Feasibility Study for Foxwood Dam (WP10580)

Feasibility Dam Design

Final

DWS Report Number: **P WMA 15/Q92/00/2113/12**

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STUDY REPORTS

The Feasibility Dam Design records the design process followed and proposed dam solution. This report forms one of the suite of reports that make-up the Feasibility Study for Foxwood Dam. The full list of reports is provided below:

Feasibility Study for Foxwood Dam: Inception Report	**P WMA 15/Q92/00/2113/1**
Feasibility Study for Foxwood Dam: Preliminary Study Report	**P WMA 15/Q92/00/2113/2**
Feasibility Study for Foxwood Dam: Environmental Screening	P WMA 15/Q92/00/2113/3
Feasibility Study for Foxwood Dam: Geotechnical Reconnaissance	P WMA 15/Q92/00/2113/4
Feasibility Study for Foxwood Dam: Alternative Water Supply Options	P WMA 15/Q92/00/2113/5
Feasibility Study for Foxwood Dam: Feasibility Study Main Report	**P WMA 15/Q92/00/2113/6**
Feasibility Study for Foxwood Dam: Koonap River Hydrology	P WMA 15/Q92/00/2113/7
Feasibility Study for Foxwood Dam: Water Requirements	P WMA 15/Q92/00/2113/8
Feasibility Study for Foxwood Dam: Agro-Economic Study Report	P WMA 15/Q92/00/2113/9
Feasibility Study for Foxwood Dam: Water Quality	P WMA 15/Q92/00/2113/10
Feasibility Study for Foxwood Dam: Geotechnical Investigation	P WMA 15/Q92/00/2113/11
Feasibility Study for Foxwood Dam: Dam Feasibility Design	P WMA 15/Q92/00/2113/12
Feasibility Study for Foxwood Dam: Project Feasibility Costing	P WMA 15/Q92/00/2113/13
Feasibility Study for Foxwood Dam: Economic Impact Assessment	P WMA 15/Q92/00/2113/14
Feasibility Study for Foxwood Dam: Record of Implementation Decisions	P WMA 15/Q92/00/2113/15
Feasibility Study for Foxwood Dam: Book of Maps	P WMA 15/Q92/00/2113/16
Feasibility Study for Foxwood Dam: Public Participation (Queries & Responses Report)	P WMA 15/Q92/00/2113/17

REPORT REFERENCE

This report is to be referred to in bibliographies as:

Department of Water and Sanitation, 2015. Feasibility Study for Foxwood Dam: Dam Feasibility Design, P WMA 15/Q92/00/2113/12

Note on Departmental name change

In 2014, the Department of Water Affairs (DWA) changed its name to the Department of Water and Sanitation (DWS). This occurred during the course of this study and as a result some reporting which was commenced and/or approved prior to the name change may still refer to DWA. References herein to DWA and DWS should be considered one and the same.

EXECUTIVE SUMMARY

The Department of Water and Sanitation is investigating the feasibility of developing a multi-purpose dam on the Koonap River near Adelaide in the Eastern Cape. The project is being considered for implementation as a strategic initiative to mobilize the water resources in the area as a stimulus for socio-economic development in this rural, economically depressed region. This initiative would support the objectives of the National Development Plan (NDP) and is consistent with the National Water Resource Strategy 2 (NWRS2).

The Foxwood Dam site is located immediately upstream of Adelaide (coordinates 32˚40'30"S, 26˚16'0"E) in the Koonap River catchment. The Koonap River catchment, with an area of 3 334 km², is situated in the Eastern Cape Province and lies within the Fish to Tsitsikamma Water Management Area (WMA). Foxwood Dam has a catchment of 1 091 km².

The proposed Foxwood Dam is a composite gravity concrete – earthfill embankment dam designed for a final height corresponding to 1 MAR storage with a 50 year sedimentation allowance. The FSL is 615,0 masl. This report is intended to refer to the feasibility design aspects of the dam and addresses the following design aspects:

Site details	Stability analysis
Hydrology	Outlet works
Geology	Construction
Spillway	Dam safety aspects
Selection of Full Supply Level & dam type	Land acquisition and realignment of services
Cost estimates	

It is a Category III dam and as such the detailed design will be carried out in terms of the current National Water Act Chapter 12 sections 117 to 123. The spillway is designed to:

- Pass the Recommended Design Flood (RDF) of 2 063 m³/s (1 in 200 year flood event) with dry freeboard.
- Pass the Safety Evaluation Discharge (SED) of 6 200 m³/s (PMF) without freeboard.

The freeboard selection was undertaken in accordance with SANCOLD Guidelines on Freeboard for Dams Volume II. 5,5 m freeboard is provided between the Full Supply Level (615,0 m) and embankment crest level (620,5 m). Key dam statistics are given in Table 1 below. The design of the spillway may be optimized during detailed design with the Kovacs + Δ method (5 218 m³/s).

This design includes a spillway bridge to access the right bank crest. This form of access is not favoured by the Department but appears to be significantly less costly than an alternative road access cut through the right bank abutment. This will need to be considered in the detailed design stage of this project.

The type of dam was selected on the basis of the optimal URV using a common bill of quantities and rates. The capacity was selected on the basis of environment and operation considerations given that the EWR at the 1 MAR storage spills sufficiently that combined with the downstream intermediary catchments requires the least amount of EWR from the dam outlets. The spillway selection was based upon the above flood criteria and the selected dam type. Two feasible combination of options were considered:

- Side-channel spillway around the left abutment; with either rockfill or earthfill main embankment
- Central overtopping section with either a concrete gravity or composite embankments

The composite dam option was the lowest comparative URV for the 1 MAR capacity. Benefits to this selection include:

- Smaller footprint of the dam site
- Reduced excavation of materials
- Greater opportunity for flow energy dissipation

The downstream face of the spillway is sloped at 0,6H:1V (or 59 degrees). This is the maximum steepness determined from the stability analysis. This is steeper than normally accepted 1:0.75 to 0.7, this is a function of the large spill basin blocks that were set to the ground level rather than the hard horizon. The toe of the concrete gravity dam has a 15 m long stilling basin block which is stepped to follow the ground level. The return is protected by a cascade system of graded large rocks and rip rap underlain by a crusher graded filter. The concrete gravity dam has been checked for global stability using the load combinations and Factors of Safety. In all cases the dam performs satisfactorily. If, during the next stage of design, and geometric or material amendments are made, the global stability will need to be reassessed. In addition, once the construction technique is confirmed the stability at intermittent stages will also need to be evaluated.

To model the embankment stability, as per guidance within Chapter 11 of Geotechnical Engineering of Dams (Fell et. al. 2005) the drawdown state has been analysed assuming instantaneous drawdown with phreatic surface close to the embankment surface. This resulted in an upstream and downstream slope of 1:4 and 1:3 respectively being selected. The results of seepage analysis on the embankment showed that the following will need to be considered within the final design:

- An internal chimney and blanket drain is required to reduce the elevation of seepage through the embankment dam and at the toe
- A cut off trench and grout curtain are likely to be required to reduce the risk of seepage through the alluvial soils and weathered bedrock beneath the embankment dam
- A grout curtain is required to reduce seepage pressures beneath the concrete gravity dam and the left hand side abutment.

Table 1: Key dam statistics

DAM STATISTICS	
Full Supply Capacity:	54,9 million m^3
50 year silt volume	6,11 million m^3
1:20 year Long Term Yield (after allowing for total EWR flows)	15,9 million m^3/a
Reservoir Surface Area at HFL	463 ha
Overall length of wall:	485 m
Length of spillway including bridge piers (net 250m):	267 m
Maximum height of NOC above foundation:	48,5 m
Excavation volume:	234 388 m^3
Earthfill and backfill material volume:	584 820 m^3
Total volume of reinforced concrete:	51 840 m^3
Total concrete volume:	220 183 m^3

The outlet tower is located in the left concrete gravity abutment, which allows for conventional concrete construction methods to be carried out independently of the bulk concrete in the spillway gravity section. The outlet works have been designed with a twin stack 1 000 mm steel pipe system to allow for 100% redundancy and make provision for discharge of the maximum EWR (6 m^3/s) as well as for all downstream abstractions. To ensure that adequate water quality is maintained, multiple level off takes are included in the tower (x4). The individual stacks can discharge the full requirement with a pipe design, a constraint was set at flow velocities < 8 m/s. Access to the outlets works and valve chamber can be via the external face of the concrete wall or internally through the gantry and gallery.

The DWS guidelines recommend that river diversions for a composite dam consisting of both concrete and earth fill sections are designed for floods with a 1:20 year return interval and that a 1:50 year return interval is used for earthfill embankment. The following flood flows were used for the river diversion arrangements at Foxwood Dam:

- 1:20 year flood discharge 555 m^3/s
- 1:50 year flood discharge 985 m^3/s

This is achieved by a combination of initially diverting through a box culvert and then maintaining a low length of spillway during construction of the embankment.

The construction method of the spillway was on the basis of a concrete gravity structure suitable for roller compacted concrete (RCC) construction, (as was the cost estimate.) The earthfill section has been based on the quality of the soils present in the excavations and generally within the basin. There will need to be a careful selection process to determine the suitability of construction materials. This will require monitoring and further detailed investigation, as there were samples exhibiting dispersive characteristic in addition to the distinction between shoulder and core materials. There are no naturally occurring filter quality sands located within the area. Consequently filter sand, aggregates and rip rap can be manufactured from an identified and investigated quarry site, located in a competent dolerite outcrop, suitable for this type of production.

The cost estimates have been based on preliminary drawings and bills of quantities for the various options, (Earthfill, rockfill, gravity and composite) Conservative rates were provided by the quantity surveying department of a large civil engineering contractor who was at that stage completing a similar type and size of dam. The rates were supplied mid-2014.The total capital estimated capital cost of the 1 MAR composite dam based on measured quantities with all associated cost related to the dam construction including detailed design, compensation, relocation of various services and associated fees is **R 2 084 186 082**. See Table 2 below for a breakdown of the dam structure costs and Table 3 for a breakdown of the main project costs.

The URV associated with these costs was calculated based on the VAPS guidelines for annual maintenance, operation and life cycles. The URV, at an **8% discount rate**, is **R 11.77 /m^3**.

Table 2: Summary of estimated dam structure construction cost

Description	Amount (ZAR)	Comment
PRELIMINARY & GENERAL	239 411 545	*30% of all items*
WATER CONTROL-RIVER DIVERSION	5 118 848	
DRILLING & GROUTING	65 895 189	
Earthfill	5 772 591	
Concrete Gravity	60 122 598	
GRAVITY SPILLWAY	434 835 032	

Description	Amount (ZAR)	Comment
GRAVITY NOC	26 515 352	
EARTHFILL EMBANKMENT	105 196 437	
OUTLET WORKS	64 306 681	
Concrete Works	21 204 550	
Mechanical Equipment	39 102 131	
Structural Steelwork	1 750 000	
Electrical Equipment	2 250 000	
INSTRUMENTATION	7 500 000	*Provisional Sum*
Miscellaneous 10% & Landscaping 2,5%	88 670 942	*(12,5% of cost* (excl P&G))
DAM CONSTRUCTION (incl. P&G)	1 167 651 897	

Table 3: Summary of estimated dam structure construction cost

Foxwood Dam Feasibility Cost Estimate	**ZAR**
Foxwood Dam Structure (only)	1 167 651 897
Dam Access Road	9 412 689
Bulk water pipeline and pumpstation	8 887 960
Gauging Weir & other DWS hydrology structures	5 451 000
Relocation of R344 (MR00638)	126 599 941
Relocation of water supply canal	20 400 000
Land matters - land costs	10 239 625
Land matters - fixed improvements	25 764 000
Graves relocation	300 000
Eskom relocation cost	2 200 000
Telkom relocation cost	500 000
Environmental management	5 000 000
GRAND TOTAL	**1 382 407 112**
Contingencies 15%	207 361 067
TOTAL DAM CONSTRUCTION (incl contingency)	**1 589 768 179**
Professional Fees 15%	238 465 227
TOTAL COST (incl design fees)	**1 828 233 406**
VAT	255 952 677
TOTAL DAM COSTS	**2 084 186 082**

Comments were received from DWS on the first draft of this report. The comments and the responses are included in Appendix I. The agreed modifications and clarification have been included in this report and recorded in the Record of Implementation Decisions (RID) (DWS 2015f) where relevant.

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LIST OF ACRONYMS

ACRONYM	
ADM	Amathole District Municipality
AIP	Alien Invasive Plants
APP	Approved Professional Person
AW	Amatola Water
DAFF	Department of Agriculture, Fisheries and Forestry
EPP	Emergency Preparedness Plan
EIS	Ecological Importance And Sensitivity
EWR	Ecological Water Requirements
DFL	Full Supply Level
DT	Discharge Tables
FSL	Full Supply Level
FRPS	Fish River Pumping Scheme
FSC	Full Supply Capacities
GRA	Groundwater Resources Assessment
HFL	High Flood Level
HFY	Historic Firm Yield
IEI	Integrated Environmental Importance
LTY	Long Term Yield
MAP	Mean Annual Precipitation
MAR	Mean Annual Runoff
MRU	Management Resource Units
NEMA	National Environmental Management Act
NGA	Ground Water Database
nMAR	Natural Mean Annual Runoff
NOC	Non overflow section
Nxuba	Nxuba Local Municipality
PES	Present Ecological State
PMF	Probable Maximum Flood
PSP	Professional Service Provider
RDREM	Reserve Estimation Model
RID	Record of Implementation Decisions
RMF	Regional Maximum Flood
RTS	Reservoir-Triggered Seismicity
SA	South Africa
SCI	Socio-Cultural Importance
SEF	Safety Evaluation Flood

ACRONYM	
SFR	Streamflow Reduction
StatsSA	Statistics South Africa
STOMSA	Stochastic Model of South Africa
URV	Unit Reference Value
UWP	UWP Consulting (Pty) Ltd
VAPS	Vaal Augmentation Planning Study
WARMS	Water Use Registration Database
WfW	Post Retief Working For Water
WMA	Water Management Area
WR-IMS	Water Resources Information Management System
WRUI	Water Resource Use Importance
WRYM	Water Resources Yield Model
WSA	Water Service Authority
WSDP	Water Services Development Plan
WSP	Water Service Provider
WTW	Water Treatment Works

LIST OF UNITS

MEASURE	UNIT
Height	m.a.s.l.
Distance	m or km
Dimension	mm, m
Flow rate	l/s or m^3/s
Area	m^2, ha or km^2
Volume (storage)	m^3, million m^3

1 INTRODUCTION

1.1 Feasibility Study for Foxwood Dam

The Department of Water and Sanitation is investigating the feasibility of developing a multi-purpose dam on the Koonap River near Adelaide in the Eastern Cape. The proposed dam site is known as Foxwood and was identified for the development of the water resources of the Koonap River as far back as the 1960's. The project is again being considered for implementation as a strategic initiative to mobilize the water resources in the area as a stimulus for socio-economic development in this rural, economically depressed region. This initiative would support the objectives of the National Development Plan (NDP) and is consistent with the National Water Resource Strategy 2 (NWRS2).

Development of the Foxwood Dam would, in the first instance, provide additional, high assurance water supplies for domestic use; this would significantly improve the resilience of the limited supplies now available from the Koonap River without the benefit of storage, and would make water available to meet any increasing needs for domestic, municipal and industrial use.

The effective development of a major storage dam at the Foxwood site would regulate the variable runoff in the Koonap River to the extent that, after full provision is made for maintaining the Reserve to ensure the health and integrity of the resource itself, a significant quantity of water would be made available for irrigation development at an appropriate level of assurance. It is this resource that would be mobilized, together with land and human resources in the region, to provide a stimulus for socio-economic development.

The Foxwood Dam site is located immediately upstream of Adelaide (coordinates 32˚40'30"S, 26˚16'0"E) in the Koonap River catchment (see Figure 1). The Koonap River catchment, with an area of 3 334 km², is situated in the Eastern Cape Province and lies within the Fish to Tsitsikamma Water Management Area (WMA). Adelaide is in the Nxuba Local Municipality (Nxuba) within the Amathole District Municipality (ADM) (see Figure 2 below). ADM is the Water Service Authority (WSA) in the Nxuba Municipality and Amatola Water (AW) is the Water Service Provider (WSP). Adelaide current is supplied with potable water via an off-channel canal from the Koonap River that feeds a storage dam to the north of the town. This system is backed-up by a transfer pipeline from the Fish River (installed as an emergency intervention during historic times of drought) and municipal boreholes. Comprehensive details on the existing water supply infrastructure are provided in the Alternative Supplies Report (DWS 2015).

1.2 Scope of Design Report

This report records the design process followed for the design of the proposed dam at Foxwood.

The proposed Foxwood Dam is a composite gravity concrete – earthfill embankment dam designed for a final height corresponding to 1 MAR storage with a 50 year sedimentation allowance. The FSL is 615,0 masl. Foxwood Dam is classified as a Category III dam.

This report addresses the following design aspects:

- Site details
- Hydrology
- Geology
- Spillway
- Selection of Full Supply Level & dam type
- Stability analysis
- Outlet works
- Construction

- Dam safety aspects
- Land acquisition and realignment of services;
- Cost estimates

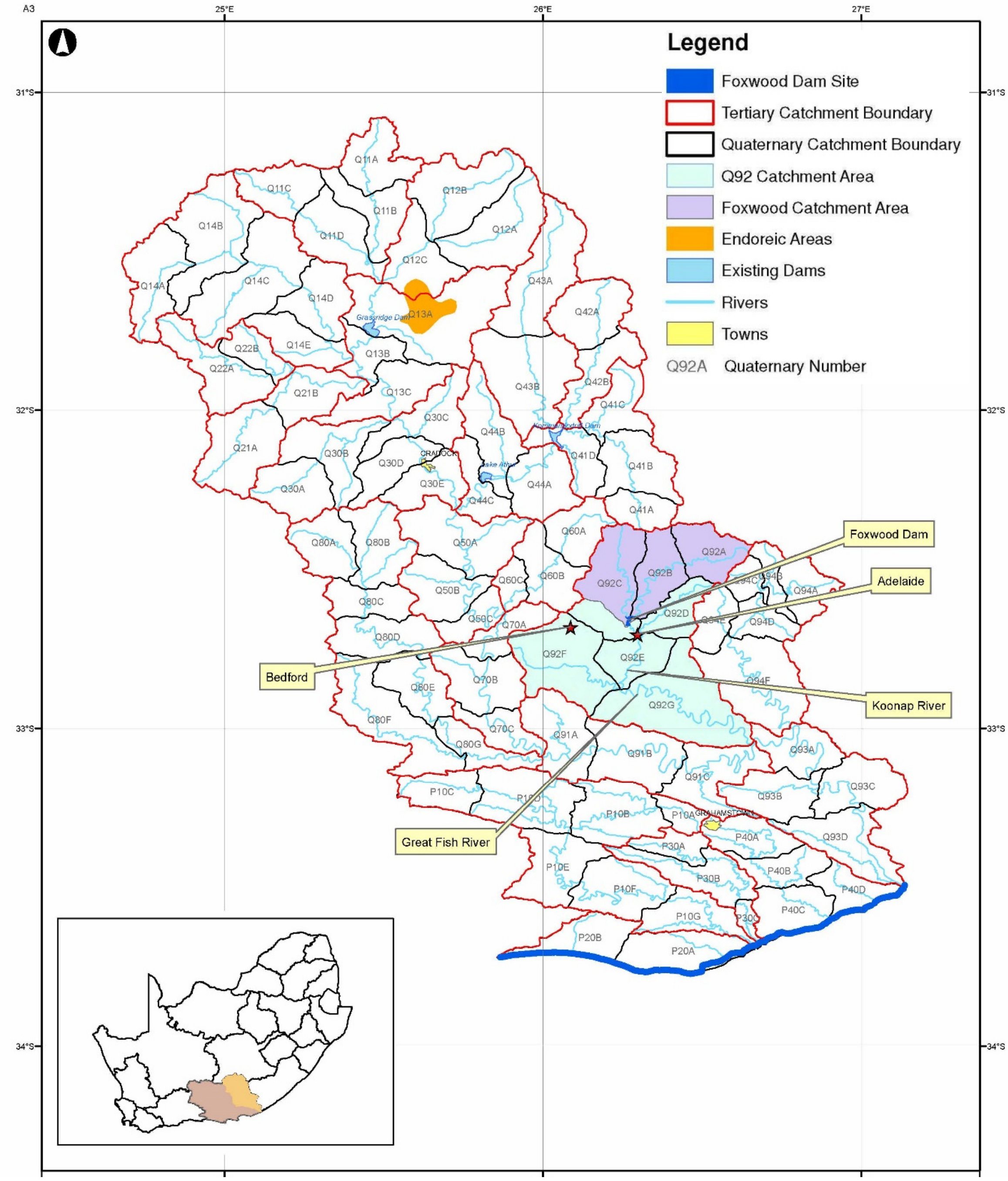

Figure 1: Fish River Catchment with Koonap River Sub-catchment

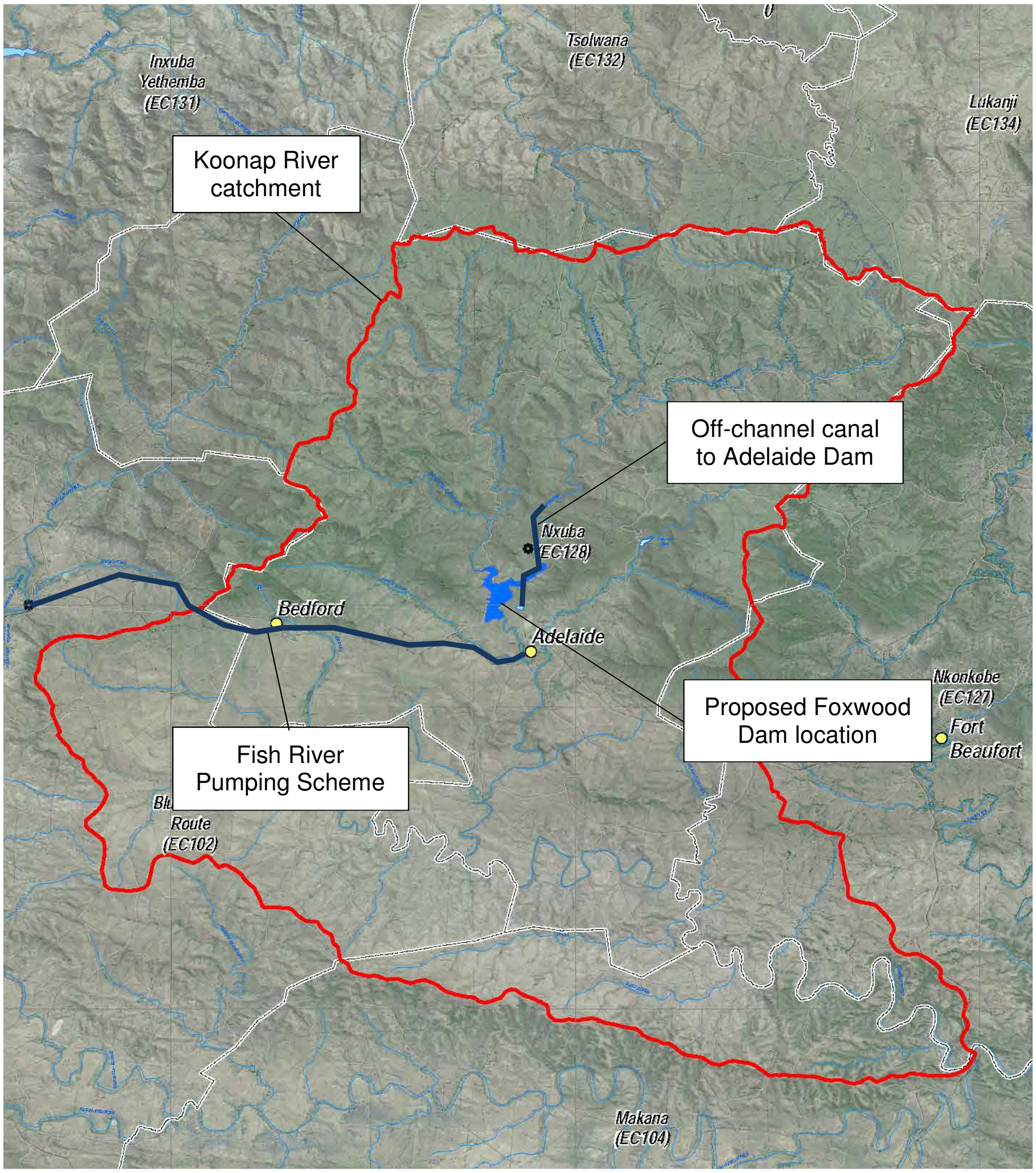

Figure 2: Foxwood Dam location within Koonap River catchment

2 SITE DETAILS

2.1 Locality

The Foxwood Dam site is located upstream of the Eastern Cape town of Adelaide (coordinates 32˚40’30”S, 26˚16’0”E) in the Koonap River catchment.

