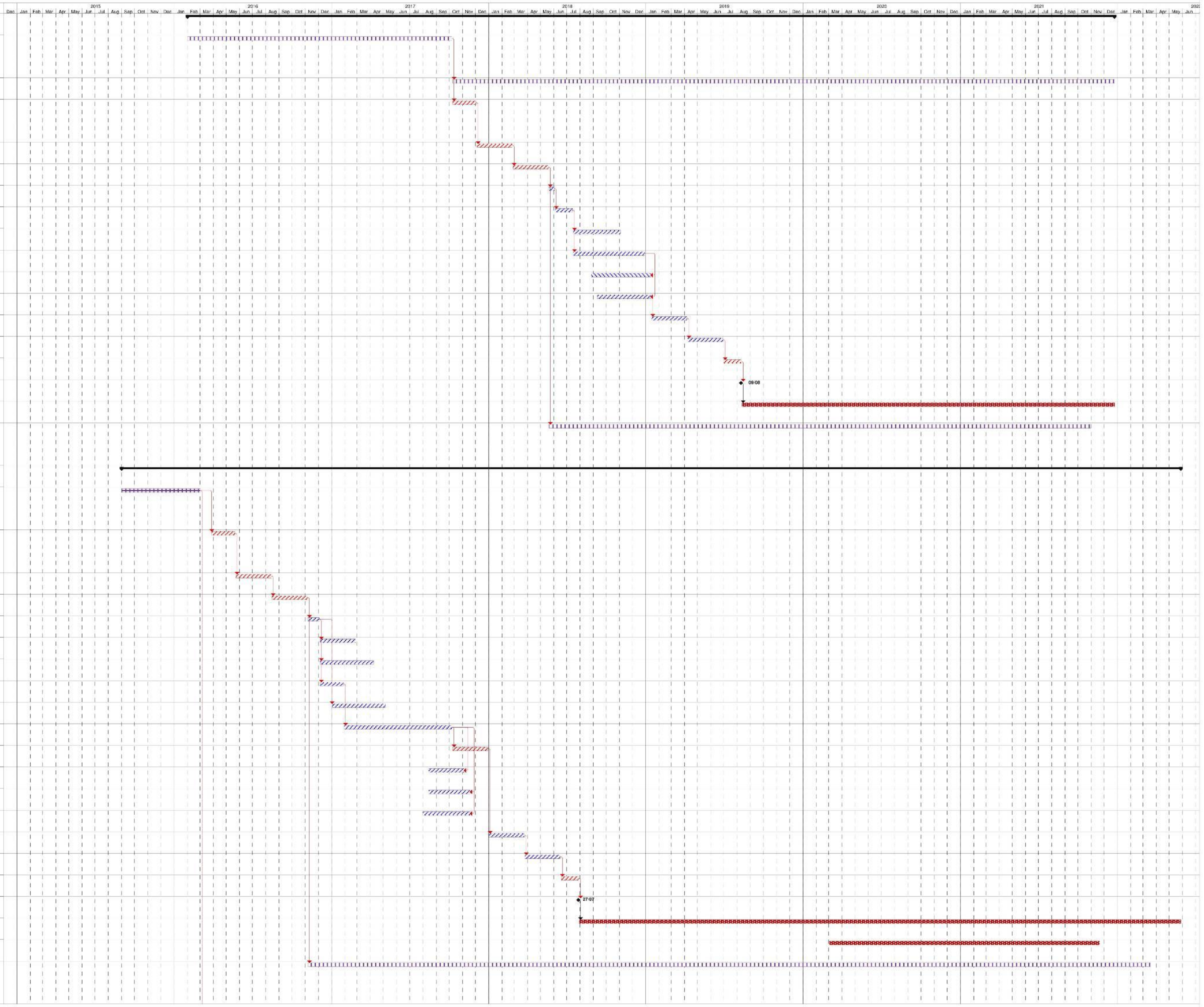


Government or Departmental Task · PSP Design Activities · Works Construction · Environmantal Task · Other Key Activities Required · Milestone · Summary