2.2 Affected areas

The location of Foxwood Dam within the context of Adelaide and showing current water supply infrastructure is shown in Figure 2. Adelaide is located within Nxuba Local Municipality (Nxuba) within the Amathole District Municipality (ADM). ADM is the Water Service Authority (WSA) responsible for water services in the Nxuba and Amatola Water (AW) is the Water Service Provider (WSP).

2.3 Access route to the site

Adelaide is located 170 km West North West from East London which is the nearest large airport. It is on the R63 via King Williams Town and Fort Beaufort. Access to the left bank is approximately 4 km up the R344 to Tarkastad. There is a municipal gate and an ill-defined track which will only partially reach the Koonap River, approximately 500 m to the dam centerline and river. See Figure 3 below.

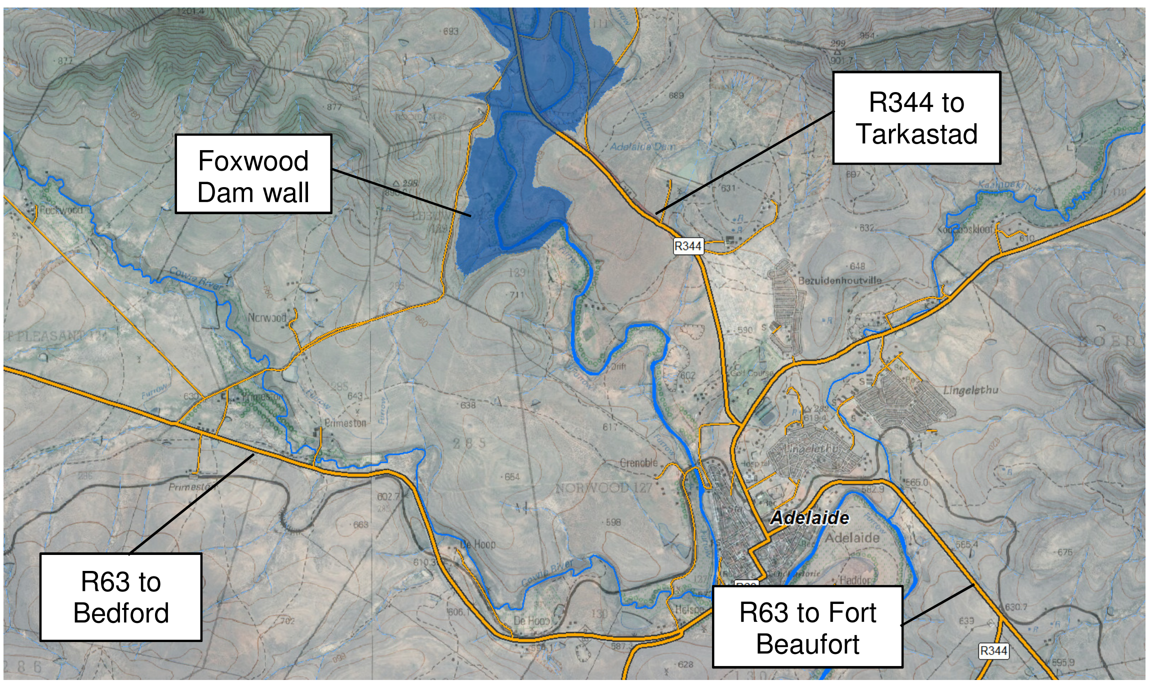

Figure 3: Plan showing regional access to proposed Foxwood Dam

3 HYDROLOGY

3.1 Introduction

The area of the Foxwood Dam catchment is 1 091 km² which is 33% of the total area of the Koonap River catchment (3 334 km²). Important tributaries of the Koonap River include the Braambospruit, Mankazana, Waterkloof and Enyara Rivers. The Foxwood Dam and Lower Koonap River catchments have similar land use in that both catchments are rural in nature with agriculture the dominant activity. The Koonap River catchment falls within the Eastern Cape Province and has no major towns. The small towns of Adelaide and Bedford are located within the lower Koonap River catchment. Water related infrastructure in the Koonap River catchment is dominated by run of river abstractions or diversions for domestic use and for the irrigation of crops ranging from pastures to citrus. The full hydrological study is published as a separate report (DWS 2015a).

3.2 Ecological Water Requirements

The intermediate level Ecological Water Requirements (EWR) study identified the Recommended Ecological Category (REC) as a C-category at both EWR sites, which is the same as the Present Ecological State (PES). The operating rule recommended by the Reserve specialist is that the low flow EWR assurance rule should be implemented at these sites. Provision for the EWR requires a discharge of up to 6 m^3/s. Further information on the EWR is provided in section 5.3.

3.3 Hydrological analyses

3.3.1 Rainfall and stream flow

The Koonap River catchment falls within the summer rainfall zone, but is located adjacent to the year-round zone of coastal catchments, which means rainfall can occur at any time of the year. The Mean Annual Precipitation (MAP) varies from 662 mm in the northern headwater catchments in the Winterberg Mountains to 446 mm in the southern Enyara catchment.

Information about rainfall was obtained from previous studies and from the DWS in the Eastern Cape. A total of 21 rain gauges in and around the Koonap River catchment were identified and screened using standard validation tests. After screening, 4 gauges were excluded from further analysis. The remaining gauges were used in the patching process to generate catchment rainfall records for the period 1920 to 2011 (92 years). The Mean Annual Symons Pan Evaporation (MAE) in the Foxwood Dam catchment area is in the order of 1651 mm.

There are two operational flow gauges within the Koonap River catchment. The Q9H030 gauge is located in the headwaters of the Foxwood Dam catchment. The Q9H002 gauge is located just downstream of the proposed site for Foxwood Dam.

3.3.2 Rainfall-runoff calibration and natural flows

The naturalized stream flows for all catchments were generated and compared with previous studies. The results of the comparison show similar unit runoffs across studies. The naturalized MAR at the proposed Foxwood Dam site is 47,61 million m^3/a.

3.4 Yield model configuration

The Water Resources Yield Model has been configured to assess the historic, long-term and short-term capability of the Foxwood Dam system for a range of live storage capacities ranging from 23,8 million m^3 to 95,2 million m^3. These live capacities are equivalent to nMAR's (Mean

Annual Runoff) of 0,5 nMAR to 2 nMAR. Analyses were undertaken based on a monthly time-step and at-present day (2011/12) development levels.

3.4.1 Scenario development

Three water requirements scenarios have been addressed in previous studies (DWS 2015a):

- Scenario 1: Best estimate of present day (2012/13) development levels with Foxwood Dam.
- Scenario 2: Best estimate of present day (2012/13) development levels with Foxwood Dam and Total Flow EWR assurance rule implemented.
- Scenario 3: Best estimate of present day (2012/13) development levels with Foxwood Dam and Low Flow EWR assurance rule implemented.
-

3.4.2 Yield Assessment

The results of the firm yield, long term and short term stochastic yield assessments for Foxwood Dam for range of storage capacities are provided for scenarios 2 and 3 in Table 4 and Table 5.

Table 4: WRYM model results - Historic and long term yields of proposed Foxwood Dam for range of storage capacities

Reservoir capacity as a ratio of nMAR	FSL Elevation	Wall height	Live storage	Dead Storage	FSC	EWR KOON1	EWR KOON2	HFY	Critical period		Long term yield ($10^6m^3/a$) at Recurrence Interval		
	(m.a.s.l)	(m)	(10^6m^3)	(10^6m^3)	(10^6m^3)	(million m^3/a)			Start	End	1:20	1:50	1:100
Scenario 2 – Foxwood Dam system with EWR rule supplied for total flows (incl. high flows)													
0,5 nMAR	608,5	33,5	23,81	6,11	29,92	7,86	13,00	6,88	7/1944	4/1948	9,7	7,8	6,7
0,75 nMAR	611,6	36,8	35,71	6,11	41,82	7,86	13,00	9,69	7/1944	3/1950	13,7	11,1	9,3
1,0 nMAR	614,6	39,6	47,61	6.11	53,72	7,86	13,00	12,52	7/1944	4/1950	15,9	13,3	11,3
1,5 nMAR	619,5	44,5	71,42	6.11	77,52	7,86	13,00	17,50	7/1954	9/1970	19,8	16,9	14,9
2,00 nMAR	623,1	48,1	95,22	6.11	101,33	7,86	13,00	18,91	7/1954	12/1970	22,8	19,5	17,2
Scenario 3 – Foxwood Dam system with EWR rule supplied for low flows (excl. high flows)													
0,5 nMAR	608,5	33,5	23,81	6,11	29,92	2,18	5,30	10,23	7/1944	4/1948	12,8	11,0	9,5
0,75 nMAR	611,6	36,8	35,71	6,11	41,82	2,18	5,30	13,36	7/1944	3/1950	17,2	13,8	12,4
1,0 nMAR	614,6	39,6	47,61	6,11	53,72	2,18	5,30	16,56	7/1944	3/1950	19,1	16,4	14,6
1,5 nMAR	619,5	44,5	71,42	6,11	77,52	2,18	5,30	20,47	11/1986	4/1997	22,9	20,3	18,0
2,00 nMAR	623,1	48,1	95,22	6,11	101,33	2,18	5,30	21,88	7/1954	12/1970	26,2	22,8	20,6

Table 5: WRYM model results - Short term yields of proposed Foxwood Dam with live storage capacity of 1nMAR

Recurrence Interval	Short term yields for various starting storages ($10^6m^3/a$)					
	100 %	80 %	60 %	40 %	20 %	10 %
Results for scenario 2 for 1nMAR dam with Total Flow EWR						
1:5	28,7	27,7	25,7	23,5	19,2	14,4
1:10	23,0	21,8	20,2	17,5	12,9	9,3
1:20	19,0	17,6	15,9	13,1	9,0	6,2
1:50	15,4	14,0	11,9	9,4	5,7	3,5
1:100	12,8	11,7	10,4	7,1	4,5	2,3
1:200	11,3	10,6	8,7	5,8	3,6	1,8
Results for scenario 3 for 1nMAR dam with Low Flow EWR						
1:5	32,0	30,6	29,0	26,6	21,6	15,6
1:10	26,3	24,8	23,0	20,4	15,7	11,0
1:20	22,1	21,0	19,1	15,8	11,7	8,0
1:50	18,5	16,9	15,1	12,0	8,4	5,6
1:80	16,4	15,4	12,7	10,8	6,7	4,3
1:100	15,3	13,7	11,1	9,9	5,6	3,3

For both scenarios for live storages of 1,5 nMAR and greater the yield gained relative to increased storage capacity is insignificant as shown by a flattening of the storage-yield relationship.

The selection of dam size and dam type is discussed later in this report.

3.5 Flood hydrology

Table 6 gives a comparison among all the different methods used to determine the flood-frequency analysis for Foxwood Dam. The Rational formula consistently yields lower peaks than the Q9H002 analysis (adjusted for catchment area), whereas the converse is true for the Kovacs method. The remaining three methods are much closer but have flatter slopes, with the result that peaks for shorter return periods are above Q9H002 (adjusted) and are below Q9H002 (adjusted) for the longer return periods.

Table 6: Comparison among all methods (peak discharges in m^3/s)

Return Period	Q9H002 adjusted	Unit hydrograph	HRU Regional	Rational Formula	Kovacs method	SDF method
5-year	176	223	349	203	710	282
10-year	332	334	548	257	1 008	438
20-year	555	476	787	317	1 346	629
50-year	985	739	1 165	511	1 865	913
100-year	1 457	1 043	1 500	758	2 318	1 151
200-year	2 063	1 350	1 877	1 084	2 816	1 420

The frequency curves derived from the various methods are compared in Figure 4, where it can be seen that the curve obtained from analysis of the stream flow record at Q9H002 is somewhat steeper than the other curves. Only the Unit Hydrograph and Rational Formula yield similar peaks for the 5-year flood, with the other three being somewhat higher. The Kovacs curve is consistently the highest for all return periods, whereas the other curves – apart from the HRU Regional curve – reflect lower peaks for return periods in excess of 50 years.

Nevertheless, the curve based on Q9H002 falls within the range of the curves obtained from the different methods and can be taken to be a reasonable estimate of the flood-frequency relationship at Foxwood Dam.

This should be considered in the context that Foxwood Dam will be a large dam with a high hazard potential and is classified as a Category III see Section 14 DAM SAFETY ASPECTS.

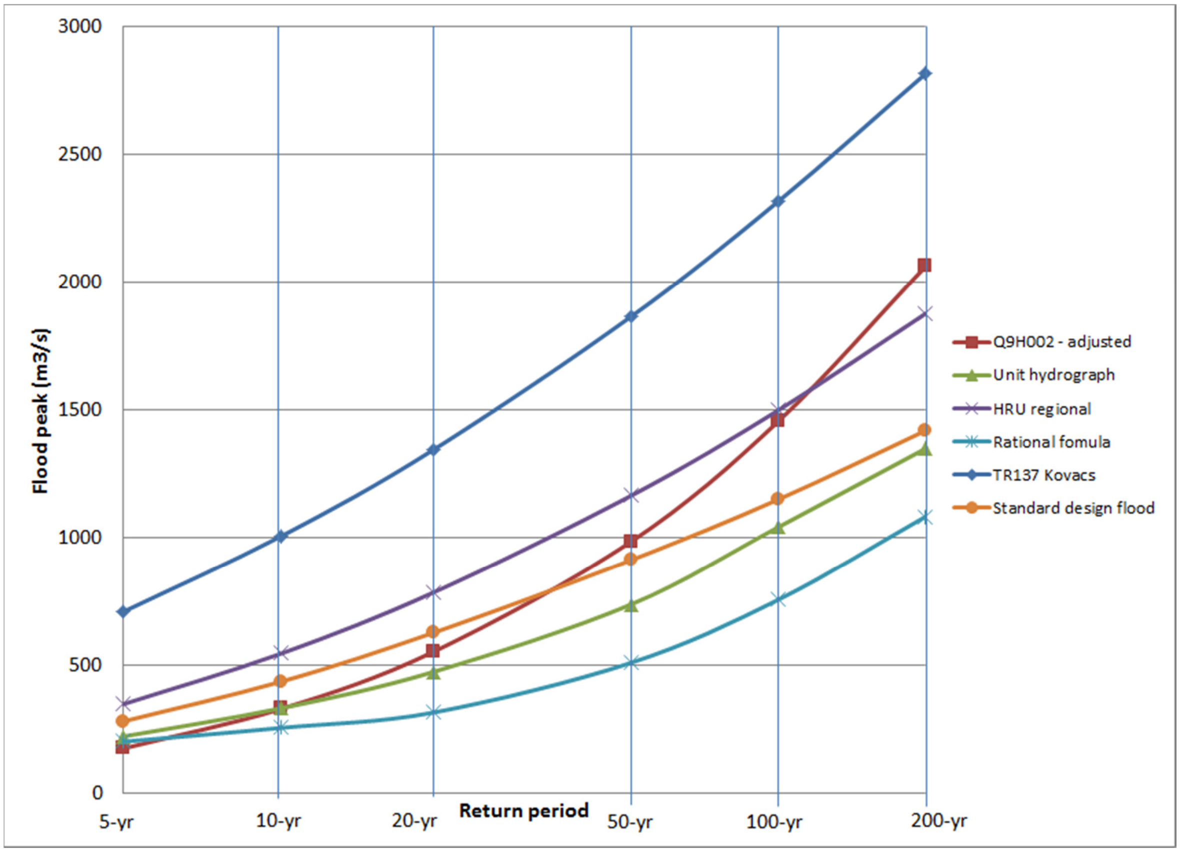

Figure 4: Comparison of Flood Frequency Curves at Foxwood Dam

3.5.1 Probable Maximum Flood

In the absence of adequate data on extreme storms, the 1 in 10 000 flood is sometimes adopted for the Probable Maximum Flood (PMF). Extrapolation of the frequency curve yields a 1 in 10 000 flood peak of 9 900 m³/s (before adjustment for catchment area). However, such a large extrapolation is not reliable and depends to a great degree on the frequency distribution adopted. Another quick method is to calculate the Regional Maximum Flood (RMF) at the site using a "K" value for the next highest region – in this case 5,4. The resulting flood peak is 5 218 m³/s (when using this value, no reduction is allowed for flood attenuation.).

As PMF hydrographs are required for routing purposes the Unit hydrograph method has been adopted. This method yielded the following PMF hydrographs covering a range of storm durations which are shown in Table 7. Note that the 3-hour storm yielded the highest peak of 7 250 m³/s, which is approximately 1,75 times the RMF of 4 164 m³/s. As this ratio is less than 2 the PMF peak can be considered a reasonable, if conservative, estimate.

Table 7: Hydrographs for the PMF

Time Hours	Discharge (m^3/s) for a range of storm durations				
	3-hour	4-hour	6-hour	8-hour	12-hour
0	0	0	0	0	0
1	191	153	114	94	69
2	868	696	518	427	316
3	2944	2360	1759	1448	1072
4	5666	4695	3498	2880	2132
5	**7250**	6356	4850	3993	2956
6	6672	**7014**	5744	4729	3501
7	4896	6260	6309	5288	3915
8	3537	4650	**6444**	5733	4244
9	2781	3431	5647	**6004**	4514
10	2255	2718	4721	5970	4736
11	1845	2204	3215	5192	4916
12	1509	1804	2563	3959	5063
13	1237	1480	2086	3013	**5117**
14	1025	1218	1714	2415	4973
15	856	1011	1412	1972	4303
16	714	844	1166	1624	3314
17	596	784	968	1340	2550
18	493	584	806	1106	2053
19	403	480	667	915	1678
20	319	386	546	755	1378
21	244	302	440	617	1129
22	175	226	345	497	921
23	118	159	261	392	749
24	65	99	185	298	604

3.5.2 Routing of PMF Hydrographs through Reservoir

A survey of the reservoir basin of the proposed Foxwood Dam yielded the relationship between wall height and storage volume for capacities up to and beyond 2 nMAR. This information, in conjunction with depth-discharge characteristics of the proposed spillway, enable the flood routing exercise to be undertaken to determine the maximum depth and discharge over the spillway during passage of the PMF.

A 250 m wide spillway was assumed with a coefficient of 2,0, yielding the following relationship between depth and discharge:

Discharge (m^3/s) = 2,0 x 250 x (Depth (m)) ^{1,5}

Four reservoir capacities were analysed as shown in Table 8. The data is based on a (natural) MAR of 47,61 million m^3 and a dead storage (for sediment accumulation) of 6,11 million m^3.

Table 8: Full supply capacity (FSC) and spillway crest elevations for selected dam sizes

Live capacity (%MAR)	FSC (million m^3)	Crest elevation (masl)
50	29,92	608,5
100	53,72	614,6
150	77,52	619,5
200	101,33	623,1

The PMF flood hydrographs in the table above were routed through the reservoir, starting with the 3-hour (i.e. shortest storm duration) hydrograph. It was not necessary to route any hydrograph longer than a 6 hour duration as it became clear that the longest duration yielding the highest peak was only 4 hours. The results are summarized in the Table 9 below, where the greatest peaks for each dam size are highlighted:

Table 9: Maximum PMF discharge and depth for a range of dam sizes

Storm Duration (h)	50% MAR dam		100% MAR dam		150% MAR dam		200% MAR dam	
	Disch. (m^3/s)	Depth (m)	Disch. (m^3/s)	Depth (m)	Disch. (m^3/s)	Depth (m)	Disch. (m^3/s)	Depth (m)
3	**6 694**	**5,5**	**6 126**		5 816		5 750	
4	6 555		**6 200**	**5,4**	**6 035**	**5,2**	**5 954**	**4,9**
6			5 990		5 937		5 897	

3.5.3 Flood Estimations used for the feasibility design

Estimated flood flows are required for the hydraulic design of the spillway. This is discussed later in this report in Section 6.

The basis for the selection of the dam freeboard is also reliant on flood level estimations. The freeboard is either:

- The Recommended Design Flood / Discharge (RDF / RDD) un-routed over the spillway with a dry freeboard contribution or
- The Safety Evaluation Discharge (SED) is the PMF routed with no dry freeboard.
- The preferred SED method of the Department is the Kovacks + Δ. The selection of this flood estimation method results in a **16%** reduction of the SEF which has an appreciable effect on the spillway length. It could potentially be reduced from 250 m to 200 m for a similar freeboard.

A comparison of the different methods and different spillway lengths is given in Table 10 below.

Table 10: Freeboard selection

Dam and catchment data							
Dam size (MAR)		Fetch length (m)	Average depth (m)	Basin area (ha)	Capacity (million m$^{3)}$	Average basin depth (m)	Upstream Slope
100%		3 279	18,9	463	53,72	11,6	1:4,0
Estimated Flood Flows							
Spillway Widths	m		250	250	225	200	175
PMF Routed	m^3/s		6 200				
RDD (1 in 200 flood)	m^3/s	2 063					
SEF Kovacks + ∆	m^3/s			5 218	5 218	5 218	5 218
Surcharge (m)							
RDD surcharge	m		4,97	4,97	5,15	5,38	5,66
SEF surcharge	m		5,36	4,78	5,12	5,54	6,06
Freeboard	m		5,36	4,97	5,15	5,54	6,06
Selected Freeboard	m		**5,50**				

The following conclusions are taken from the flood estimations:

- A spillway length of 250 m was select to limit the overall freeboard to be less than 6 m;
- The RDD was selected with a return period of 200 years and based on the Q9H002 adjusted basis of 2 063 m^3/s.
- For a 1 MAR dam the SEF (PMF) is 6 200 m^3/s which has a 4 hour storm duration.
- The freeboard has been estimated on the basis of the SANCOLD Guidelines on freeboard for dams Volume II (SANCOLD 2011). The freeboard calculation is discussed later in this report in Section 6.
- Optimisation will form part of the detailed design phase of the project, however, this study considered the routed PMF to be the basis of this feasibility design

This study has selected the SEF (PMF routed) for the selection of spillway dimensions:

- Spillway length 250 m
- SEF discharge 6 200 m^3/s/m
- SEF freeboard requirement 5,5 m

For reference, Arup's full report on flood hydrology is provided in Appendix J.

3.5.4 Loss of storage due to sedimentation

The Koonap River catchment falls within Region 9 of the sediment yield potential map of Southern Africa (WR90, Vol. 5, Map 8.2, 1994). The estimated average rate of sedimentation in the Upper Koonap River catchments is 185 tons\km²\annum based on the Rooseboom methodology (Rooseboom, et al, 1992). This region is characterized by medium erodibility indices.

The loss of storage from sedimentation for the proposed Foxwood Dam was determined for various life spans for a reservoir capacity of around 1 nMAR and is summarised in Table 11 below.

Table 11: Dead storage volumes for Foxwood Dam

Life span	Dead storage volume (million m³)	
(years)	100 % trap efficiency	95 % trap efficiency
20	4,21	4,00
30	5,19	4,93
40	5,89	5,60
50	6,43	6,11

In terms of the yield analysis a life-span of 50 years has been assumed for Foxwood Dam and dead storage of 6,11 million m³ for all storage capacities.

4 GEOLOGY

This section provides a summary on the geological conditions at the dam site and properties of the materials to be used for the dam construction. It is based on the results of the geological investigations performed during August 2013. Full details of the investigation can be found in the site investigation report (DWS 2015c).

4.1 Geological investigations

The geotechnical investigation (GI) comprised the following:

- Trial hole excavations;
- Sampling of unconsolidated soils retrieved from trial holes;
- Drilling of boreholes;
- In situ testing in boreholes;
- Geophysics (seismic surveys); and
- Laboratory testing

4.2 Description of geology

The results of borehole and seismic data is included in Drawings Nos. 225739-GEO-0602 Preliminary Founding Level for Dam Wall & 225739-GEO-0601 Geotechnical Site Investigation Key Plan (Appendix F).

Based on the GI, the dam site and reservoir basin is underlain by sedimentary rocks of the Balfour Formation; Adelaide Subgroup; Beaufort Group; Karoo Supergroup. Rocks consist mainly of grey mudstone and shale with subordinate grey and buff-coloured sandstone.

It is evident from the desk study and the geotechnical investigation that a significant amount (3,0 m – 14,0 m depth) of alluvial silt, sand and cobbles & boulders overly underlying competent mudstone and/ or siltstone rock. It is clear from the boreholes drilled that the rock immediately underlying the alluvial sediment is weathered to depths as great as 24,8 m; in some cases highly weathered. The rock underlying weathered rock is only slightly weathered to unweathered and persists to the end of each borehole at an approximate depth of 30 m.

The mudrocks, comprising mostly olive and grey mudstone, with a high silt component at times approaching siltstone classification, alternate with sandstone units less than a metre up to tens of metres thick consisting of buff/grey, fine grained ultra-lithofeldspathic sandstone, in the approximate ratio 20 % sandstone and 80 % mudstone.

The sandstone displays flat-bedding, through cross-bedding and micro-crosslamination. Sandstone rock is mostly massive. Relatively rapid refusal of excavation will occur in areas underlain by slightly weathered or unweathered sandstone or siltstone. Sandstone is a much hardier rock and is less prone to weathering on exposure than mudstone.

The mudstone is poorly stratified or massive. Near-surface rock generally comprises relatively softer or medium hard rock which quickly hardens with depth to rock that is hard and difficult to excavate. Mudstone undergoes differential weathering on exposure and rapidly fragments into angular pebble to cobble sized rock rubble.

Post-Karoo dolerite occurs in the area as large sheets; sills and dykes. Dolerite deposits are extensive starting approximately 5 km north of the dam site. In its unweathered state dolerite is a dark grey, hard, hypabyssal igneous rock intruded into the host sedimentary rocks. No dolerite was encountered in any of the boreholes drilled along the centreline or spillway, however, boreholes were drilled in dolerite at the target quarry site, Q1, some 5 km distance from the dam itself along the R344 gravel road. Given its rather erratic occurrence dolerite can be expected to occur on a localised scale.

Seismic geophysics conducted at site revealed numerous palaeochannels situated in the mudstone bedrock below the dam centreline and borrow sites C6, D1 and D2. These palaeochannels are mostly aligned parallel to the current Koonap River channel and are inferred as old tributaries that would have once flowed into the river. An inferred fault plane was observed north of the left flank spillway and partially relates to closely to widely jointed sandstone retrieved from boreholes drilled at the site. The geological plan shows no indication of faulting, however, localised faulting is not uncommon and should be expected.

4.2.1 Site Seismic Hazard Appraisal

There are two areas of seismicity that typically need to be investigated to determine the likelihood, or otherwise, of seismic risk for a dam. The first is reservoir-triggered seismicity (RTS) whereby the additional hydrostatic pressure of reservoir impoundment and the weight of the dam triggers a seismic or several seismic events, and the second is to evaluate the predicted peak ground accelerations, based on the seismic hazard assessment, as determined from seismic history and tectonic stability of a particular area.

Reservoir Triggered Seismicity Risk Appraisal

Literature survey research and ICOLD Bulletin 137 (Reservoirs and Seismicity) (ICOLD 2011) indicates that it is well established that *large* dams can trigger earthquakes. Most RTS cases have been observed for dams over 100 m high – but even dams half those heights are also believed to have induced quakes. Reservoirs can both increase the frequency of earthquakes in areas of already high seismic activity or cause earthquakes to happen in areas previously thought to be seismically inactive.

As the height of the Foxwood Dam is well short of the description of typical dams which have promoted RTS activity, the risk of RTS for the Foxwood Dam is considered to be low.

The most widely accepted explanation of how dams cause earthquakes is related to the extra water pressure created in the micro-cracks; joints and fissures in the rock under and near a reservoir. When the pressure of the water in the rocks increases, it acts to lubricate faults which are already under tectonic strain, but previously prevented from slipping by the friction within the rockmass surfaces. With added pressure and fault lubrication the rockmass shifts with resultant earthquake or seismic results.

As there is no geological faulting mapped in the Foxwood Dam reservoir basin, this also suggests the RTS is low risk.

Seismic Hazard Evaluation

Foxwood Dam is located on the African Tectonic Plate which, in comparison with other tectonic plates, is stable with low movement - especially so when compared to other inter-plate obduction or subduction zones. Much of the Africa Plate and specifically the South African area can be considered to be a zone of ‘low tectonic activity’. This does not mean that this particular area is totally exempt of any seismic activity but rather that the risk is relatively lower.

Seismic Hazard of any particular area is represented by the peak ground acceleration: the higher the value the greater the risk of seismic activity. The *probable* peak ground acceleration for a particular area as based on a history of earthquake activity in that area. Such evaluation has already been undertaken for most parts of southern Africa (SANS 10160-4:2010). The higher seismically active areas are located in the gold mining zones of Gauteng and the Free State where seismic events are triggered, on occasion, through deep mining. Other higher category areas include the Ceres area of the Western Cape; southern Namibia (hot spot); parts of Lesotho (Katse Dam), and the southern and northern borders of KwaZulu Natal. A Seismic Hazard Evaluation of South Africa, as conducted by the Council of Geosciences, indicates a seismic hazard subdivision into zones varying in 'g' value ranging from less than 0,04g to a maximum of 0,24g.

The Eastern Cape Province has a general *low* acceleration value of 0,04g with the zone around Adelaide being approximately of 0,06g. This is a particularly low 'g' value which indicates that the Foxwood Dam area is in a low risk seismic area and therefore has a *low seismic hazard risk* potential. This is supported by the UNESCO (2007) Earthquake Risk in Africa assessment where this area falls into the lower earthquake intensity modified Mercalli Scale of I – V.

Therefore, the recommended seismic design parameters based on ICOLD Bulletin 72 (Selecting Seismic Parameters for Large Dams) and used in the stability analysis (see sections 7 & 8) are as follows:

- an Operating Basis Earthquake (OBE) of 0,05g;
- a Maximum Credible Earthquake (MCE) of 0,24g.

4.3 Dam site and basin

Geotechnical investigation shows that dam alignment presented in drawing number 225739-GEO-0602, (Preliminary Founding Level for Dam Wall) contains 3,0 m to 14,0 m depth of alluvial silt, sand and cobbles & boulders. This is underlain by competent mudstone and/ or siltstone rock. The bedrock underlying the alluvial sediment is shown to be weathered to depths as great as 24,8 m; in some cases highly weathered. The rock underlying weathered rock is only slightly weathered to unweathered and persists to the end of each borehole at an approximate depth of 30 m. For the purposes of the foundation stability analysis for the full extent of the dam foundation we have applied the rock parameters of the Balfour mudstone as given in Table 12.

Based on the above it has been taken that with a composite central river bed gravity spillway, excavations of typically 10 m to 17 m of alluvial sediments are anticipated.

Drawing 225739-DAM-0901 in Appendix F provides information on the expected founding depths for the concrete and embankment sections.

4.4 Construction Materials

Six nearby potential borrow pit locations were investigated with trial pits for potential earthworks materials. These are located as follows:

- Borrow Pit C2, C6 & C7 – on lower valley slopes 1 km to 3 km north of the proposed dam location, within the dam basin;
- Borrow Pit C3 – located immediately north of the dam basin
- Borrow Pit D1 – on the left valley side 200 m upstream of the proposed dam wall location, within the dam basin;
- Borrow Pit D2 – immediately downstream of the proposed dam location.

In addition, trial pits within the proposed embankment and gravity dam foundations were also undertaken. All locations showed a thickness of alluvial or colluvial materials which are likely to provide a high proportion of suitable embankment fill material.

The finer grained slightly cohesive sandy SILT material which is indicated by boreholes to be within the top 4 m to 5 m of alluvium should be selected for the central and upstream portions of the embankment. Below 4 m to 5 m depth, the alluvium increases in gravel and cobble content. This material would be suitable for placement in the downstream shoulder downstream of the chimney drain. A cut – fill calculation should be undertaken to assess the viability of this approach.

There is estimated to be more than sufficient appropriate material within the borrow pits and quarry site. Further information is provided in section 11 and full details of available volumes are provided within Section 8.5 of the Geotechnical Investigation Report (DWS 2015c).

There is unweathered dolerite from the quarry site (Q1) which has excellent durability and high strength quantities, making it suitable as concrete aggregate, riprap and fine aggregate. Finely crushed dolerite may also find use as filter material between the clay core and shell materials – this being the most reasonable acquisition of filter material - as no clean natural sands are available along the banks of the Koonap River. Sandstone must under no circumstances be used as riprap since experience shows that these rocks, where exposed, will undergo conchoidal fracturing; subsequent loss of riprap energy dissipation efficiency; and ultimately beaching along the upper upstream face.

4.5 Engineering assessment

The results of the investigation indicate that it is possible to construct a composite earthfill and concrete gravity dam provided that cognizance is taken of the following certain issues:

- The thick mantle of transported soil on the right hand flank of between 5 and 20 metres implies that special consideration will need to be given to foundations for the dam. Extensive excavation and backfill operation may be required, with the use of grouting;

- Material suitable for the construction of an embankment has been identified within borrow pits and under the dam centerline. However, it should be noted that there is a wide variability in quality and onsite selection of materials will be necessary during construction.

- There is an abundance of potential embankment fill material within borrow pits and the dam foundations. Rockfill material can be obtained from the excavation of the mudrocks and sandstone, however, given the depth of these materials the volumes will be limited. Spillway excavations will be the best choice to provide relatively high durability sandstone that will find use as rockfill/ ‘dirty’ rockfill material. Extensive quantities of earthfill material are available but these are potentially dispersive requiring gypsum stabilization.

- A Hard rock source for sand drain filters, concrete aggregate, rip-rap and fine aggregate is available at potential quarry site Q1, some 5 km north of the dam location.

- No natural clean sand was found on site, requiring crushing of dolerite to produce fine aggregate and filter requirements.

- The earthfill materials encountered within the borrow pits and under the dam centerline show variable potential for dispersive soils which will require detailed assessment during further design stages, if this material is to be used for the core. Properly designed and constructed filters adjacent to potentially dispersive material in the embankment is essential to prevent possible piping due to seepage.

- Thickness of compressible alluvial deposits presents a risk of differential settlement between earthworks and structures founded at different depths. Consideration should be given to construction phasing and / or localized removal of compressible soils.

- Relatively thin cover of alluvial deposits on the left hand flank, and rock jointing presents a risk of excessive seepage. Grouting of the foundations and abutment of the concrete gravity sections may be required.

A summary of the geotechnical design parameters of materials likely to be encountered within the reservoir basin, gravity and embankment dam construction are provided in the tables below.

Table 12: Balfour Mudstone design parameters

Design Parameter	Basis	Value	Unit
Unconfined Compressive Strength (UCS)	6 UCS tests (35,5 MPa to 210 MPa)	40	MPa
Phi'	Assumed from Tomlinson 7th Ed. Table 2.2	27°	
Cohesion, c'	Correlated from UCS & RQD (Tomlinson Section 2.3.6) to 4 MPa. Cripps and Taylor 1981 suggest lower bound of 2 MPa for Coal Measures Mudstone with UCS of 9 to 103 MPa.	2	MPa
Young's Modulus for Settlement	Correlated from UCS & RQD	300	MPa
Permeability	Correlated from Packer test Lugeon Values	3×10^{-6}	m/s
Permeability below 27 m depth	Correlated from Packer test Lugeon Values	1×10^{-7}	m/s
Grouted zone Permeability	Ciria C514, Section 6.3, lower limit permeability of rock mass grouting	1×10^{-7}	m/s
Bulk Density	From 6 UCS tests (25,6 kN/m^3 to 27 kN/m^3)	26	kN/m^3
Allowable bearing capacity	Using Tomlinson table 2.3 for cemented Mudstone.	4	MPa

Table 13: Alluvium design parameters

Design Parameter	Basis	Value	Unit
Undrained shear strength, Cu	Assumed with correlation from LL	90	kPa
Phi'	Correlated from Plasticity Index using BS8002 (c'=0)	31°	
Young's Modulus for Settlement	No data - assumed	5	MPa
Permeability	3 remoulded falling head tests	1×10^{-7}	m/s
Bulk Density	Average proctor compaction from 17 tests at centreline	17,5	kN/m^3

Table 14: Embankment earthfill design parameters

Design Parameter	Basis	Value	Unit
Undrained shear strength, Cu	Assumed with correlation from LL	90	kPa
Phi'	3 Triaxial Test (c' = 0)	32,5°	
Permeability	3 remoulded falling head tests	5×10^{-8}	m/s
Bulk Density	From 3 remoulded falling head tests	18	kN/m^3

Embankment Settlement	BS6031 Code of practice for earthworks	1% of Embankment height	

Table 15: Filter design parameters

Design Parameter	Basis	Value	Unit
Permeability	Requirement for filter design	1×10^{-4}	m/s
Bulk Density	Assumed on guidance from BS8002	21	kN/m^3
Phi'	Assumed	30°	

4.6 Recommended Further Investigations for Detailed Design Stage

The investigation undertaken at the site and the material properties described in the section above are believed to be sufficient for this stage of the project. Nevertheless as many of the input parameters into this analysis have been assumed or correlated, it is recommended to undertake a complementary ground investigation prior to the detailed design. It is also cautioned that a limited number of samples indicated varying degrees of dispersivity. Due care and site monitoring will be required during the construction phase.

More detailed geotechnical testing of embankment construction materials including strength, settlement characteristics, and dispersivity should be undertaken prior to detailed design. In-situ undrained strength of the colluvium / alluvium have been assumed and may be critical to embankment design and stability. In addition the following recommendations are made:

- Further in-situ permeability testing including falling head tests in the superficial deposits would help refine the seepage analysis and grouting requirements (see Section 9).
- The ground outside the dam centreline is not well defined by boreholes and assumptions have been made for stability analysis models. A wider spread of boreholes should be undertaken particularly at the upstream toe of the embankment where the stability analysis is sensitive to ground conditions. Further drilling of boreholes in a lattice over the spilling footprint and spill basin extent would help define the depth to suitable bearing hard horizon.
- The ground profile for valley stability sections are assumed as trial pits refused at depths less than 3 m. Recommend more boreholes upstream to confirm ground profile in reservoir basin for stability and seepage (see Section 8 and 9).
- The thickness of alluvium should be proved in greater detail outside the dam centreline and under the gravity dam section in order quantify the availability of suitable embankment construction materials in the foundation excavations.
- Settlement analysis undertaken for this design stage (see Section 8) uses assumptions on settlement characteristics of the soil. Consolidation and / or load tests are recommended to obtain a more accurate estimate of embankment settlement during detailed design

5 SELECTION OF FULL SUPPLY LEVEL & DAM TYPE

5.1 Dam types

Based on the geotechnical investigations and topographical survey carried out, the proposed dam site is suitable for the construction of a variety of dam types, namely an earth embankment dam (homogenous or with clay core), a rockfill embankment dam with clay core, or for a concrete gravity dam (or composite earthfill – gravity concrete). An earthfill or rockfill dam would require a side-channel spillway to be excavated in the left flank whereas a gravity concrete or composite dam would spill via a central gravity concrete spillway with flows returning directly to the river bed.

A process of option selection was carried out in July 2014 in conjunction with DWS to determine the most appropriate dam size and dam type to be developed at the site. A comprehensive Options Selection memorandum was issued to DWS for approval, containing information on the different dam types and associated benefits and risks. A copy of this memorandum is provided in Appendix K.

For dams of 1 MAR and below the cost of the structure and the resulting cost of water is dominated by the side-channel spillway excavation and is significantly more expensive than a gravity concrete or composite dam structure. The use of a side channel spillway also introduces inherent risks with regard to the ground conditions through the spillway section as well as the risk of erosion at the point of return to the river. For dam sizes above 1 MAR, the lowest cost of water is achieved with an earthfill / rockfill construction due to the material balance between the spillway excavation and embankment fill. These trends are illustrated in Figure 5 below which shows the comparative URV's calculated during the option selection process.

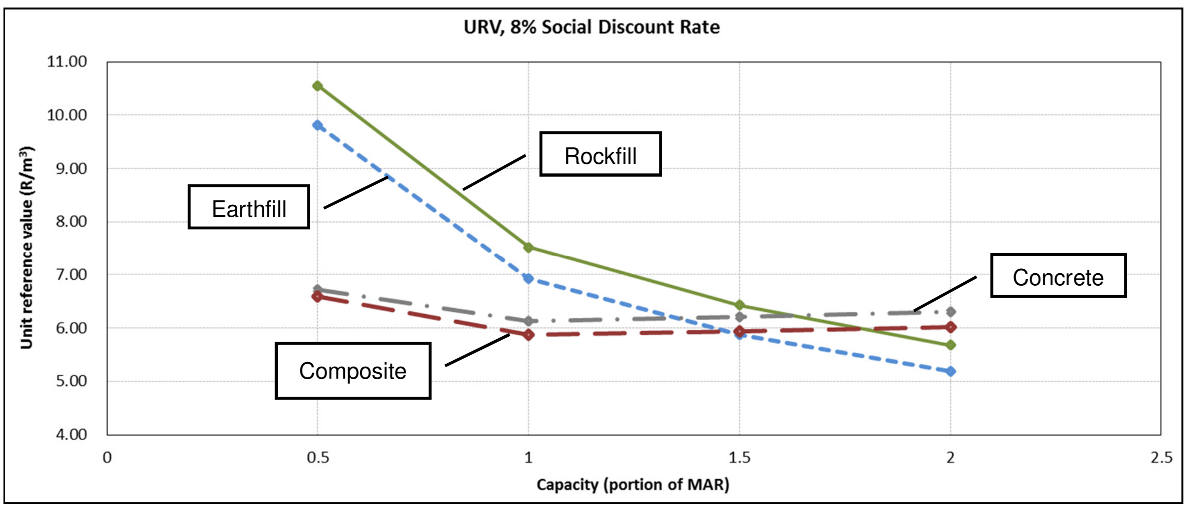

Figure 5: Unit Reference Value trends for 8% social discount rate

Given the high cost of water – independent of the proposed dam type – it is essential that maximum use of the yield is achieved. In addition, the environmental requirements of the river must be met.

5.2 Water Requirements

The domestic and commercial water requirements of the Adelaide and surrounding towns (Bedford and Fort Beaufort) can be met currently (and for the foreseeable future) by the existing water sources. The potential for the Koonap River water resource to be developed for a wider benefit (ie a managed supply into the Fish River) has also been discounted. The primary beneficiary of the potential water resource of the Koonap River will be new emerging farmers to be established downstream of the proposed Foxwood Dam as part of a Government Irrigation Scheme (DWS, 2015g). The development of such a scheme will require substantial capital and operational funding from government over a period of about 10 years to achieve sustainable socio-economic upliftment and will require the overcoming of complex institutional issues as well as the development of coordinated government policy. However, it is projected that the scheme would be able to payback the capital and operational funding required for establishment within approximately 11 years from the start of establishment and remain financially viable without further government subsidy for the farming operations. Yield from a 1 MAR dam would support the development of approximately 1 250 ha of new irrigation for high value tree crops downstream of the dam site in addition to meeting existing downstream irrigation water requirements.

5.3 Ecological Water Requirements (EWR)

The Koonap River Hydrology report (DWS, 2015a) notes that:
'The EWR operating rule recommended for the Foxwood Dam system is that high flow EWRs should be met by spills from Foxwood Dam and that the low flow EWRs can be met by inflows from the incremental catchments downstream of Foxwood Dam. This operating rule impacts the storage size of Foxwood Dam as it is important that regular spills can occur.'