3 OF 6

MZIMVUBU WATER PROJECT: IMPLEMENTATION PROGRAMME
LAST UPDATED MARCH 2015

ID	Task Name
98	**IRRIGATION SCHEME DEVELOPMENT (PACKAGE 6)**
99	Beneficiary Consultation and Agrarian Reform to Determine Irrigation Requirements
100	Development of On-Farm Infrastructure by DAFF and DRDAR
101	Prepare and Issue Bulk Water Infrastructure Design and Supervision PSP Request for Proposals
102	Tender Period
103	Evaluate Tenders and Appoint Design and Supervision PSP
104	Information Gathering and Review Period
105	Supplementary Survey of Works Areas
106	Additional geotechnical and materials investigations
107	Detailed design of works
108	Preparation of Tender Drawings
109	Preparation of Bidding Documents
110	Invitations to Tender for Construction and Tender Period
111	Evaluate Tenders Received
112	Approval from Supply Chain Management to Award Contract
113	Award of Construction Contract
114	Construction of Works
115	Relocation Action Plan: Bulk Irrigation Water Pipelines and Reservoirs: Consultation, Finalisation and Implementation
116	**LALINI DAM : DESIGN AND CONSTRUCTION (PACKAGE 7)**
117	Negotiate and Sign Agreements with ESKOM and Amatola Green Power on Energy Evacuation and Sales
118	Prepare and Issue Design and Supervision PSP Request for Proposals
119	Tender Period
120	Evaluate Tenders and Appoint Design and Supervision PSP
121	Information Gathering and Review Period
122	Supplementary Survey of Dam Site and Associated Works
123	Additional Geotechnical and Materials Investigations
124	Preliminary Design and Optimisation of Works
125	Spillway Laboratory Modelling & Optimisation
126	Detailed Design of Dam & Associated infrastructure
127	Evaluation of Design by Dam Safety Office and Issue License
128	Final Cost Estimates and Implementation cashflows
129	Preparation of Tender Drawings
130	Preparation of Bidding Documents
131	Invitations to Tender for Construction and Tender Period
132	Evaluate Tenders Received
133	Approval from Supply Chain Management to Award Contract
134	Award of Construction Contract
135	Dam Construction Period
136	Mini-Hydropower Plant Construction Period
137	Relocation Action Plan: Dam Basin and Associated Infrastructure: Consultation, Finalisation and Implementation



Government or Departmental Task | PSP Design Activities | Works Construction | Environmantal Task | Other Key Activities Required | Milestone | Summary