For dam sizes above 1 MAR, the critical period increases (up to approximately 16 years) resulting in releases being required to satisfy high flow EWR scenarios and reducing the available long term yield of the dam. Comparison of the 1:20 year long term yield of a 1 MAR dam with an operating rule allowing for high flow EWR to be met by natural spills (long term yield = 19,1 million m^3/a) and a 1,5 MAR dam with an operating rule for high flow EWR to be met by releases (long term yield = 19,8 million m^3/a) indicates a small difference. In addition, designing the dam to naturally satisfy environmental flow requirements reduces the dependency on human intervention for the safeguarding of the ecological state of the river.

5.4 Recommended dam size

Given the institutional complexity of achieving optimum take up of the yield for a new Government Irrigation Scheme and the limited increased yields from larger dam sizes due to the need to provide releases to meet EWR high flow requirements, it was agreed with DWS on 5 August 2014 that a 1 MAR dam would be most appropriate for the Foxwood Dam site.

5.5 Recommended dam type

Based on the recommended dam size, the composite gravity concrete – earthfill embankment dam was recommended as the dam type as the cheapest structure. In addition, the concrete gravity spillway provides minimum risk in terms of returning spills to the river course and reduces risk due to geotechnical uncertainty on the side channel spillway ground conditions and possibility of dispersive nature experienced in some clay materials tested. The use of a gravity concrete spillway also includes other notable flood safety benefits such as:

- The PMF and RDF design floods are best catered for with a concrete gravity spillway.
- Outlet works are incorporated within the gravity structure to an elevation suitable for effective discharge into the river bed. The other options require free standing towers and tunnels at founding depths similar to the cut off foundation.
- Diversion is managed through the installation of a diversion culvert 4,0 m wide by 3,5 m high with its invert at the river bed level of 578 masl in the middle left of the spillway section. A 60 m wide section of the spillway must be kept 4 m lower than the rest of the spillway for the duration of the spillway construction.

The technical analysis carried out within this report is based on a 1 MAR composite gravity concrete – earthfill embankment dam. A rendering showing the dam feasibility design is provided in Figure 6 below. A drawing of the dam wall general arrangement and elevation of the dam wall illustrating foundation depths is provided in Appendix F.

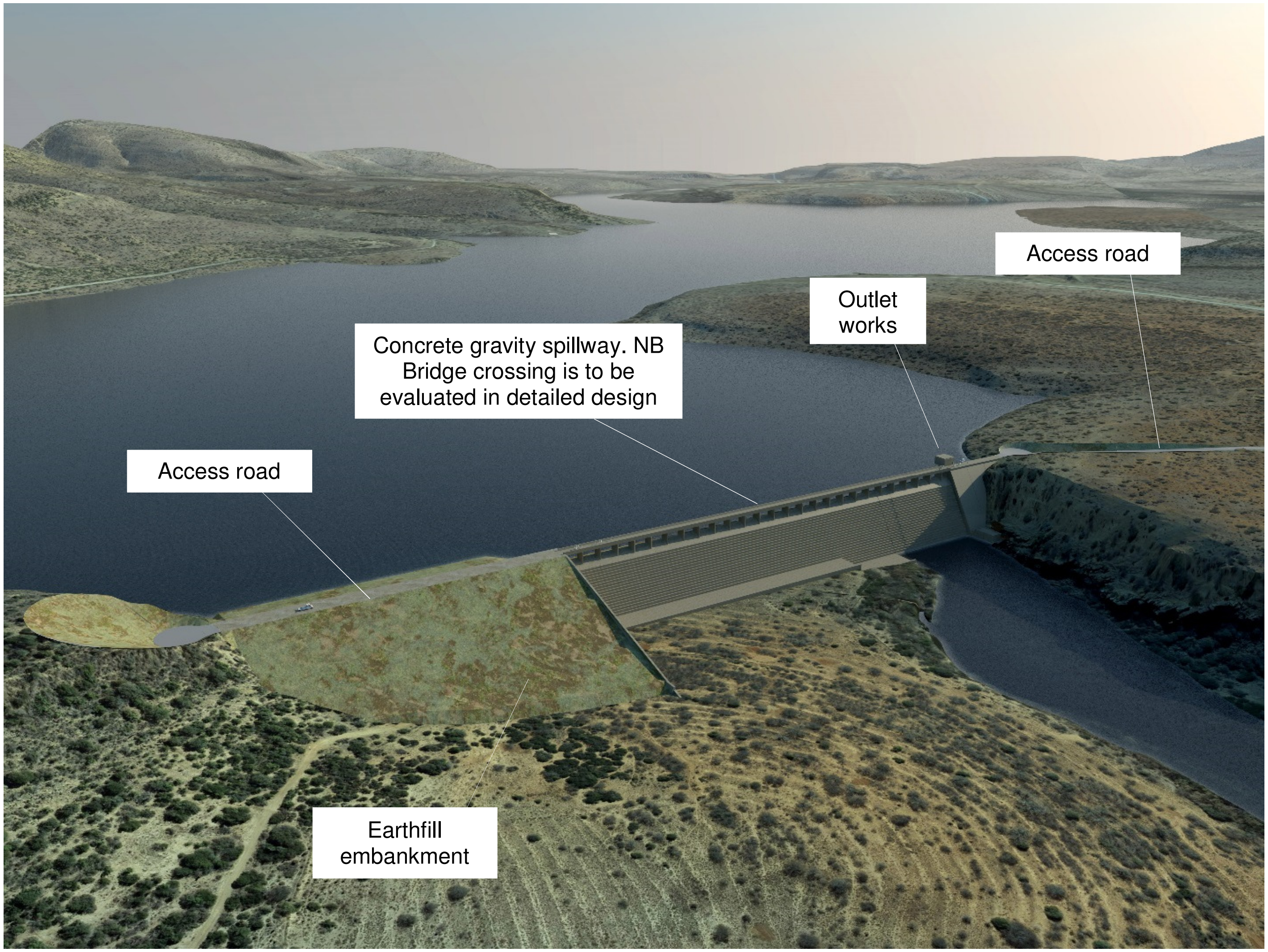

Figure 6: Render showing main structures of proposed Foxwood Dam

6 SPILLWAY DESIGN

6.1 Design philosophy

The spillway is designed to safely discharge excess flood water from the reservoir whilst maintaining the integrity of the dam and downstream valley.

6.2 Design criteria and freeboard

The dam is categorized as Category III, and as such is designed to:

- Pass the Recommended Design Flood (RDF) of 2 063 m^3/s (1 in 200 year flood event) with dry freeboard.
- Pass the Safety Evaluation Discharge (SED) of 6 200 m^3/s (PMF) without freeboard.

The design takes in to consideration the conclusions from the flood hydrology study and the recommendations from precedent reports.

This study has selected the PMF routed flood for the selection of spillway dimensions:
(See 3.5.3)

o Spillway length	250 m
o SEF discharge	5 218 $m^3/s/.,m$
o SEF freeboard requirement	5,5 m

The freeboard selection was undertaken in accordance with SANCOLD Guidelines on Freeboard for Dams Volume II.

6.3 Spillway selection

The spillway selection was based upon the above design criteria and the selected dam type. In the previous project stage (dam type selection) two options were considered:

- Side-channel spillway around the left abutment;
- Central overtopping section

The proposed dam type is a composite structure. The dam comprises a central concrete gravity dam, with an earthfill right abutment. The recommended spillway type is therefore a central overtopping section at the location of the concrete gravity dam. Benefits to this selection include:

- Smaller footprint of the dam site
- Reduced excavation of materials
- Greater opportunity for flow energy dissipation
- More economical solution

6.4 Proposed Layout

The spillway comprises three distinct elements: the spillway crest, the channel and the stilling basin.

6.4.1 Ogee Spillway Crest

The ogee spillway crest conveys any water above Full Supply Level (FSL) to the downstream face. The hydrology study (see section 3) adopted an ogee spillway crest length of 250 m.

However, since that study, a bridge spanning the overflow section has been introduced to the design. The assumed width of bridge pier is 0,6 m and the maximum bridge span is assumed to be 12,0 m (therefore 21 piers). To take account of the pier and abutment effects, as well as the width of the piers, the required length of spillway is 267,0 m. This has been determined using the method in Section 9.11 in *Design of Small Dams (USBR 1987)*, assuming round-nosed piers and square abutments.

An ogee weir shape with a coefficient of weir discharge of 2,0 has been selected whilst routing flood flows (see section 3). For the purpose of this feasibility study, the ogee shape is designed to the PMF head of 5,4 m. The profile is constructed of compound radii in accordance with Section 9.10 of *Design of Small Dams*. This is acceptable where the height of the ogee spillway crest is greater than one-half the design head. We note that optimisation of the designed flood return period for the ogee may consider the 1:200 year flood versus the PMF.

Savings in the cross-section area could be made if the ogee profile is determined using the following equation:

$$\frac{y}{H_0} = -K\left[\frac{x}{H_0}\right]^n$$

Where K and n are constants dependent upon the upstream inclination and velocity of approach. It is recommended that this is explored in further detail in the next stage of design.

The ogee spillway crest could be more economic if the profile is designed to a reduced head. The reduced head should be limited to 75 % maximum head to avoid cavitation risk. It is recommended that this is explored in further detail in the next stage of design.

The ogee spillway crest calculations assume a free discharge at all design flows. The position of the bridge, the level of the bridge soffit and any utility services should be carefully considered so that it they not affect the ogee spillway crest hydraulics. DWS have noted that this form of access is not favoured by the Department however it appears to be significantly less costly than an alternative road access cut through the right bank abutment. This will need to be considered in the detailed design stage of the project.

Ogee spillway crest details are shown on drawing 225739-DAM-1201 (Appendix F)

6.4.2 Downstream face

Flow over the ogee spillway crest is directed to the stilling basin via a stepped downstream face. Flow energy will be dissipated by the steps in the spillway. A step height of 1 200 mm has been selected for the following reasons:

- A larger step height is preferable for energy dissipation;
- A step height of 1 200 mm is considered a deterrent for persons attempting to climb the dam face.

The downstream face is sloped at 0,6H:1V (or 59 degrees). This is the maximum steepness determined from the stability analysis. This makes the step length 720 mm.

This is steeper than normally accepted 1:0.75 to 0.7, this is a function of the large spill basin blocks that were set to the ground level rather than the hard horizon.

The inception point of air entrainment is important to locate, as the un-aerated spillway portion could be prone to cavitation damage. During PMF flow conditions the theoretical inception point is past the toe of the dam due to a relatively large unit discharge. This suggests that the stepped spillway will not contribute to energy dissipation during the PMF event. However, the stability assessment discussed later in this report confirms that the dam itself remains stable under these conditions.

Spillway flow conditions have been checked for the 200 year flood (the Recommended Design Flood). The inception point is on the face of the dam, indicating that some energy dissipation will occur on the steps. However, a significant proportion of the stepped spillway is subject to un-aerated flow conditions. It is therefore recommended that aerators are considered in the next stage of design, upstream of the RDF inception point.

Using the method in Boes (Boes 2012), a side wall height of 2,4 m is recommended to contain the aerated PMF flow. Containing spillway flow is particularly important at the right abutment to protect the embankment dam. This recommended height should be reassessed if aerators are adopted later in the design.

See drawings 225739: -DAM-1201: - DAM-1002 and - DAM-1003 for the layout and details of the downstream face (refer Appendix F).

6.4.3 Stilling basin

Initial calculations indicated that a stilling basin length of 34,5 m was required to contain the hydraulic jump during a 200 year flood event. This would result in a large excavation at the toe of the dam. However, upon further investigation of the tailwater it was concluded that the tailwater is of a significant depth so that it greatly contributes to energy dissipation (see Figure 7 below). For example, at the 200 year flood event the depth would be in the region of 14 m.

It is understood that a rail bridge downstream of the site may contribute to the high tailwater level. A sensitivity analysis was undertaken by removing the bridge and assuming normal depth at the bridge model node. This yields a tailwater depth of 12,3 m during the 200 year event.

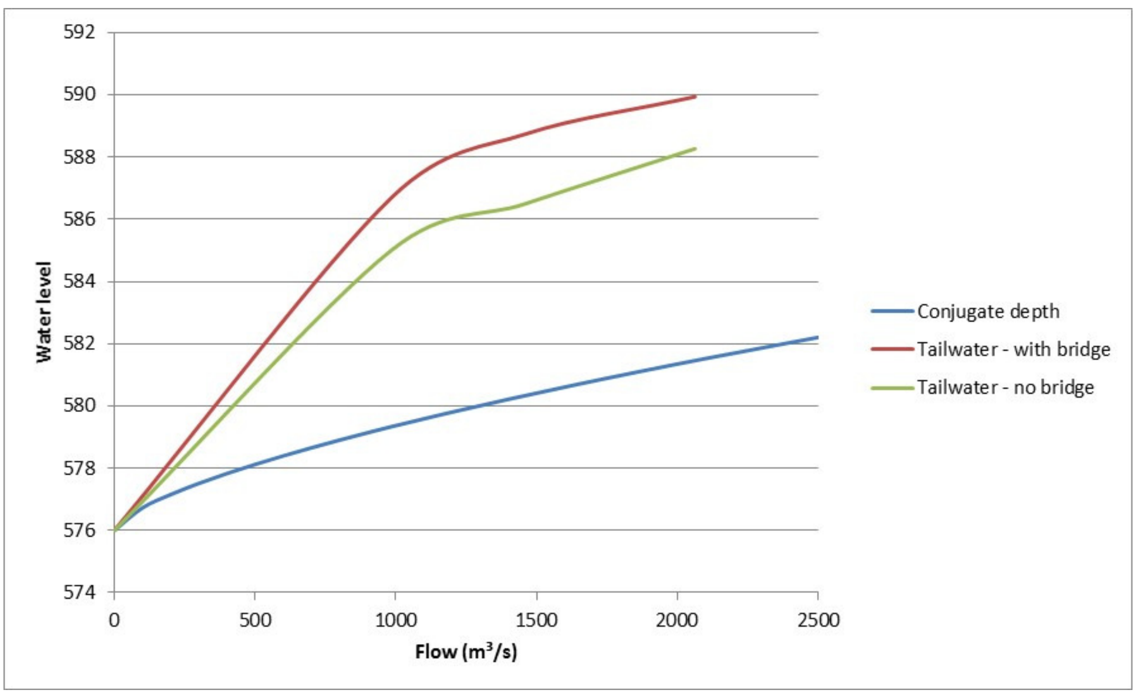

Figure 7: Modelled tailwater depths for different flows

The predicted tailwater depths are much greater than the theoretical conjugate depth of the hydraulic jump (predicted to be in the region of 5,5 m, in the RDF). Therefore the hydraulic jump in the basin should be contained within the tailwater.

The tailwater curve was developed in HEC-RAS. The tailwater curve at the location of the dam is given in Figure 7.

The toe of the concrete gravity dam has a 15 m long stilling basin block which is stepped to follow the ground level. The return is protected by a cascade system of graded large rocks and rip rap underlain by a crusher graded filter. Details of the stilling basin and downstream erosion protection are shown on drawings 225739: -DAM-1002,–DAM-1003 and DAM-1203 respectively. An extract from drawing 225739-DAM-1203 is given in Figure 8

During the design process the Department reviewed this aspect of the design and considered that the three tier basin was not sustainable and rather than rely on a cascading bolster system to return the flow to the natural river line, excavate out ‘fan’ like return. This will require further geotechnical investigation to determine more precisely the hard formation topography in the area to be considered and the optimal unitary level of the basin.

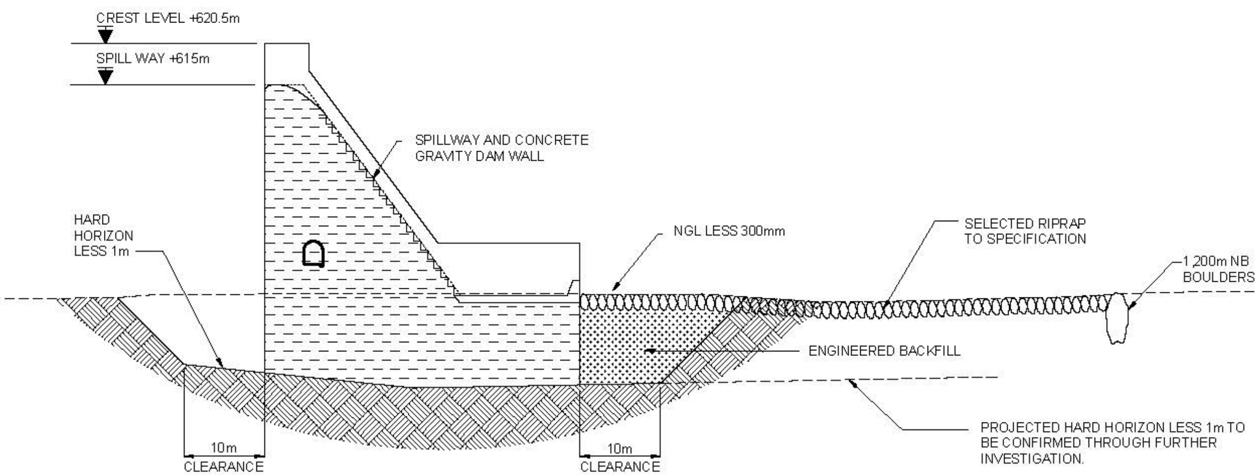

Figure 8: Section through still basin

6.4.4 Right Abutment

The right abutment is an L shaped retaining wall which extends into the embankment as a wrap around to allow the upstream surface slope to be 2 m below the FSL. The downstream wall extends to allow for retention and protection of the embankment and downstream toe.

During the design process, the Department reviewed this aspect of the design and considered that an L shape retaining wall would be more expensive than a full rap around retaining wall by extension of the gravity wall construction with approximate length 75 m. This will need to be considered in the detailed design phase. This may well lead to considering constructing the wall completely as a concrete gravity structure as opposed to composite.

6.5 Recommended future considerations for detailed design

6.5.1 Hydraulic Modelling

Due to the relatively large discharge, sensitivity of the embankment dam to erosion and the high tailwater depths, it is recommended that the spillway is modelled during the next phase of design in order to:

- Confirm the rating curve of the ogee spillway crest
- Determine the performance of the stepped spillway
- Confirm the height of spillway retaining walls
- Ensure the embankment is not subject to turbulent flow from the spillway and/or tailwater
- Determine flow velocity downstream of the dam to confirm the design of erosion protection

6.5.2 Other considerations

It is recommended that the following items be explored further during the next phase of design:

- A reduction in the design head for the ogee crest shape, whilst avoiding the risk of cavitation
- Establish the requirement for a spillway bridge access to the right bank crest
- Confirmation that the bridge design allows free discharge and optimization of span to spillway length
- Requirement for aerators on the spillway steps
- If the cross section of the gravity dam changes, the length of stilling basin should be revisited to ensure stability
- As per above the revision of the basin tiers to a single level and excavate out the return to the natural river line
- The determination and selection of flood method will need to be reviewed and the spillway length adjusted and optimised accordingly.

7 STABILITY ANALYSIS OF CONCRETE SECTIONS

The concrete gravity section of the dam is shown on Drawings 225739: -DAM-1201 and -DAM-1301. (Appendix F)

The stability analysis of this section has been undertaken using the load combinations and Factors of Safety in USBR Design of Small Dams (USBR 1987), presented below.

7.1 Loads

The following loads have been considered:

7.1.1 Dead load

Dead weight of the concrete gravity dam. Superstructures such as bridges can be included in the dead load, however, in the absence of a fixed bridge design this has been omitted from the analysis. This is seen as a conservative assumption, and may lead to cost savings later in the project.

The unit weight of concrete is taken to be 24 kN/m^3.

7.1.2 Hydrostatic load (reservoir)

In the usual and extreme load cases (see below) the hydrostatic (water) level is taken to be at Full Supply Level (FSL).

Further to the hydrostatic load, the structure is also subject to uplift pressures across 100 % of the base. Considering the foundation rock as fractured mudstone with a permeability of 10^{-6} m/s, the uplift pressure is assumed to respond instantaneously with reservoir level. There is a drainage gallery located near the upstream face. The presence of the drainage gallery (with vertical drains in to the foundation) means that there can be a reduction in uplift near the upstream face. For this stage of the design a drainage effectiveness of 50 % will be assumed in accordance with USACE (1995).

7.1.3 Hydrostatic load (tailwater)

A hydraulic jump stilling basin is provided to dissipate the energy of the flow in the spillway. The action of the hydraulic jump will push the tailwater downstream of the dam. Therefore a restoring tailwater hydrostatic load will not be considered here.

7.1.4 Hydrostatic load (flood)

In the unusual load case, the hydrostatic (water) level is taken to be at Maximum Water Level (MWL) generated by the PMF. This is noted to be slightly conservative as the water flowing over the ogee spillway crest will be velocity head rather than static head. With water flowing over the section there will be a tailwater. This will be considered for uplift calculations, however it is assumed that the tailwater does not offer a restoring force as it is being used in the energy dissipating process (USBR 1987) Uplift is assumed to respond instantaneously to reservoir level, therefore the uplift pressures are increased. Due to the tailwater the pressure distribution is trapezoidal. The other uplift assumptions are still applicable.

7.1.5 Earth pressure

If there is no significant tension at the downstream toe then the deflections will be sufficiently nominal to not cause additional strain in the downstream soil mass. Therefore at-rest earth pressure coefficient is deemed appropriate. In the unusual case, the soil mass at the downstream toe is assumed to be eroded away due to overtopping of the dam (a conservative assumption considering the provision of riprap).

7.1.6 Silt load

The anticipated accumulation of silt over the design life of the reservoir is 6 million m^3. From the reservoir storage curve, this represents a nominal depth. Therefore for the purposes of the stability analysis a greater depth of 1 m is chosen.

7.1.7 Seismic load

Based on section 4, the peak ground acceleration for the Maximum Credible Earthquake (MCE) is taken as 0,24g. In line with Section 6 of BRE *An Engineering Guide to Seismic Risk to Dams in the UK (BRE 1991),* this has been reduced by 2/3 for the horizontal load, and by a further 1/2 for the vertical load.

7.1.8 Temperature

In the absence of construction/expansion joint details, the stresses generated due to the volumetric change of concrete following temperature rise have not been considered. However, it must be a consideration in future design phases.

7.1.9 Ice

It is anticipated that no ice load shall be present at this site.

7.2 Load combinations

Stability assessment has been carried out in accordance with USBR guidance. DWS have noted that additional load cases should be considered during detailed design in accordance with the requirements of Directorate Civil Engineering. The following load combinations have been considered in the feasibility study:

Table 16: Load combinations for concrete section stability analysis

Load	Usual	Unusual	Extreme
Dead load	Yes	Yes	Yes
Hydrostatic load	Yes – FSL	Yes – MWL	Yes – FSL
Silt	Yes	Yes	Yes
Earth	Yes	Yes (not at downstream toe)	Yes
Uplift	Yes	Yes	Yes
Earthquake	No	No	MCE

7.3 Stability criteria

The analysis considers the global stability of the structure as a whole. There is no obvious weak point (change in section) or details of construction joints, therefore at this stage of the design intermediate failure plans within the body of the dam shall not be considered. However, this must be considered in later design stages. It is assumed the dam may fail by sliding (along the concrete-rock interface), overturning (about the downstream toe), or insufficient bearing capacity. The compressive strength of the concrete is also checked.

7.3.1 Sliding

The following minimum factors of safety are required against sliding failure:

Table 17: Load combinations for concrete section stability analysis

Material/Interface	Usual	Unusual	Extreme
Concrete/rock interface	3,0	2,0	>1,0

7.3.2 Overturning

No tensile capacity is permitted in the concrete and rock. In order to meet this criterion, the resultant location should be:

Table 18: Required resultant location for overturning

Load combination	Location of resultant force
Usual	Within middle ⅓
Unusual	Within middle ½
Extreme	Within base

7.3.3 Foundation failure

Foundation bearing pressure should be:

Table 19: Required foundation bearing pressure

Load combination	Foundation bearing pressure
Usual	< allowable
Unusual	< allowable
Extreme	< 1,5 allowable

7.3.4 Concrete strength

Allowable compressive stress should be:

Table 20: Allowable concrete compressive strength

Load combination	Allowable compressive stress
Usual	0,33f’c (FoS 3,0)
Unusual	0,5f’c (FoS 2,0)
Extreme	1,0f’c (FoS 1,0)

In the above, f’c is the characteristic compressive strength of the concrete at the section being considered. For this analysis f’c is taken as 40 N/mm^2.

7.4 Design parameters

In addition to those discussed above, the following design parameters have also been adopted:

Table 21: Concrete dam design parameters

Parameter	Assumption
Dam crest level	620,4 m
Spillway crest level (also FSL)	615,0 m
Silt level	578,0 m
Existing Ground Level	577,0 m
Foundation level	572,5 m
Unit weight of earth fill	18 kN/m³
Unit weight of water	10 kN/m³
Angle of friction (fill)	32,5 degrees
Unit weight of silt	17 kN/m³
Angle of friction (silt)	20 degrees
Allowable bearing capacity of rock foundation	4 000 kN/m²

7.5 Stress and stability analysis

Limit state analysis was undertaken on the overflow section of the concrete gravity dam, along with the upper part of the non-overflow section (that part greater than 615,0 m).

As discussed in Section 6.4, the concrete at the toe of the dam is required for both stability and erosion protection. A minimum length of 15 m is required if the downstream face of the dam is 0,6H:1V.

7.6 Results

The results are presented in Appendix A and summarized below:

Table 22: Usual case

	Acceptable	Calculated
Sliding Factor of Safety	3	9,96
Resultant location	Middle third	This is acceptable
Foundation bearing pressure	< allowable	This is acceptable
Maximum compressive stress	0,33f'c	This is acceptable

The base is in compression.

Table 23: Unusual case

	Acceptable	Calculated
Sliding Factor of Safety	2	7,67
Resultant location	Middle half	This is acceptable
Foundation bearing pressure	< allowable	This is acceptable
Maximum compressive stress	0,5f'c	This is acceptable

The base is in compression.

Table 24: Extreme case

	Acceptable	Calculated
Sliding Factor of Safety	1	6,58
Resultant location	Within base	This is acceptable
Foundation bearing pressure	< 1,5 x allowable	This is acceptable
Maximum compressive stress	1,0f'c	This is acceptable

The base is in compression.

7.6.1 Additional load scenarios

Two additional load scenarios were considered:

- Usual with full uplift – simulating blocked drainage. This met the design criteria.
- Unusual with no hydrostatic load – simulating end-of-construction. This met the criteria except the base is not 100 % in compression. However in this scenario it is the toe which is in tension which is considered acceptable.

7.7 Conclusions

The concrete gravity dam has been checked for global stability using the load combinations and Factors of Safety recommended in USBR Design of Small Dams. In all cases the dam performs satisfactorily. If, during the next stage of design, and geometric or material amendments are made, the global stability will need to be reassessed. In addition, once the construction technique is confirmed the stability at intermittent stages will also need to be evaluated.

Recommendations for further ground investigation prior to the undertaking of detailed design which take into account the above analysis are included in Section 4.

8 STABILITY ANALYSIS OF EMBANKMENT SECTIONS AND SETTLEMENT

8.1 Embankment details

Figure 9 below illustrates the main components of the earthfill embankment section.

8.1.1 Embankment Cross Section

Analysis of particle size tests indicate that material from proposed borrow pits and beneath the dam alignment is a sandy silt with clay. Recompacted laboratory permeability tests indicate a permeability on the order less than 1 x 10^{-8} m/s can be achieved. It is therefore proposed to construct an earthfill embankment using site won alluvial / colluvial material with selection of lower permeability fill in the core and a chimney drain incorporated (See Drawing No. 225739-DAM-1002). Figure 9 below illustrates the key structure of the earthfill embankment section.

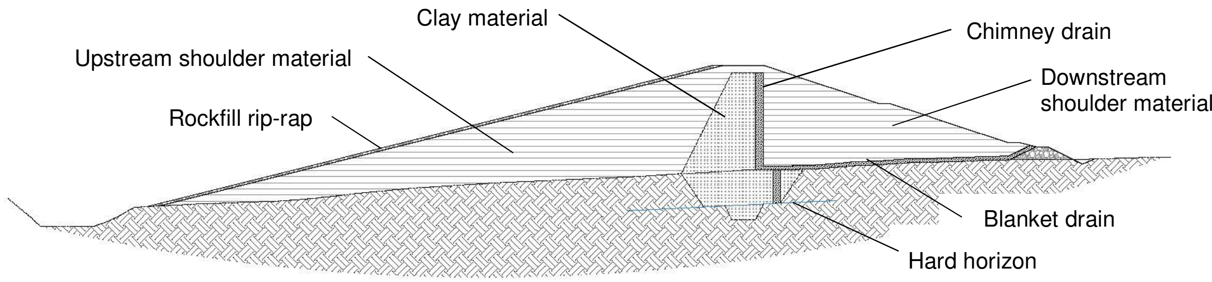

Figure 9: Earthfill embankment section

The design of the earthfill embankment relies on the internal chimney and blanket filter drain. The granular filter material prevents the migration of fine particles out of the central core from prolonged seepage and facilities the drainage of the downstream shoulder. Requirements for particle size of the filter is likely to be relatively narrow, and therefore supply may need to be from crushed rock.

Protection of the upstream face of the embankment from wave damage and erosion is provided by a layer of rip rap. The rip rap is required to be a coarse angular sound rockfill, placed on a finer granular bedding layer. The Site Investigation report (DWS 2015c) suggests this material may be sourced from the nearby dolerite quarry.

The embankment can be considered to have the characteristics of an earthfill embankment at this stage. Information on material specification is included in Section 11. It is recommended that the material properties are re-assessed during the final design stage, and that the material properties used for the purpose of this feasibility design be used and/or accepted with caution. In particular the risk from dispersive soils within borrow pit materials should be investigated fully.

The general cross section details for the embankment are provided in Table 25 below

Table 25: General embankment cross section details

Detail	Value
Surface strip	300 mm
Crest width	10 m
Crest level	620,5 m
Upstream slope	1 in 4

Detail	Value
Downstream slope	1 in 3 slopes between 3 step berms
Core crest width	6 m
Core slope	1 in 0,5
Main Core Trench depth	To hard horizon (notionally 8 m)
Additional Core Trench depth	4 m below hard horizon
Core Trench slopes	1 in 0,67
Grout curtain depth	27 m below ground level
Filter drain thickness	2 m

8.1.2 Ground Model and Design Profile

Boreholes and trial pits along the embankment alignment indicate the strata consist of sandy and clayey SILT alluvium / colluvium overlying bedrock. Borehole descriptions indicate the alluvium increases in cobble and boulder content with depth, however no geotechnical testing has been undertaken at depth and for feasibility design no separate unit has been included. Bedrock along the majority of the embankment alignment is Balfour Mudstone, apart from the left abutment which is indicated to be Sandstone above approximately 605 m elevation.

Embankment section design profile:

- Alluvium to 10,4 m depth (at the Section location of highest embankment), overlying;
- Balfour Mudstone.

Concrete Gravity design profile for seepage analysis:

- Alluvium to 2 m depth, overlying;
- Balfour Mudstone

8.1.3 Material Properties

Refer to Section 4 for tables of material design parameters.

8.2 Cross sections analysed

The section analysed is the highest embankment section at the interface with the concrete gravity section. Since the analysis, the length of the concrete spillway has increased, and the length of the embankment section has decreased. The height of analysed section is therefore slightly higher than is proposed by the feasibility design, however this difference is considered slightly conservative.

The existing ground level under the centerline of the embankment section analysed is 589,1 m.

8.3 Cases investigated and stability criteria

Embankment dam stability of the upstream and downstream slopes has been assessed under the following conditions:

- Maximum Water Level (MWL) at 620,4 masl – steady seepage; u/s and d/s slopes
- Rapid Drawdown from Full Supply Level (FSL) of 615 masl to 580 masl, u/s slope
- End of construction, u/s and d/s.
- Seismic analysis will be carried out at MWL for an Operating Basis Earthquake (OBE) of 0,05g and at FSL for a Maximum Credible Earthquake (MCE) of 0,24g.

The global factors of safety have been adopted in accordance with USBR Guide – Chapter 6, are shown in Table 26.

Table 26: Factors of Safety for embankment design

Loading condition	Minimum Factor of Safety
Steady seepage with reservoir at MWL (u/s and d/s)	1,5
Rapid drawdown (u/s)	1,3
End of construction (u/s and d/s)	1,3
Seismic-OBE (u/s and d/s) Seismic MCE (u/s and d/s)	>1,1 <1, with allowable displacements

8.4 Stability calculations

Stability analysis has been undertaken in Oasys Slope Version 19.0 SP3 build 26, using Bishop method of analysis. Global Factors of Safety have been used to allow the minimum factor of safety in line with Section 8.3 to be recorded.

8.4.1 Drawdown analysis input

As per guidance within Chapter 11 of Geotechnical Engineering of Dams (Fell et. al. 2005) the drawdown state has been analysed assuming instantaneous drawdown with phreatic surface close to the embankment surface.

8.4.2 End of construction analysis input

Fell et. al. also suggest end of construction can also be assumed for undrained conditions to be acting. As the granular filter layers should be free draining pore pressure should not build up. A nominal Ru value of 0,2 has been assumed for these layers.

8.5 Findings and results

Full results are presented in Appendix C. The initial embankment geometry analysed included steeper slopes (1 in 3 upstream, and 1 in 2,5 downstream) which failed to meet the required factor of safety under the following conditions:

- Drawdown on the upstream slope FoS 1,24
- Max Credible Earthquake on the downstream slope FoS of 0,97,
- Undrained end of construction on upstream FoS 1,01.
- Undrained end of construction on downstream slope FoS 1,19.

The embankment slopes were subsequently slackened to that shown in design drawings and the following sections discuss this design:

8.5.1 Critical case at end of construction

These results show end of construction is the critical for both upstream and downstream slopes. These were initially analysed with *Cu* of alluvium at 70 kPa increasing with depth due to the borehole log descriptions showing increase cobble content and lower fines with depth. Due to the method in which Oasys Slope increases *Cu* from a set elevation, it was difficult to ensure the equivalent strength was being measured on the upstream and downstream alluvium. Therefore an average *Cu* of 90 kPa has been assumed for the alluvium. Correlations from plasticity tests indicate this is feasible, however due to its sensitivity to embankment design, a more detailed field and laboratory testing of undrained strength of alluvial deposits should be undertaken in future design stages.

The proposed embankment geometry to achieve the required factor of safety for end of construction is shown in Figure 10. The different slope angles for upstream and downstream slopes is due to the fact that that the natural ground surface is higher by approximately 13 m at the downstream side. This higher ground is present across the entire embankment length. The presence of the frictional filter layer is also contributing to stability on the downstream face. The filter layer will also help reduce pore pressure on the downstream side during construction, although this is not been taken into account in this analysis.

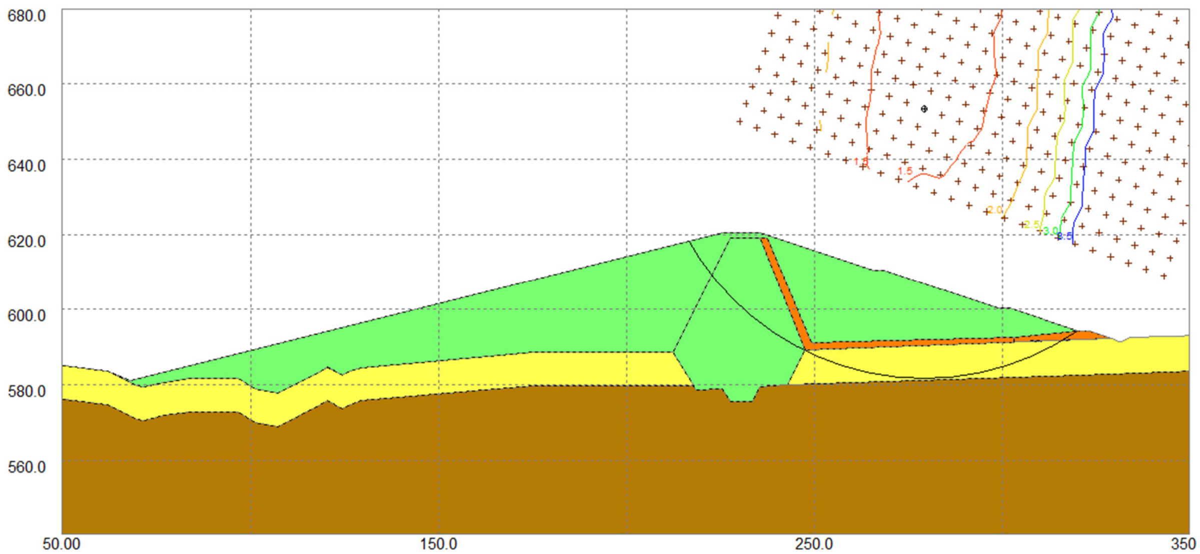

Figure 10: Final Slope geometry used – 1:4 upstream, 1:3 downstream slopes (Run 14 in Appendix B)

8.5.2 Seismic stability analysis

Seismic analysis in Oasys slope uses pseudo-static analysis incorporating a percentage of g applied as a horizontal acceleration to the soil mass. The proposed embankment profile achieves adequate factors of safety during an OBE, however during an MCE the upstream slope achieves a factor of safety of 0,74 (Figure 11). Factor of safety less than 1 is acceptable as long as horizontal displacements are small and catastrophic failure is unlikely to occur. The BRE Report 187 *An Engineering Guide to Seismic Risk to Dams in the United Kingdom* (1991) quotes the following assessment by Ambraseys:

Log u = 2,3 – 3,3 (k_c/k_m) for 0,1 < (k_c/k_m) < 0,8
Where u = downslope displacement in cm
k_m is the design seismic coefficient
k_c is the critical seismic coefficient to reduce FoS to 1
For Foxwood, Log u = 2,3 – 3,3 (0,16/0,24)
u = 1,3cm

Therefore negligible displacement is anticipated to occur. This value of downstream displacement is considered acceptable in relation to likely failure geometry shown in Figure 11.

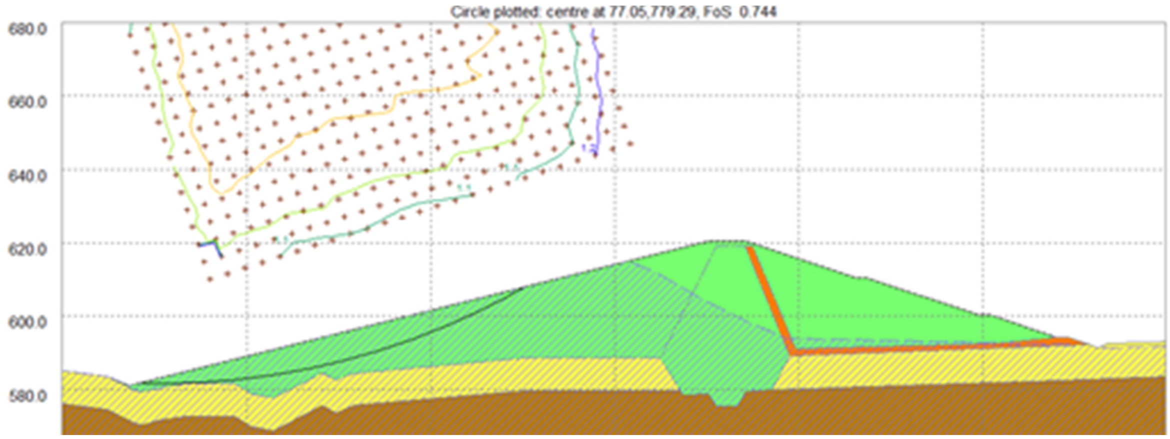

Figure 11: MCE on 1:4 upstream slope (Run 18 in Appendix B)

8.6 Valley Slope Stability Analysis

Two locations of natural valley slopes in the reservoir basin have been analysed for stability on rapid drawdown of the reservoir. No pre-existing instability of the natural ground at the site has been identified. The Section locations shown in Figure 12 were chosen for their steepness and proximity to the dam location.

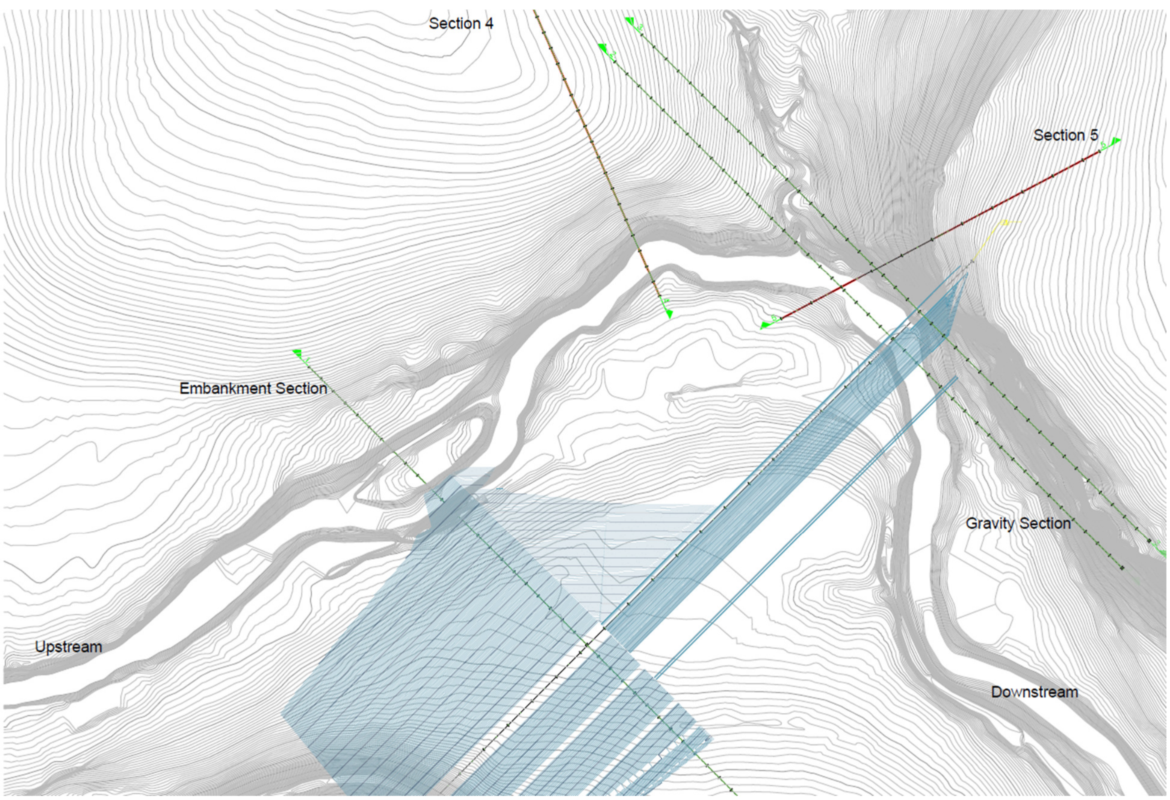

Figure 12: Section locations

Section 4 showed small slips close to river which would occur when the reservoir is drawn down over 25 m depth. These are not considered a risk to dam stability.

Section 5 shows slips with factor of safety less than the target of 1,3 occur in the steep valley side (Figure 13). This is a location where the ground profile has not been well defined by the ground investigation, although nearby borehole BH6A and BH7A indicate up to 1,5 m of colluvium over bedrock. It is likely the potential for these shallow slips to occur is a low risk to the dam. It is likely this risk may be ruled out by further ground investigation at detailed design stage.

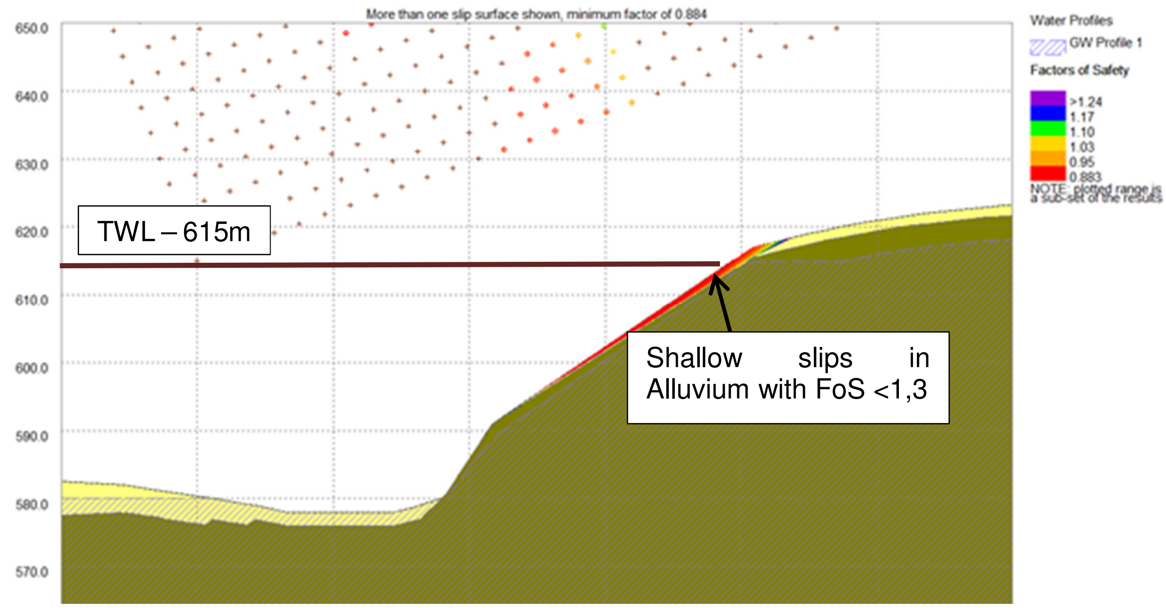

Figure 13: Section 5 Rapid drawdown (Run 31 in Appendix B)

8.6.1 Summary of Stability modelling results

A summary of the stability load cases and their results is included in Table 27 below:

Table 27: Factors of Safety achieved in embankment design

Loading condition	Location	Required Factor of Safety	Minimum Factor of Safety Achieved
Steady seepage with reservoir at MWL	u/s	1,5	2,5
	d/s		1,9
Rapid drawdown	u/s embankment	1,3	1,4
	valley slopes		**0,9***
End of construction	u/s	1,3	1,3
	d/s		1,3
Seismic-OBE	u/s	>1,1	1,73
	d/s		1,66
Seismic MCE	u/s	1,0 with allowable displacements	**0,74**
	d/s		1,02

* shallow slips in area with poorly defined ground profile – further investigation recommended

The above table shows that the minimum required factors of safety were achieved in all loading conditions apart from the following:

- Stability of the natural valley slopes within the reservoir basin on rapid drawdown. A factor of safety of 0,9 was achieved, however the ground profile was poorly defined in an area of steep ground. It is recommended further investigation is undertaken at the detailed design stage.
- Stability of upstream slope of the embankment during MCE. Factor of Safety of 0,74 was achieved, however the likely displacements were considered tolerable.