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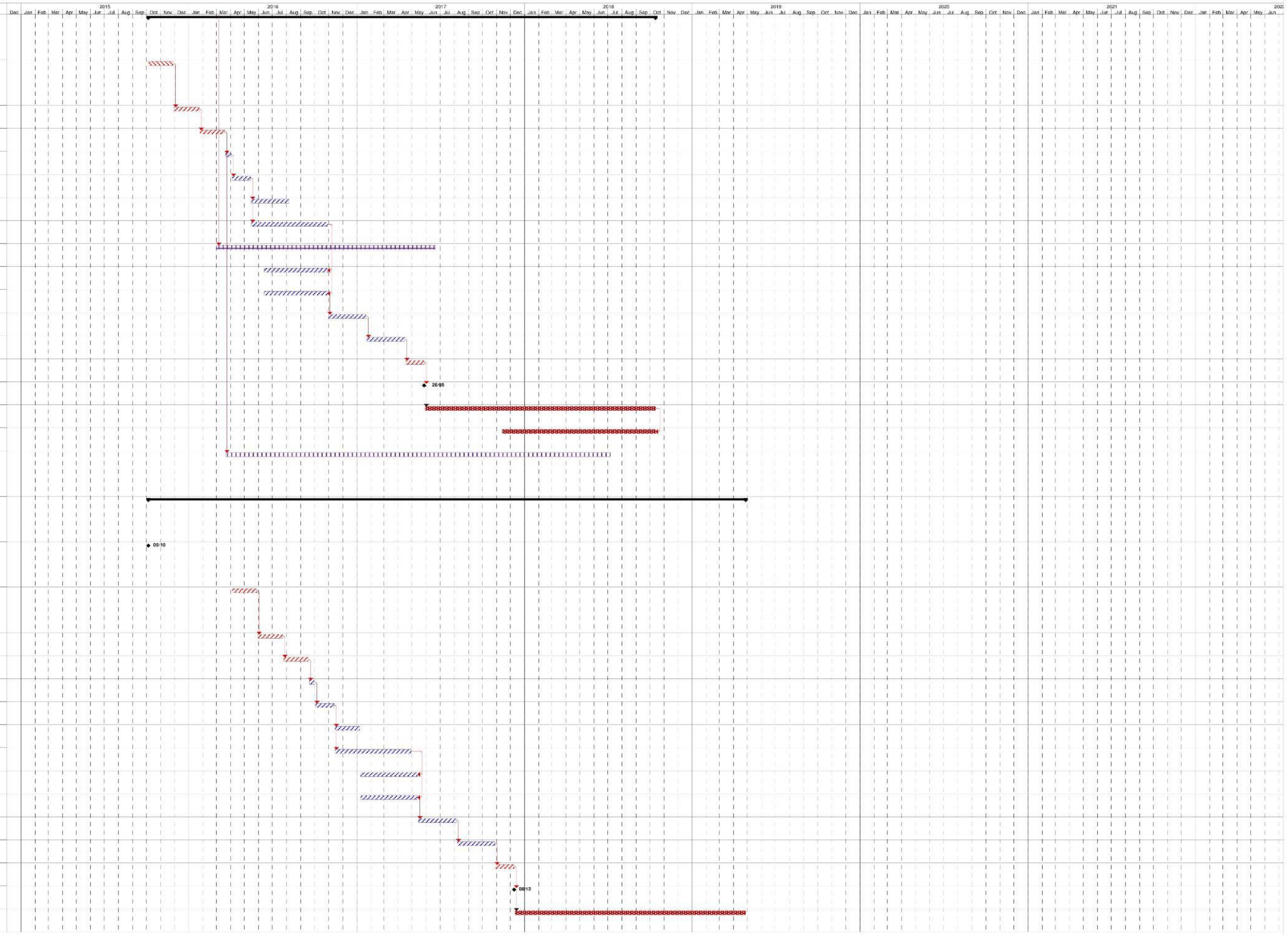
ID	Task Name
138	**LALINI ADVANCE ACCESS ROADS, BRIDGES AND POWER SUPPLIES (PACKAGE 8)**
139	Prepare and Issue Design and Supervision PSP Request for Proposals
140	Tender Period
141	Evaluate Tenders and Appoint Design and Supervision PSP
142	Information Gathering and Review Period
143	Supplementary Survey of Road and Power Lines
144	Additional Geotechnical and Materials Investigations
145	Detailed Design of Roads, Bridges and Power lines
146	Application to ESKOM for Power Supply
147	Preparation of Tender Drawings
148	Preparation of Bidding Documents
149	Invitations to Tender for Construction and Tender Period
150	Evaluate Tenders Received
151	Approval from Supply Chain Management to Award Contract
152	Award of Construction Contract
153	Construction of Access Roads and Bridge
154	Construction of Power Lines
155	Relocation Action Plan: Access Roads, Bridges and Power Lines: Consultation, Finalisation and Implementation
156	**LALINI OPERATIONS ACCOMMODATION VILLAGE AND VISITORS CENTRE (PACKAGE 9)**
157	LALINI OPERATIONS ACCOMMODATION VILLAGE AND VISITORS CENTRE (PACKAGE 9)
158	Prepare and Issue Design and Supervision PSP Request for Proposals
159	Tender Period
160	Evaluate Tenders and Appoint Design and Supervision PSP
161	Information Gathering and Review Period
162	Supplementary Survey of Works Areas
163	Additional Geotechnical and Materials Investigations
164	Detailed Design of Works
165	Preparation of Tender Drawings
166	Preparation of Bidding Documents
167	Invitations to Tender for Construction and Tender Period
168	Evaluate Tenders Received
169	Approval from Supply Chain Management to Award Contract
170	Award of Construction Contract
171	Construction of Works



Government or Departmental Task | PSP Design Activities | Works Construction | Environmental Task | Other Key Activities Required | Milestone | Summary