8.7 Settlement

Settlement analysis has been carried out in order to establish the scale of settlement expected around the embankment and concrete gravity dam interface. Modelling was undertaken in Oasys pdisp. The model assumes the following parameters:

- 8,8 m thickness of Alluvial deposits overlying Mudstone
- Alluvial Deposit properties of Cu=90 kPa, Eu=5 MPa as per Section 4
- Mudstone properties of Eu=300 MPa
- Embankment loading associated with height of 31 m of $\gamma = 18$ kN/m^3
- Gravity dam loading associated with height of 31 m of $\gamma = 24$ kN/m^3.

Full results are presented in Appendix D. The results show that total settlement of foundation soils in the order of 600 mm may be expected due to embankment loading. In addition 1 % of internal self-weight settlement of the embankment may be anticipated equating to approximately 900 mm of total embankment and foundation settlement. Approximately 50 % of settlement may be anticipated to occur during construction (conservative assumption based on Burland et.al. 1978), and therefore 450 mm of post construction settlement may be anticipated if the alluvium were to remain in place.

Settlement of the concrete gravity dam is limited to approximately 50 mm due to its founding at depth within the mudstone.

The location of the highest anticipated settlement is close to the interface with the concrete gravity dam. Should the concrete wall elements be constructed prior to earthworks, the embankment soil would move the total amount adjacent to the concrete wall. It is therefore recommended that the retaining walls at the interface are founded on mudstone and that alluvium is excavated and replaced under the embankment footprint close to the interface. The transition to founding the embankment on natural ground should be graded at a minimum of 1:3 and benched to reduce differential settlement. The interface settlement could be minimized if the embankment is constructed first.

An additional camber of approximately 450 mm is recommended for design due to anticipated long term settlement. The upper portions of the embankment slopes are commonly steepened to allow for settlement back to the design profile. The critical case for stability of the embankment is indicated to be undrained construction and therefore the detailed design should ensure the short term stability of the embankment is not compromised. The recommendations in terms of instrumentation in Section 14.2 should be noted.

It should be noted that no consolidation testing of the alluvial deposits is available and settlement characteristics have been estimated. The values of settlement should be properly evaluated during the final design stage and based on sufficient and credible laboratory test results.

8.8 Recommendations

Recommendations for further ground investigation prior to the undertaking of detailed design are included in Section 4. The outcome of the stability and settlement calculations also has implications for instrumentation monitoring which is included in Section 14.2.

9 SEEPAGE ANALYSIS

Seepage analysis has been undertaken in order to establish the amount of seepage which may occur through the embankment dam, and through the embankment and gravity dam foundations, and the extent of grouting works required. Steady state seepage has been carried out to a steady state analysis at Top Water Level using the Geo-slope software SEEP/W. Full seepage analysis is presented in Appendix E.

9.1 Embankment seepage analysis

Seepage through the earthfill embankment profile appears to be controlled by the internal chimney drain. Seepage and hydraulic gradients appear acceptable for the proposed embankment geometry. The inclusion of a grout curtain beneath the key trench appeared to have limited effect on seepage analysis (Figure 14). This is on the basis of homogeneous flow conditions through the rock.

However, borehole logs indicate the risk of connected permeable discontinuities may exist through the rock. High Lugeon values were recorded up to depths of 27 m within boreholes, and therefore a grout curtain up to 27 m below ground level is recommended for the embankment section and right hand side abutment.

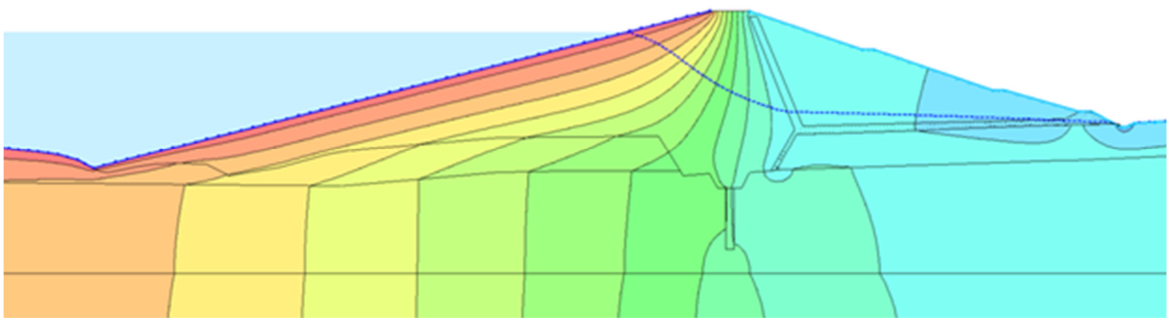

Figure 14: Results of Seepage analysis Run 8 in Appendix E – total head

9.2 Concrete gravity section seepage analysis

Seepage analysis under the concrete gravity dam close to the left abutment indicate that a grout curtain to approximately 27 m depth is required to reduce the hydraulic exit gradient downstream (Figure 15). This depth is shown by low Lugeon values in borehole BH3A and BH5A at depth.

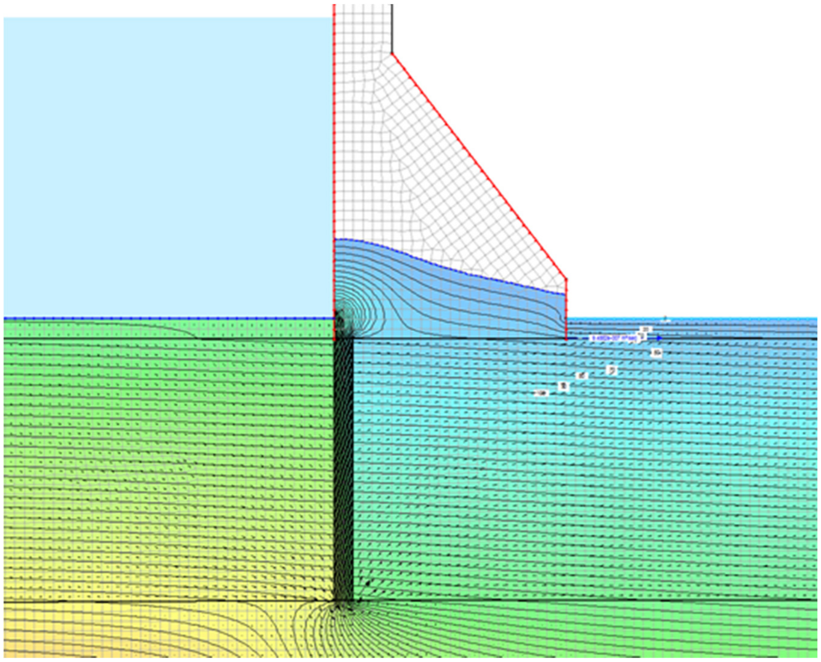

Figure 15: Results of Seepage analysis Run 11 in Appendix E – Pore pressure contours (kPa)

9.3 Seepage analysis conclusions

The results of seepage analysis show that the following will need to be considered within the design:

- An internal chimney and blanket drain is required to reduce the elevation of seepage through the embankment dam and at the toe
- A cut off trench and grout curtain are likely to be required to reduce the risk of seepage through the alluvial soils and weathered bedrock beneath the embankment dam
- A grout curtain is required to reduce seepage pressures beneath the concrete gravity dam and the left hand side abutment.

Recommendations for instrumentation and monitoring of seepage is included in Section 14.2.

10 OUTLET WORKS

The outlet works have been designed to make provision for discharge of the anticipated maximum environmental water requirements (6 m^3/s) and all downstream off takes and to ensure that with multiple level off takes adequate the water quality is maintained. The pipe design constraint was set at flow velocities < 8 m/s. Consequently there limit is not generally the hydraulic gradient. The velocity in the system can be increased but there is a concern that the bottom discharge sleeve valves may be subject to vibration at higher velocities.

10.1 Layout

The outlet tower is located in the concrete gravity left abutment, which allows for conventional concrete construction methods to be carried out independently of the bulk concrete in the spillway gravity section. Refer to the outlet work drawing in Appendix F.

The outlet works are designed with a twin stack system to allow for 100% redundancy for maintenance purposes and to make provision for discharge of the maximum EWR (6 m^3/s) The outlet works also provide multiple downstream off takes to ensure adequate water quality is maintained in the discharges. Although the individual stacks can discharge the full discharge with a pipe design, a constraint was set at flow velocities < 8 m/s. It is recommended that in order to deliver the EWR that both stacks be used. This will result in < 4 m/s flows.

- Irrigation water is released to the river for run-of-river abstraction downstream of the dam.
- The irrigation peak requirement for the approximate 1 600 ha of downstream irrigators (existing and proposed) should not exceed 2 m^3/s.

Access to the upstream tower is off the wall crest via the main access road to the dam wall. The gate house includes a gantry crane to operate the service gates, screens and facilitate maintenance of isolation valves and pipework.

The location of the instrument chamber will be finalized during detailed design and could be accommodated within this secured building. This tower will also provide access to the drainage gantry and the valve chamber. The valve house chambers can also be accessed by a stair way down the left bank non-overflow section. Machinery and equipment that cannot be removed through the inlet tower can be winched up a railed skid ramp up the same slope adjacent to the stairway.

The lowest intake level is based on the estimated sediment level after 50 years (586 masl) with the minimum operating level proposed at 590 masl. This estimate should be reviewed as part of the sedimentation review at detailed design as it has been suggested that, in the absence of a detailed investigation, the stability analysis should include for a third height deposition of silt on the wall face.

10.2 Intake Tower

There is a dual system pipe work in the intake structure includes multi-level intakes at different levels, with butterfly isolation valves at each intake structure for selecting the level at which water is to be drawn off. The minimum operating level will be approximately 590 masl. The intake tower consisting of dry and wet chambers is required for maintenance. The intakes are protected with precast concrete trash racks and fine screens to prevent blockage by floating debris. An emergency gate is required for closure for maintenance purposes at the bell mouth entrances. The tower intake chamber includes a motorised gantry and crane for removal of equipment and the raising and lowering of the emergency gate.

On the recommendations of the water quality report (DWS 2015b) the intake tower has 4 x Ø1,0 m off takes. The first is at 5 m below FSL with the second, third and fourth offtakes at further drops of 7,5 m, 7,5 m and 7,0 m. The lowest intake is approximately 10 m above the lowest riverbed level and less than 1 % of the total storage. The requirement for 4th offtakes may not be required, as it could well have to be blocked off in the future. The next highest off take is approximately at 7 % of total storage. This should be finalised in the detailed design stage as the 4th offtake may have application in the initial filling.

10.3 Valve chamber and river outlet

The valve chamber is situated in the toe of the left abutment with the floor of the operating chambers being 8 m above natural riverbed. This is estimated to be in the 1:30 return period range at ± 750 m^3/s.

The river outlets provide a maximum discharge of 6 m^3/s via Ø 1 000 mm intake pipes. The discharge is controlled through 2 x bottom discharge sleeve valves which can be isolated with similar sized butterfly valves. The stilling wells need to be designed to the manufacturers specification and may require hydraulic modeling, possibly by CFD. Rudimental discharges can be gauged using the discharge weir outlet to stilling well and adequately measured at the downstream gauging weir which will also incorporate any seepage losses from the dam. The 2 x bottom discharge sleeve valves were selected to allow for a controlled discharge back into the river as opposed to open discharge through a horizontal valve.

The valve chambers are set out such there is provision for a future pump area sufficiently large to accommodate possible pump sets and possible turbine installation. A Ø 400 mm blanked off flange connection will be allowed to make provision for this off take.

If required, the Adelaide bulk water supply pumping unit will be housed in one of the valve house chambers. Initially this will be a 150 mm Ø bulk water steel delivery pipe will return up the external face of the left abutment adjacent to the access stair way. Alternatively the pipe could be routed internally through the gantry, depending maintenance access preference by the detailed designers and DWS.This pipe will then connect into the Ø180 mm HDPE main pipe line which links into the current raw water supply line to the Adelaide treatment works.

10.4 Pumping station outlets hydroelectric potential

A preliminary estimate of the hydro potential of Foxwood Dam, based on the projected annual agricultural releases, yielded an estimate 180 kW. This is not considered a viable supply that the Department would be willing to manage within this facility. This can be reviewed in the detailed design stage of this scheme.

10.5 Operating rules for releases

The essential operating rules post construction of these work are to meet the following discharges:

- The Environmental Water Requirements estimated to be at a maximum of 6 m^3/s.
- Provide for primary water requirements.
- Ensure that the existing downstream water license users are supplied with their allocations and at peak demands.
- Provide and meter all allocated water both from the dam and from water user's river abstractions to reconcile the discharges.
- Provide adequate quality water both to Adelaide and to the river by use of the multiple offtakes.

- Control measurement and recording for dam discharges to be measured at the new downstream gauging weir.
- The management of discharges must be clearly understood as the system can potentially exceed the maximum flow required.
- Irrigation peak flow confirmation.

10.6 Recommendations future considerations for detailed design

It is recommended that the following items be developed further during the detailed design:

- Consider the corrosion protection aspects of the pipe design particularly where pipe transition into cast-in concrete occurs
- Final backwater levels and protection of equipment and power connections
- Layout of instrumentation and ducting
- Power distribution and ducting
- Gantry lighting and ventilation
- Sedimentation build up projection on lowest intake

11 CONSTRUCTION MATERIALS

11.1 Materials availability

Details of the earthfill embankment construction materials is provided in Section 4.4. It is anticipated that the majority of earthfill materials for the earthfill embankment would be sourced from within foundation excavations. Additional material may be sourced from borrow pits investigated.

A potential Dolerite rock source for aggregate and rip-rap was also investigated to the north of the site at quarry site Q1, some 5 km north of the dam location, along the R344 gravel road.

11.2 Earthfill Requirements

The stability and seepage analysis undertaken shows that the proposed earthfill embankment is feasible. Analysis of samples from borrow pit areas indicate that suitable earthworks materials are present within the site (see Table 28 below), however borehole logs suggest an increase in granular content with depth along the embankment alignment. It is therefore recommended for selective winning of material from borrow pits and the dam foundations with the following classifications:

Table 28: Appropriate source locations for different materials

Material Type	Proposed use	Anticipated source
Selected Less Permeable material	within the core	May be obtained from near surface deposits, although the volume and extent of suitable material is not well defined
Less Free draining material	within the upstream shoulder	Anticipated to be readily available from near surface deposits within borrow pits and dam foundations
More Free draining material	within the downstream shoulder	Anticipated to be readily available at depth within borrow pits and dam foundations

The minimum requirements for the earthfill material are presented in Table 29.

Table 29: Minimum material specification requirements

Specification	Selected Less Permeable core material	Less Free draining material	More Free draining material
Grading (mm)			
<0,002	>10%		
<0,063		>40%	
<2,0		80 – 100%	
Undrained Strength	>90 kPa	>90 kPa	>90 kPa
Drained Angle of Friction (with c' = 0 kPa)	Anticipated >26°	>32°	>32°
Compacted Permeability	< 5 x 10^{-8} m/s	< 5 x 10^{-8} m/s	1 x 10^{-2} to 1 x 10^{-4} m/s

11.2.1 Filter drain

An internal blanket and chimney drain is required to filter seepage through the embankment fill materials. The anticipated specification requirements of the filter drain material are therefore based on particle size test results of the material encountered within borrow pits and within the embankment foundations. A full record of the filter design calculations is included in Appendix B. The specifications required of the filter drain are:

- D_{15} (particle size by which 15 % are smaller by dry mass) less than 0,7 mm
- Uniformity (d_{60}/d_{10}) less than or equal to 10
- Permeability less than 1 x 10^{-4} m/s

In addition, as the filter is critical to embankment design, to ensure its long term performance the source rock of crushed filter material should have the following properties:

- Resistance to abrasion (Los Angeles test) < 40 %
- Unconfined compressive Strength >60 MPa

11.2.2 Rip Rap

A layer of Rip Rap is required on the upstream embankment slope to provide protection against wave erosion and damage. Dimension requirements of Rip Rap has been assessed in accordance with the method proposed by Hudson 1959 method included in Ciria C683, The Rock Manual. The fallowing assumptions have been used in the assessment:

- Density of Dolerite of 2 700 kg/m^3 (based on published data);
- Wave height of 2,3 m for 1:100 year storm event as assessed in Freeboard calculations.

The following resultant parameters have been calculated:

- Median stone diameter D_{n50} of the Rip Rap is 0,56 m
- Thickness of Rip Rap layer is 1,1 m
- Median diameter D_{n50} of the bedding layer is 0,25 m
- Thickness of the bedding layer is 0,8 m.

A full copy of the Rip Rap design calculations are included in Appendix B.

11.3 Concrete

- **Cement**

The method of construction for the spillway section will be determined in the detailed design stage or by the Contractor. Whether roller compacted, structural or mass gravity, it will comply generally with SANS 1200 and SANS 1491 Parts 1&2 as applicable to GGBS and Fly Ash. The application of approved extenders will be determined based on the results of trial mixes conducted during the implementation stage.

- **Fine Aggregate**

The geotechnical investigations stated that there are no commercial sources of natural sand or filter materials available. These will have to be manufactured from a potential dolerite quarry site which has been identified approximately 6 km from site or other established commercial sites in the region. The applications process for both the reservation of the potential site and the approval process should be considered at the earliest opportunity once the scheme has been given approval to proceed to the next stage.

- **Course Aggregate**

The geotechnical report indicates that as a result of three boreholes the quarry is suitable as stated above for the manufacture of all grades of aggregate.

- **Water**

Water quality from the river has no indication of deleterious content provided organic matter appropriately settled and filtered out. The pH will need to be monitored at regular intervals to determine if there are any fluctuations beyond specification limits.

12 MISCELLANEOUS DAM DESIGN COMPONENTS

12.1 Dam Access Road

The access road to the dam will be off the R344. The access to the dam will have at either side, large bowel cuts into the insitu embankment allowing sufficient turning circle for vehicles. The right bank is accessed over a spillway bridge. This was incorporated as access to the right bank crest will result in significant cut and a relatively high maintenance road; however this will be investigated more fully at detailed design. Assess to the right bank downstream will be off the R63 along an internal gravel farm road. See drawing 225739-DAM-0902.

12.2 Gallery

A gallery is provided in the body of the concrete gravity dam to provide a means of access and space for drilling drainage holes and grouting the foundation, if required during operation of the dam. In addition the gallery could be used to provide access to the valve chamber – this should be considered in detailed design.

An approximate location of the gallery is shown on the drawings. The minimum dimensions of the gallery is 1,5 m wide by 2,4 m high with an arched roof. The gallery should be a minimum distance of 2,5 m from the upstream face of the dam (5% of the maximum anticipated depth of reservoir) and a minimum distance of 1,5 m from the foundation rock surface.

The dimensions quoted here are indicative only and should be reassessed at the next design phase so that all the requirements of the gallery (for example, foundation drainage, internal drainage, grouting the foundation cut-off, utility services, instrumentation etc.) are met.

The size and location of the gallery may also be affected by the construction method adopted.

12.3 Grouting

The results of the ground investigation and seepage modelling yielded that the grouting to reduce permeability within the rock foundations will be required. The seepage analysis shows the concrete gravity section founded on rock will require a grout curtain to approximately 27 m depth in order to reduce hydraulic gradient beneath and at the downstream toe of the dam. Shallow rock is present within the left abutment, and grouting will also be required to prevent excessive seepage of water laterally through the valley side.

The requirement for grouting beneath the embankment section is not shown by the seepage analysis, however it is recommended to include the grout curtain due to high near surface Lugeon values and the risk of connected permeable discontinuities within the rock. Indicative grouting layout is shown on the feasibility design drawings. The full extent and layout must be verified during the detailed design stage.

12.4 Drainage

12.4.1 Concrete Gravity Dam Drainage

Drainage holes are required from the gallery in to the foundation rock. The concrete stability analysis has assumed the drainage is located 2,5 m from the upstream face of the dam (as above) in order to relieve uplift pressure. This assumption must be confirmed during the detailed design.

Minimum dimensions of drainage pipe should be 125 mm diameter pipes at 3 m centres. The depth of the drainage holes should be approximately 50 % of the vertical depth of the grout curtain drill holes.

12.4.2 Embankment Dam Drainage

Internal chimney filter drainage is included within the embankment profile to reduce downstream pore pressures and present seepage emerging on the downstream face. The seepage analysis undertaken and the current embankment profile allows for a two meter thickness of drainage material. This design could potentially be optimized during the detailed design phase with further ground investigation and testing to refine the recompacted permeabilities of proposed embankment construction materials.

Toe drainage is required at the downstream toe of the embankment, including a collection drain, access manholes, measuring weirs and a discharge point.

12.5 Handrails

It is proposed that handrails are provided at all locations where falls from height could occur, in the interest of public and workforce safety.

13 CONSTRUCTION

13.1 Programme

A high level programme has been included in Appendix G. The duration of the contract is affected by the selected gravity concrete method selected. The programme is based on a roller compacted (RCC) estimated production which will only commence when there is adequate abutment completion to allow for uninterrupted placing. The time frame is dependent on which part of the seasons the contract is commenced in. Generally it is most suitable to commence the contract prior to the end of the rainy season. The bulk volumes are relatively small such that the earth works could be completed in one year as can the gravity section.

The constraints to construction will be to supply and maintain adequate stockpiles of quarry manufactured filters; aggregate and rip rap. The other matters which will influence closure will be the bridge construction and road realignment; land matters; power and telecommunication relocations; and any environmental matters which may come up during the study and construction. It is recommended that where feasible, these contracts be commenced and completed prior to the construction of the dam or one year prior to inundation.

13.2 Concrete construction

Determination as to whether the spillway will be a conventional shuttered and jointed gravity concrete or the currently favored RCC with less jointing and shutter fixing, will be determined at the detailed design stage or by the selected contractor. As discussed earlier the location of the outlet works in the left bank abutment allows for the relatively uninterrupted placing of mass concrete in the spillway while the more structural reinforced concrete works in the abutments and intake tower can be completed without interfering with the bulk placing. There is an allowance in the pricing build up for precast galley shuttering. The specifications for which ever method(s) of placing, will be finalised in the design stage and will conform to DWS specifications.

13.3 Earth embankment construction

Specifications for the construction of earthworks materials must be carefully finalised during the final design stage considering the proposed source materials and project programme. The following sections provides discussions on likely construction requirements.

Normally the preparation of the contact area involves the construction of cut-off's, ploughing and treatment of the foundation with lime and/or gypsum. Spreading can be done by means of dozers and/or graders depending on the availability of construction equipment. Typical layer thickness of 300 mm prior to compaction may be adopted. Strict control on material moisture contents should be observed and monitored through verification testing. Wetting of material may be required to meet specification requirements.

A review of borrow materials and compaction test should form part of the detailed design phase investigations. Verification testing by means of a nuclear densometer (Troxler or Humbolt), and regular classification tests must also be performed in order to ensure that the correct clay is used.

13.3.1 Filter drains

The method of construction of the internal chimney drain should consider the potential for contamination from cohesive embankment fills which may reduce the performance of the chimney drain. Construction of the entire embankment profile in uniform lifts is likely to be required to prevent stability issues. Typical solution to prevent contamination of granular drains is to cover with temporary geotextile separator during construction of layers of cohesive elements, then excavate a trench through the cohesive layer and backfill with granular fill. This process is repeated through the entire embankment construction.

13.3.2 Upstream Rip-rap slope protection

The upstream slope must first be prepared by removing all stones and smoothing the surface. After the preparation of the upstream slope a 200 to 300 mm layer of sand can be spread, thereafter a 200 to 300 mm layer of crushed gravel can be spread on the sand. The rip rap can then be dumped on to the surface by means of tippers or any other appropriate construction equipment.

13.4 Core Cut off Trench

Construction of the core cut off trench will need to consider construction phasing of the underlying grout curtain. It is recommended that on completion of the excavation and blasting a concrete grout cap be placed with imbedded stand pipes. A grout cap in mudstones will reduce the exposed weathering and rework as well as limit the hydraulic fracture leeks at the surface.
This phasing of construction has the following sequences:

- The excavation of the cut off trench
- Clean and trim trench
- Cast concrete grout cap
- Commence the drilling and grouting procedures

13.5 Backfill

Backfilling of temporary excavations will be required surrounding the foundations for the concrete gravity dam and retaining walls. Suitable excavated materials must be stockpiled and sealed while not in use. The excavation must be done in such a manner to allow for sufficient space between structures and the undisturbed natural soil. This is necessary to ensure that proper construction equipment can be accommodated in order to perform proper back filling. On the downstream side it must be ensured that drainage water is diverted away from the structure. The permeability of the backfill material must also be considered, and must be modified if deemed necessary.

13.6 Labour intensive construction

Labour intensive construction of the dam as opposed to labour intensive construction tasks on the dam was considered. It was considered that the approximately 300 000 m^3 of concrete could be substituted with a rubble masonry resulting in a labour intensive alternative. Estimates of production of rubble masonry were obtained from an experienced contractor who specializes in this form of dam construction. The estimated construction per day per 100 labourers was 750 m^3. This implies that if the bulk of the concrete works were to be completed in a 2 year period approximately 800 personnel would potentially be on or in close proximity to the wall. The majority being on the wall. This would be neither practical nor safe. If the construction period were extended the P&G element of the project would exceed the 10 % premium. The cost of roller compacted concrete is competitive with the masonry alternative. Areas where labour intensive tasks can be carried could be in the following:

- Finishing and landscaping
- Slope Protection
- Structure backfilling
- Filter placement

There will be associated contracts which lend themselves to labour intensive construction or works:

- The bulk water pipeline routed to connect to the existing Adelaide water supply pipeline
- The canal reinstatement with a 600 mm pipeline and canal;
- R344 Road realignment and bridge;
- Grave relocations

All of these tasks could be carried out wholly or partially as a labour intensive operation. To establish, at this stage, the comparative cost advantage or otherwise of these tasks is not of significance. Different contractors have varying approaches to tasks. It is suggested that the tender incorporated a dual rate system for any item considered suitable for labour intensive work and that if this rate is less than 10 % of the machine approach then the premium on this rate will be considered without affecting the competitiveness of the tender.

13.7 River diversion

It is recommended that the river diversion strategy commence at the onset of a dry season in order to facilitate the installation of a diversion culvert 4,0 m wide by 3,5 m high with its invert at the river bed level of 578 masl in the middle left of the spillway section. A 60 m wide section of the spillway must be kept 4 m lower than the rest of the spillway for the duration of the spillway construction. The discharge capacity of the low section will be approximately 555 m^3/s, which will allow for the passing of floods during the dry season. The diversion culvert will keep the upstream water level at approximately the river bed level during normal dry season flow

As the dam height increases the flood absorption capacity of the basin will increase. The 1 in 20 year flood volume will be absorbed when the embankment is at level 592 masl and the 1 in 50 year flood volume when it is at level 603 masl.

- First stage diversion and coffer dam required for excavation and concrete placement to river bed level before installing / forming culvert for low flows;
- Second stage diversion to enable excavation and concrete placement for remainder of spillway and construction of embankment;
- Capacity of conduit and proposed height of coffer dam to take account of flow rates for 1:20 and 1:50 floods;
- Sequence of construction of embankment in relation to spillway to ensure earthworks are not overtopped; and
- Closing of conduit when dam is ready for impoundment (steel stoplogs to close opening, filling with concrete and grouting).

13.8 Borrow areas

Should material from borrow areas be required to meet the materials balance or permeabilities, the excavations should be planned to limit trafficking distance an impact on the landscape. Excavations within the reservoir basin will reduce the need for landscaping. The control of groundwater within borrow pit excavations should also be considered. It is noted that the development, operation and closure of borrow areas and quarry sites is subject to an environmental management plan, which needs to be submitted to and approved by the Department of Mineral Resources (DMR). However it is also noted that DWS has exemption from needing a license from DMR for a quarry or borrow areas

13.9 Quarries

A potential Dolorite quarry is situated north of the dam alongside the R344 road. Excavations of this quarry could improve the road alignment, and with it, safety of road users. Excavation at this location may need to consider subsequent rehabilitation with vegetation and the safety of the public. It is recommended that the relevant environmental and mining approvals be commenced as soon as possible after project approval.

13.10 Quality control

There will be a requirement for a full time materials laboratory on site with the appropriately experienced technician and support staff. There should be quality inspectors at selected borrow areas, in particular where selection of material that must be tested and approved is required. Record of all testing shall be recorded and backed up at regular interval and stored off site.

Concrete
All cement and extenders on site should be of specified quality conforming to SABS standards and should be accompanied with the manufacturer's test certificate for every batch. The storage must be off the ground and weather proof. Similar care should be exercised when transporting around the site.

Test on concrete and aggregates shall include, but is not limited to, the following:

- slump tests
- cube strengths for suitably water bath stored and record cubes 7, 28 & 90 day sampling
- VB testing for workability of mix design on the day
- at batching plant, regular moisture content of fine aggregates
- certified scale should be regularly checked against standard weight on site and serviced as per the supplier specification.

There may be a requirement to carry out select core samples for testing of permeability; strength and the various moduli. Some of these sample will come from the gantry drainage boreholes.

Earthfill
The site laboratory will be equipped to carryout Atterberg limits; Proctor tests; Crumb tests; and aggregate quality including grading angularity and flakiness. Dispersivity Pin hole testing should be carried out off site certainly in the preliminary identification and selection of embankment material. There may be a necessity to consider lime or gypsum stabilization if this justified on the basis of the economics.

Excavations
All excavation slopes must be certified as safe and if necessary support or cut back further prior to deeper excavation being required. This should be assessed and tested by the appropriately experienced staff on site. Prior to backfill the contractor is obliged to seek the authorisation of the Engineer and suitable survey and record must be completed prior to this operation.

All excavated material shall be placed in the approved designated areas for the type of material whether waste of for use in the works. The dump sides need to be trimmed to ensure they are stable and safe.

Bush Clearing - There will be requirement to dispose of vegetation of the bush clear operation. Identification and approval of vegetation must be carried out with appropriate Environmental Control Office approval.

Landscaping
All landscaping and rehabilitation of disturbed areas, within the basin and on the construction site, shall be done in accordance with the Environmental Management Programme (EMPR) and for approval by the Engineer in consultation with the Environmental Control Officer.

14 DAM SAFETY ASPECTS

This is a Category III dam and as such the detailed design will be carried out in terms of the current National Water Act Chapter 12 sections 117 to 123.

14.1 Legislation Obligations

The following is required in terms of dam safety legislation and regulations:

- Design by a professional team under supervision of an Approved Profession Person (APP) for that category of dam
- An application for a permit to construct accompanied by the design drawings, report and specifications
- During construction the APP is required to provide assurance that the dam is constructed in accordance with the specification and that alterations to the design are approved and signed off.
- All permits, licenses and temporary wayleaves are in place prior to either construction and/or closure including all requested documentation and plans.
- All conditions pertaining to the construction and impoundment of the dam are adhered.
- The APP is required to submit quarterly progress reports during construction, and a completion report and record drawings at the end of construction

14.2 Instrumentation

Instrumentation will be required to monitor the performance of the dam elements. These are likely to include the following:

- Settlement monitoring points at 20 m intervals along the crest of the embankment and concrete gravity dam
- Vibrating wire piezometers within the embankment materials to monitor pore pressure during construction and be able to prevent excessive pore pressures building up within the embankment
- Downstream piezometers within the embankment to monitor seepage
- Seepage flow monitoring channels and weirs within the gallery of the concrete gravity dam
- Flow monitoring weirs within toe drainage features for the embankment

14.3 Dam break analysis

Foxwood Dam is classified as Category III or High Hazard. Due to its proximity to the town of Adelaide and the proposed new irrigation scheme located adjacent to the river line, a dam break analysis is required, by the Dam Safety Officer, to determine the potential loss of life, damage to property/infrastructure and economic losses following a hypothetical dam failure. This will then inform the inundation mapping for the emergency preparedness planning.

Due to the composite nature of the dam, at least four scenarios should be considered:

- Failure of the concrete gravity section with no incoming flood (a “sunny day” failure)
- Failure of the embankment section with no incoming flood (a “sunny day” failure)
- Failure of the concrete gravity section during a flood event (PMF and/or 200 year)
- Failure of the embankment section during a flood event (PMF and/or 200 year)

The dam break hydrographs should be routed downstream as far as practically possible i.e. as far as detailed topographic survey allows which extends to the confluence of the Fish River. This would probably be when the flood flow returns to the natural watercourse.

The flood outlines will also contribute to inundation mapping and for the emergency preparedness planning.

14.4 Emergency draw-down

Rapid draw down of the reservoir may be required to prevent a potential failure of the dam. The outlet structure is capable of drawing down 90 % of capacity from full in a 3 month period. If a lesser period is required the outlets system can be designed to accommodate this at the next stage, however care must be taken on the impacts downstream.

14.5 Emergency Preparedness Plan (EPP)

An EPP will need to be developed at an early stage of the detailed design phase, to allow for any mitigation measures that may be required, and if required incorporated in the design.

The EPP for the dam will include the following:

- Notification Flow Chart
- Communications
- Levels of Emergency – normally 4 ranging from minor to evacuation and national
- Owners details
- Disaster Management Authorities Chain and contacts with alternates
- Dam details and documentation including emergency indicators
- Dam emergency mitigation measures
- Inundation mapping
- Preparedness plan
- Suitable equipment and materials for local repair of potential minor events, such as sand bags
- Affected Parties
- Possible conditions for reoccupation and economic consequence for repair both for the dam as well as affected parties

15 DAM STATISTICS

The tables below provide a summary of the key dam statistics

Table 30: Dam Statistics - Locality

LOCALITY	
Province:	Eastern Cape
District Municipality:	Amathole District
Co-ordinates of dam:	32° 40'30'' S 26° 16' 0'' E
Nearest town by road:	Adelaide

Table 31: Dam Statistics - Catchment

CATCHMENT	
Drainage Number:	Q92
River:	Koonap
Catchment Area:	3 334 km^2
Mean Annual Precipitation (MAP):	513 mm
Mean Annual Runoff (MAR):	79,6 million m^3/a

Table 32: Dam Statistics – Structural Information

STRUCTURAL INFORMATION	
Type of dam:	Composite concrete
Overall length of wall:	485 m
Length of spillway (including piers):	267 m
Total length of left bank NOC:	48 m
Length of earth fill on right bank:	163 m
Length of outlet works:	58,375 m
Non-overspill crest level:	620,5 masl
Spillway crest level:	615,0 masl
Lowest foundation level:	571,6 masl
Maximum height of NOC above foundation:	48,9 m
Recommended Design Discharge (1:200):	2 063 m^3/s
Excavation volume:	234 388 m^3
Earth fill and backfill material volume:	584 820 m^3
Total volume of reinforced concrete:	51 840 m^3
Total volume mass concrete:	220 183 m^3

Table 33: Dam Statistics – Reservoir Information

RESERVOIR INFORMATION	
High Flood Level (HFL)-1:100:	617 m
Design Flood Level (DFL) 1:200:	617,50 m
Safety Evaluation Flood Level:	620,50 m
Full Supply Capacity:	54 995 984 m³
Lowest Draw Down Level	585,40 m
50 year Silt Volume	6,1 million m³
Reservoir Surface Area at HFL	4 634 414,49 m²

Figure 16: Dam Area-Storage capacity curve

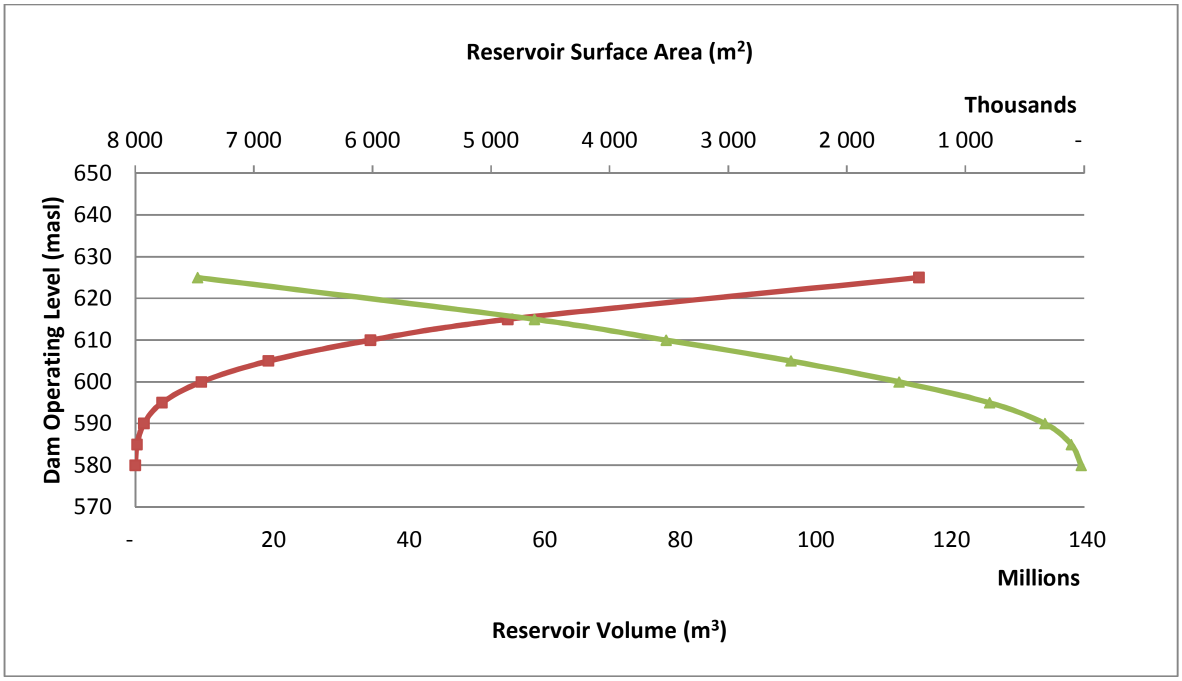

Table 34: Dam Statistics – Design Flood Peaks

DESIGN FLOOD PEAKS	
Return Period (Years)	**Discharge**
5	176 m³/s
10	332 m³/s
20	555 m³/s
50	985 m³/s
100	1 457 m³/s
200	2 063 m³/s
Recommended Design Flood (PMF)	2 063 m³/s
Regional Maximum Flood (RMF)	5 218 m³/s
Safety Evaluation Flood (SEF) (PMF routed)	6 200 m³/s

Table 35: Dam Statistics – Outlet Works

OUTLET WORKS	
River Outlet - 2 x Bottom Discharge Sleeve Valves	6 m^3/s
Maximum design pipe velocity	4 m/s
Multiple Intake	4 Nº

16 COST ESTIMATES

Full details of the dam and project costing are provided in the costing report (DWS 2015d).

A summary of the costed Bill of Quantities for the dam works is provided in the Table 36 below. Refer to Appendix H for the full bill. The rates used against the cost items for the dam structure have been sourced through consultation with a reputable major contractor currently completing a large dam in South Africa. The contractor provided rates to an Arup's draft Bill of Quantities.

Table 36: Summary bill of quantities for the main dam works

Description	**Amount** (ZAR, excl VAT)	Comment
PRELIMINARY & GENERAL	239 411 545	*30% of all items*
WATER CONTROL-RIVER DIVERSION	5 118 848	
DRILLING & GROUTING	65 895 189	
Earthfill	5 772 591	
Concrete Gravity	60 122 598	
GRAVITY SPILLWAY	434 835 032	
GRAVITY NOC	26 515 352	
EARTHFILL EMBANKMENT	105 196 437	
OUTLET WORKS	64 306 681	
Concrete Works	21 204 550	
Mechanical Equipment	39 102 131	
Structural Steelwork	1 750 000	
Electrical Equipment	2 250 000	
INSTRUMENTATION	7 500 000	*Provisional Sum*
Miscellaneous 10% & Landscaping 2,5%	88 670 942	*(12,5% of cost* (excl P&G))
DAM CONSTRUCTION (incl. P&G)	1 167 651 897	

The above cost is for the dam structure works only and excludes associated infrastructure and also excludes contingencies, professional fees and VAT. Full details of the project cost (including associated infrastructure such as road and services relocation and land purchase) are provided in the Costing Report (DWS 2015d) including allowances for contingencies and professional fees as well as VAT included in the report.

The URV resulting from the total project cost of R 2 084 million is R 11.77 /m^3 (DWS 2015e)

17 CONCLUSIONS AND RECOMMENDATIONS

The selection of a composite dam with a concrete gravity spillway and earthfill embankment was based on both engineering and construction considerations as well as having the optimum URV for the selected storage capacity. A storage capacity of 1 MAR was selected as being appropriate for both the foreseeable development plans of both Adelaide and the agricultural irrigation models investigated as well as having the least environmental impact on the management of the river and dam.

There has been a full geotechnical investigation to allow for a feasibility costing of the dam and associated works. The embankment materials are largely anticipated to be selected from excavation of the deep over burden overlaying the spillway and cutoff. Dispersive indicators have been recorded in some samples, indicating care will need to be taken in the selection of the fill materials. There were no natural filter sands economical identified in the area. It is anticipated that the filter materials, concrete aggregate and rip rap will have to be produced from potential dolerite quarry site. It is recommended that at the earliest possible date the process of approvals to develop this quarry site be commenced.

The spillway has been analysed as a concrete gravity structure which can be either conventionally placed or RCC. There is recommendation that it be modeled to determine the actual discharge coefficient, the effect of the backwater on the stilling basin design and the cavitation / aeration potential on the stepped section. The designed and modelled spillway incorporates a bridge to provide access to the earth embankment crest on the right bank. This increases the spillway length from 250 m to 267 m. DWS have expressed reservations regarding the incorporation of a bridge on the dam structure and this must be reviewed during detailed design. The dam is categorized as Category III, and as such the spillway is designed to pass the Recommended Design Flood (RDF) of 2 063 m^3/s (1 in 200 year flood event) with dry freeboard and pass the Safety Evaluation Discharge (SED) of 6 200 m^3/s (PMF) without freeboard. It is noted that this may be optimized during detailed design with the Kovacs + Δ method (5 218 m^3/s).

The downstream face of the spillway is sloped at 0,6H:1V (or 59 degrees). This is the maximum steepness determined from the stability analysis. This is steeper than normally accepted 1:0.75 to 0.7, this is a function of the large spill basin blocks that were set to the ground level rather than the hard horizon. This can be reviewed during detailed design although further geotechnical investigation would be required to confirm the hard horizon and extent of excavation to provide a return to river. The toe of the concrete gravity dam has a 15 m long stilling basin block which is stepped to follow the ground level. The return is protected by a cascade system of graded large rocks and rip rap underlain by a crusher graded filter.

The earthfill embankment section will be constructed using site won alluvial / colluvial material with selection of lower permeability fill in the core and a chimney drain incorporated. The earthfill embankment has upstream and downstream slopes of 1 in 4 and 1 in 3 respectively.

The outlet works are designed with a twin stack system to allow for 100% redundancy to make provision for discharge of the maximum EWR (6 m^3/s) and all downstream off takes and to ensure that with multiple level off takes adequate water quality is maintained in the discharges. Although the individual stacks can discharge the full discharge with a pipe design, a constraint was set at flow velocities < 8 m/s, it is recommended that in order to deliver the EWR that both stacks be used. This will result in < 4 m/s flows. Access to the outlets works and valve chamber can be via the external face of the concrete wall or internally through the gantry and gallery.

The river diversion strategy should commence at the onset of a dry season in order to facilitate the installation of a diversion culvert 4,0 m wide by 3,5 m high with its invert at the river bed level of 578 masl in the middle left of the spillway section. A 60 m wide section of the spillway must be kept 4 m lower than the rest of the spillway for the duration of the spillway construction.

The construction programme includes for commencement of the various ancillary contracts such as the realignment of the R344 including a new 91 m bridge as well as other associated land matters.

The Environmental Impact Assessment is current underway (commenced late 2014.) This may well add to the detailed design considerations.