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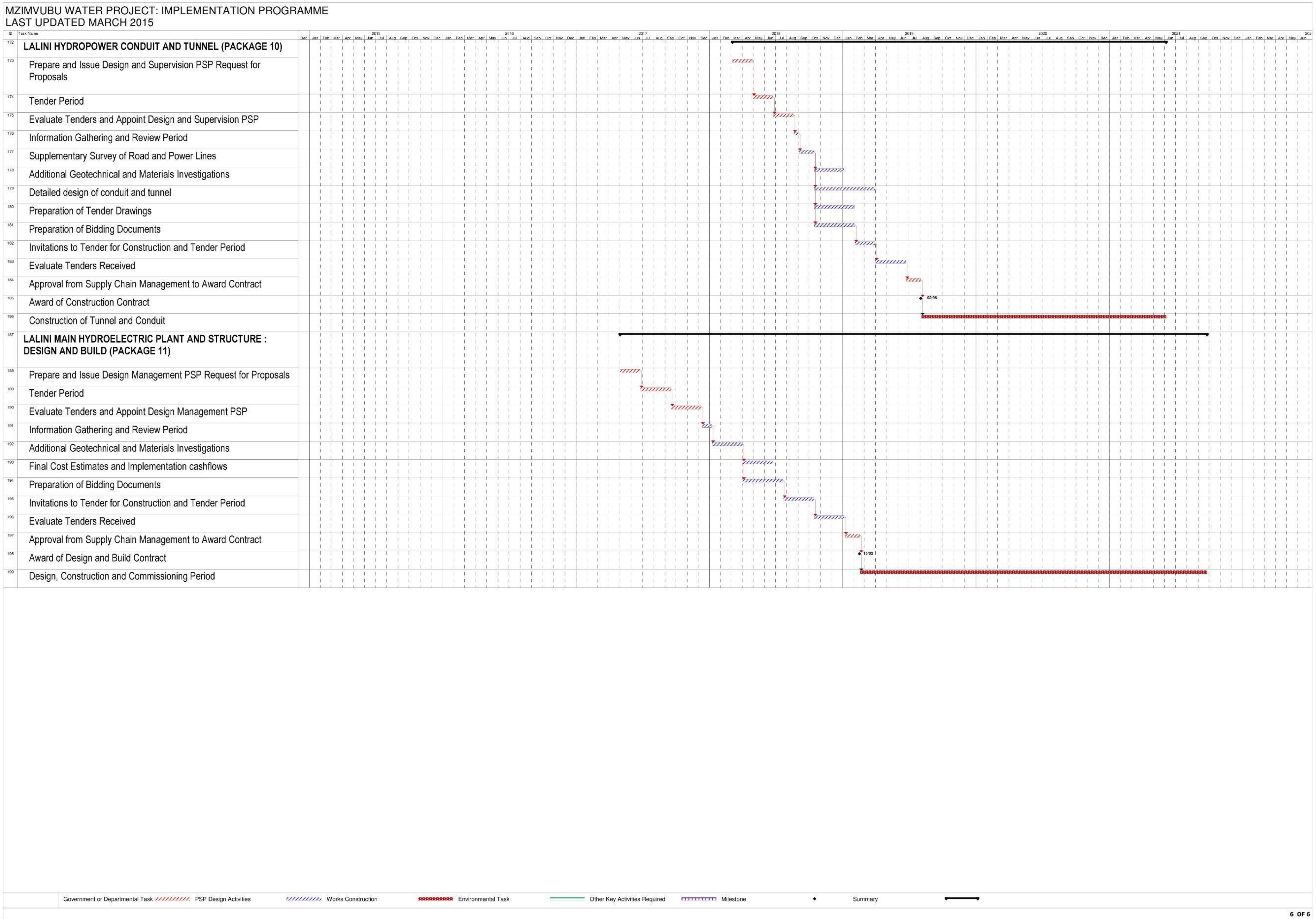
MZIMVUBU WATER PROJECT: IMPLEMENTATION PROGRAMME
LAST UPDATED MARCH 2015
ID
Task Name
172 LALINI HYDROPOWER CONDUIT AND TUNNEL (PACKAGE 10)
173 Prepare and Issue Design and Supervision PSP Request for Proposals
174 Tender Period
175 Evaluate Tenders and Appoint Design and Supervision PSP
176 Information Gathering and Review Period
177 Supplementary Survey of Road and Power Lines
178 Additional Geotechnical and Materials Investigations
179 Detailed design of conduit and tunnel
180 Preparation of Tender Drawings
181 Preparation of Bidding Documents
182 Invitations to Tender for Construction and Tender Period
183 Evaluate Tenders Received
184 Approval from Supply Chain Management to Award Contract
185 Award of Construction Contract
186 Construction of Tunnel and Conduit
187 LALINI MAIN HYDROELECTRIC PLANT AND STRUCTURE : DESIGN AND BUILD (PACKAGE 11)
188 Prepare and Issue Design Management PSP Request for Proposals
189 Tender Period
190 Evaluate Tenders and Appoint Design Management PSP
191 Information Gathering and Review Period
192 Additional Geotechnical and Materials Investigations
193 Final Cost Estimates and Implementation cashflows
194 Preparation of Bidding Documents
195 Invitations to Tender for Construction and Tender Period
196 Evaluate Tenders Received
197 Approval from Supply Chain Management to Award Contract
198 Award of Design and Build Contract
199 Design, Construction and Commissioning Period
2015
2016
2017
2018
2019
2020
2021
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Milestone
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