18 REFERENCES

Boes, 2012	Boes, R.M. (2012). Guidelines on the design and hydraulic characteristics of stepped spillways. Proc. 24th ICOLD Congress, Kyoto: Q. 94, R. 15: 203-220
BRE, 1991	An Engineering Guide to Seismic Risk to Dams in the United Kingdom, (BR 210), Building Research Establishment 1991
DWA, 2014	Foxwood Dam Options Selection Memorandum - Summary. Arup, July 2014.
DWA, 2015	Department of Water Affairs, 2014. Feasibility Study for Foxwood Dam: Alternative Water Supplies Report, P WMA 15/Q92/00/2113/5
DWS, 2015a	Department of Water and Sanitation, 2015. Feasibility Study for Foxwood Dam: Koonap River Hydrology, P WMA 15/Q92/00/2113/7
DWS, 2015b	Department of Water and Sanitation, 2015. Feasibility Study for Foxwood Dam: Water Quality Report DWS report P WMA 15/Q9200/2113/10
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DWS, 2015d	Department of Water and Sanitation, 2015. Feasibility Study for Foxwood Dam: Project Feasibility Costing, P WMA 15/Q92/00/2113/13
DWS, 2015e	Department of Water and Sanitation, 2015. Feasibility Study for Foxwood Dam: Economic Impact Assessment, P WMA 15/Q92/00/2113/14
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DWS, 2015g	Department of Water and Sanitation, 2015. Feasibility Study for Foxwood Dam: Agro-Economic Study, P WMA 15/Q92/00/2113/9
ICOLD, 2011	ICOLD Bulletin 137 Reservoirs and Seismicity - State of Knowledge. 2011
SANCOLD, 2011	South African committee on large dams (SANCOLD) - Volume II - Guidelines on freeboard for dams. Bosman DE; Basson JR; Tente T; Basson GR, 2011

USBR, 1987	U.S. Dept. of the Interior, Bureau of Reclamation, Design of Small Dams. 1987

APPENDIX A: GRAVITY DAM – RESULTS OF STABILITY ANALYSIS

ARUP

JOB TITLE	Foxwood Dam
JOB NUMBER	225739
MADE BY	PB
CHECKED BY	DC
DATE	16/09/2014
Description of spreadsheet	Global stability of the proposed concrete overflow section of Foxwood Dam
Sheet Number prefix	
Member/Location	London
Drawing Reference	
Filename	C:\Users\peter.brinded\Desktop\[Overflow Stability Analysis_Rev3.xlsx]

CONTENTS OF SPREADSHEET

Sheet	Description
Cover	
Notes	
Introduction	
Acceptability Criteria	
Overflow section parameters	
Hydrostatic Parameters	
Earth Pressures	
Flood Parameters	
Seismic Parameters - Horizontal	
Seismic Parameters - Vertical	
Usual Case	
Unusual Case	
Extreme Case - Seismic	
Calc Summary	
Calc(P)	

AUTHORISATION OF LATEST VERSION

Type and method of check		By hand
Signatures & dates:	Made by	PB
	Checked	DC

REVISIONS Current Revision Rev3

Rev.	Date	Made by	Checked	Description
Rev0	27/08/14	PB	DC	Preliminary Assessment.
Rev1	11/09/14	PB	DC	Amendments following DC check
Rev2	16/09/14	PB	DC	Geometry amended
Rev3	19/09/14	PB	-	Stilling basin reduced. Downstream fill removed for unusual

(1) Purpose of spreadsheet

The purpose of this spreadsheet is to confirm the global stability of the proposed concrete overflow section.
The stability of the section will be assessed against three load cases: usual, unusual and extreme

(2) Key Assumptions

The analysis is based on the current design and design information.
All assumptions are listed in the Foxwood Dam Design Input Statement

(3) Basis of calculations

USBR Design of Small Dams (1987)
USBR Design of Concrete Gravity Dams (1976)
USACE Gravity Dam Design (1995)
ANCOLD Guidelines on Design Criteria for Concrete Gravity Dams (2013)

(4) Sources of data & Links to other spreadsheets

Date	File path / URL	Description

(5) Special features

N/A

(6) Diary of development, including checking

(if supplement is needed to Cover page)

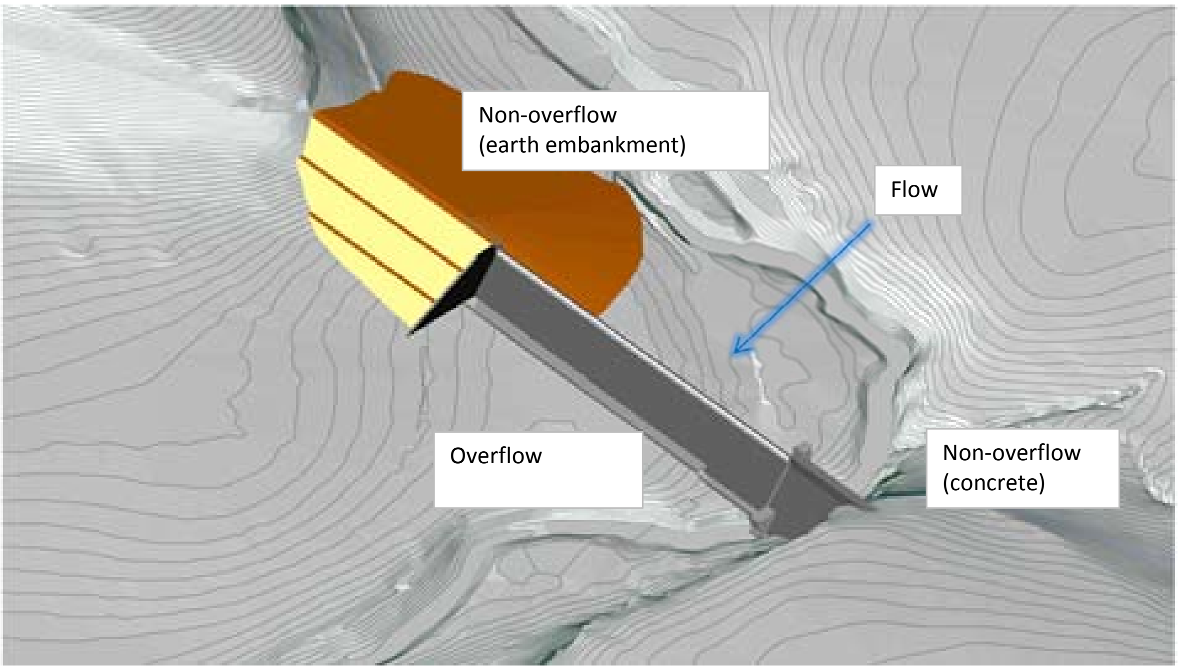
Date	Who	Description

Best Practice Guide

1. Don't duplicate raw data in the spreadsheet i.e. use cell references where possible.
2. Use colours to distinguish between fixed data, user-variable data, calculations and results.
3. Explicitly define constants to be used in equations, using named cells where appropriate.
4. Avoid password use unless essential and documented (to avoid loss of work with loss of password).
5. Ensure extracts copied to other documents can be traced back to the spreadsheet.
6. Plot to engineering scale whenever sensible to do so, and make units obvious.
7. For charts, use colours/patterns which will be distinguishable if printed or photocopied in black & white.
8. Give sheets & workbooks descriptive names.
9. Use comments to describe the purpose of individual cells and ranges of cells.
10. Use the revision facility on the cover page and maintain the diary where further details required.

The purpose of this calculation is to check the global stability of the proposed concrete overflow section.

A visual of the proposed Foxwood Dam is provided below:



The load cases that are considered are based on USBR *Design of Small Dams*

Load	Usual	Unusual	Extreme
Dead load	Yes	Yes	Yes
Hydrostatic water load	Yes – FSL	Yes – maximum design reservoir elevation (SED = PMF)	FSL
Silt	Yes	Yes	Yes
Earth	Yes (upstream submerged and downstream dry)	Yes (upstream and downstream submerged)	Yes (upstream submerged and downstream dry)
Uplift*	Yes	Yes	Yes
Earthquake	No	No	MCE

* The presence of a drainage gallery (with vertical drains in to the foundation) means that there can be a reduction in uplift near the upstream face. For this stage of the design (feasibility) a drainage effectiveness of 50% will be assumed in accordance with USACE (1995). Where the line of drains intersects the foundation within a distance of 5 percent of the reservoir depth from the upstream face, the uplift may be assumed to vary as a single straight line, which would be the case if the drains were exactly at the heel. Refer to relevant calculation for uplift diagrams.

The acceptability criteria have been determined using USBR (1987)

Input data

For sliding:

Load case	Usual	Unusual	Extreme
FoS	3	2	1

The sliding assessment is reported for the concrete/rock interface. It is assumed that the concrete gravity section will be cast in-situ (i.e. no pre-cast elements) hence a strong bond to the rock will occur.

It is noted that the same parameters would be used for sliding within the foundation rock, therefore both are covered by the same calculation.

It should be noted that there will be a 1m deep cut off in to the foundation rock. This will resist sliding to some degree but is not considered in these calculations

Sliding failure within the concrete structure is not assessed in this calculation package. It is assumed here that the the structure will be designed accordingly in subsequent design phases.

For overturning:

Load case	Usual	Unusual	Extreme
Resultant Location	Middle third	Middle half	Within base
Foundation Bearing Pressure	< allowable	< allowable	< 1.5 x allowable
Maximum Compressive Stress*	0.33f'c	0.5f'c	1.0f'c

* f'c is the characteristic compressive strength of the concrete.

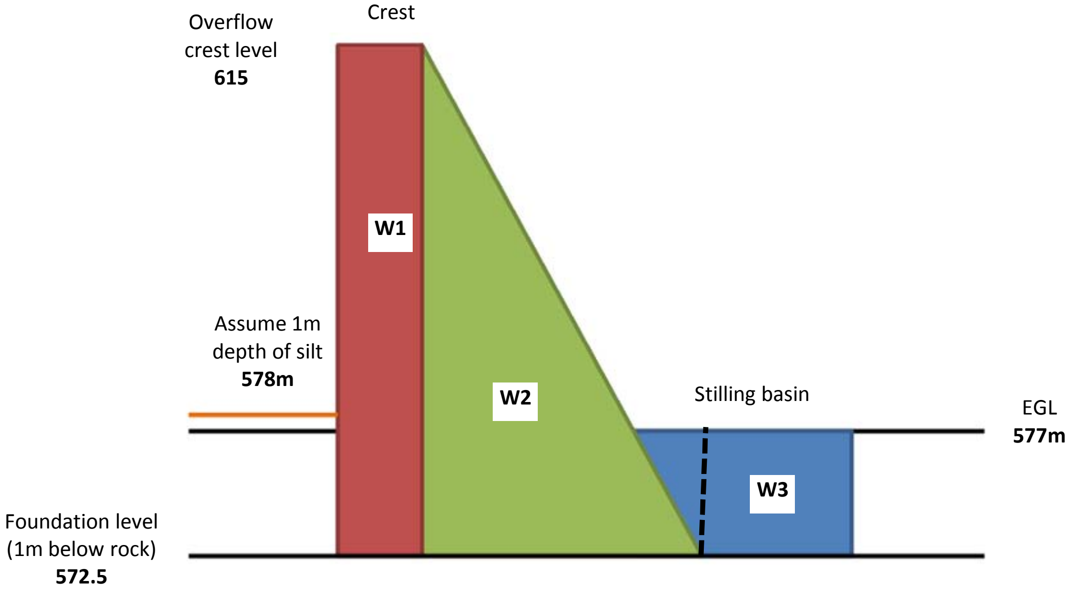
On this scheme, the value of f'c is 40.0 N/mm^2 Assuming a C32/40 concrete and adopting cube f'c

The allowable bearing capacity is 4000.0 kN/m^2 (of the rock foundation)

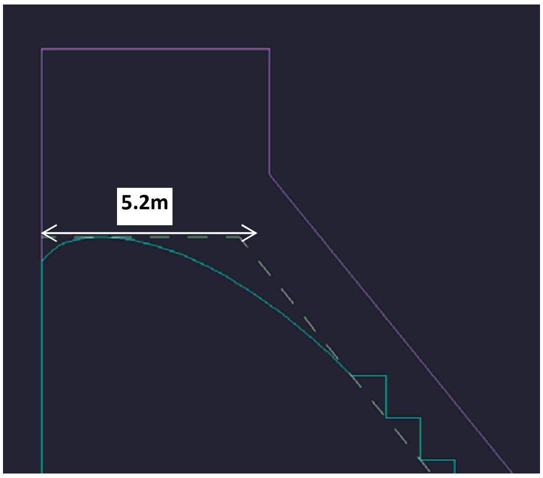
No tension at the concrete/rock interface is acceptable

Input data

The analysis is based on the following section:
(upstream and downstream faces are assumed vertical however arrangement dependent on construction method/material)



* The crest width has been determined as follows:



As can be seen above, the simplified section will slightly over-estimate the cross sectional area at the crest.

Key Levels:

Crest (also FSL)	615.0	m*
Silt	578.0	m
EGL	577.0	m (also taken as invert level of stilling basin for simplicity)
Foundation	572.5	m

* m above datum

Therefore:

Height of section	42.5	m
Depth of silt	1.0	m
Depth of EGL fill	4.5	m

Unit weight concrete	24.0	kN/m³

Slope 1 in	0.60

Key Dimensions:

Width of W1	5.2	m
Base width of W2	25.500	m
Width of stilling basin	13.00	m
Base width of W3	10.300	m

Dam Base	41.00	m

Dam self weight

Dam self weight is made up of a number of components as listed below. The dimensions for each component are given including a dimension to the upstream toe (lever arm) to assist in moment calculations.

W1	Height	42.50	m
	Width	5.20	m
	Lever Arm	2.60	m about the upstream toe
	Weight is	5304.00	kN/m
W2	Height	42.50	m
	Width	25.50	m
	Lever Arm	13.70	m about the upstream toe
	Weight is	13005.00	kN/m

W3 comprises two components: $W3_{triangle}$ and $W3_{rectangle}$

$W3_{triangle}$	Height	4.5	m
	Width	2.700	m
	Lever Arm	29.8	m about the upstream toe
	Weight is	145.8	kN/m
$W3_{rectangle}$	Height	4.5	m
	Width	10.3	m
	Lever Arm	35.85	m about the upstream toe
	Weight is	1112.4	kN/m

Total dam weight is therefore	19567.20	kN/m

The moments due to the weight about the centroid (stress analysis) and the downstream toe (overturning) are required. In the above, the lever arms have been given about the upstream toe. In the case of centroid moments, some of the moments are clockwise (+ve) and some anticlockwise (-ve). All of the weight related moments in the overturning case are anticlockwise (-ve).

The overall width of the section is - 41.0 m

Moments about centroid

Component	Weight	Lever arm to upstream toe	Lever arm to centroid	Moment
W1	5304.00 kN/m	2.60 m	-17.90 m	-94942 kNm/m
W2	13005.00 kN/m	13.70 m	-6.80 m	-88434 kNm/m
W3triangle	145.8 kN/m	29.8 m	9.30 m	1356 kNm/m
W3rectangle	1112.4 kN/m	35.850 m	15.35 m	17075 kNm/m

Total moment about centroid = -164944 kNm/m

Moments about the downstream toe

Component	Weight	Lever arm to upstream toe	Lever arm to d/s toe	Moment
W1	5304.00 kN/m	2.60 m	-38.40 m	-203674 kNm/m
W2	13005.00 kN/m	13.70 m	-27.30 m	-355037 kNm/m
W3triangle	145.8 kN/m	29.8 m	-11.20 m	-1632.96 kNm/m
W3rectangle	1112.4 kN/m	35.85 m	-5.15 m	-5728.86 kNm/m

Total moment about downstream toe = -566072 kNm/m

Usual Hydrostatic Parameters

Input data

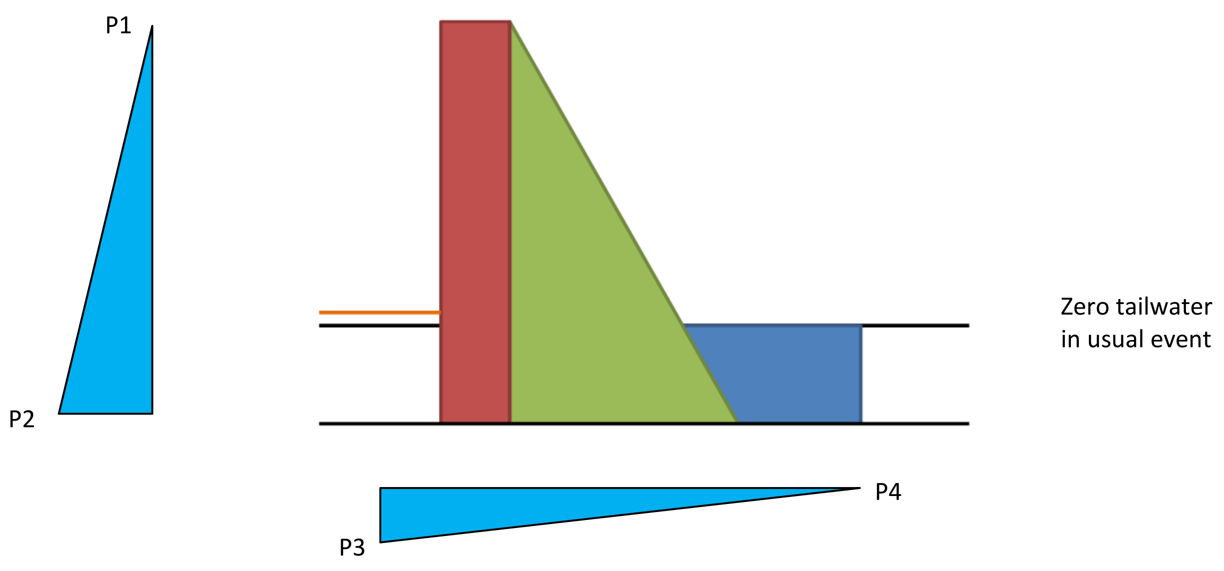
In the usual load case, the hydrostatic (water) level is taken to be at Full Service Level (FSL). This is also assumed for the extreme load case.

In the unusual load case, the hydrostatic level is taken to be at Maximum Water Level (during a PMF). This is considered in a subsequent calculation page.

Further to the hydrostatic load, the structure is also subject to uplift pressures across 100% of the base. In line with ANCOLD guidance, and considering the foundation rock as fractured mudstone with a permeability of 10^-6, the uplift pressure is assumed to respond instantaneously with reservoir level.

There is a drainage gallery located near the upstream face.
The presence of a drainage gallery (with vertical drains in to the foundation) means that there can be a reduction in uplift near the upstream face. For this stage of the design (feasibility) a drainage effectiveness of 50% will be assumed in accordance with USACE (1995).Where the line of drains intersects the foundation within a distance of 5 percent of the reservoir depth from the upstream face, the uplift may be assumed to vary as a single straight line, which would be the case if the drains were exactly at the heel.

Load diagram:



The hydrostatic pressures need to be calculated. The parameters to use in the calculations are as follows:

Unit Weight of Water	10	kN/m^3
Dam crest level (FSL)	615	m
Foundation level	572.5	m

Hydrostatic pressure:

In the usual case the hydrostatic pressure occurs between FSL and foundation level.

On the upstream side:

P1 0.00 kN/m^2
P2 425.00 kN/m^2 (hydrostatic depth x unit weight of water)

The sliding force is 9031.25 kN/m (0.5 x P2 x hydrostatic depth)

The overturning moment depends on the lever arm about the centre and the front toe of the dam. The dam is assumed to have a level foundation so both moments will have the same lever arm.

Lever arm 14.17 m
Overturning moment 127942.71 kNm/m (sliding force x lever arm)

At the downstream toe there is no tailwater therefore no restoring hydrostatic pressure.

Uplift pressure:

As stated above, there is a drainage gallery. The effectiveness of the drainage is assumed to be 50%
Therefore reduction in uplift due to drainage is 0.5

As there is no hydrostatic load on the downstream toe, the uplift distribution is triangular.

Horizontal pressure P2 425.00 kN/m^2
Revised for drainage P2 is 212.5 kN/m^2

The length of the base upon which uplift pressures act 41.0 m

Therefore uplift force is 4356.25 kN/m

Moment about the centroid:
Lever arm 6.83 m
Moment 29767.71 kNm/m

Moment about the downstream toe:
Lever arm 27.33 m
Moment 119070.8 kNm/m

There may be a case where the drains are blocked.
Therefore reduction in uplift due to drainage is 1 i.e. no reduction

As there is no hydrostatic load on the downstream toe, the uplift distribution is triangular.

Horizontal pressure P2	425.00	kN/m^2
Revised for drainage P2 is	425	kN/m^2

The length of the base upon which uplift pressures act 41.0 m

Therefore uplift force is 8712.5 kN/m

Moment about the centroid:

Lever arm	6.83	m
Moment	59535.42	kNm/m

Moment about the downstream toe:

Lever arm	27.33	m
Moment	238141.7	kNm/m

This will be considered as a sub-case to the usual load case to determine the sensitivity to adequate drainage

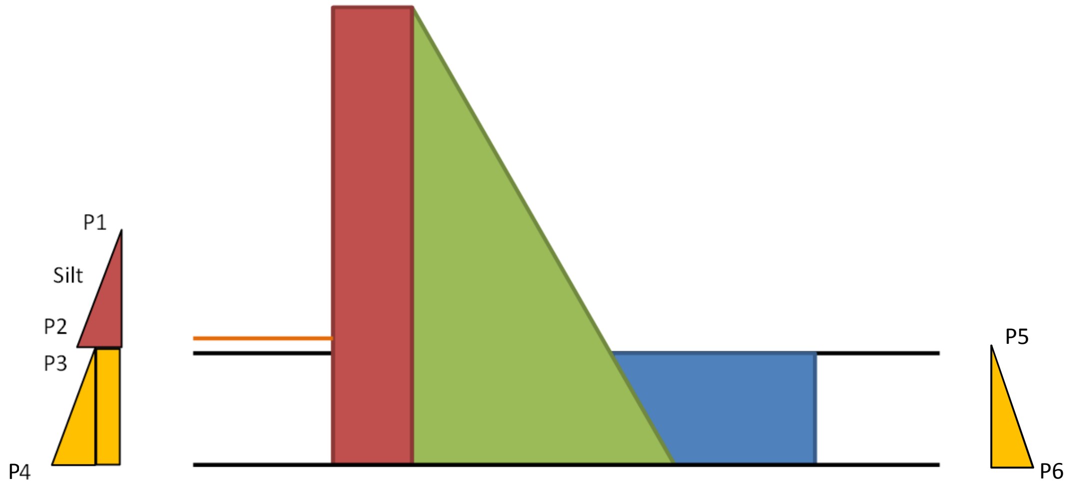
Usual Earth Pressure Parameters

Input data

There is earth fill against the lower parts of both faces. There are four pressure scenarios to consider:
Upstream fill - submerged (usual, unusual, extreme load cases)
Downstream fill - submerged (unusual) — *Submergence depends on presence of tailwater.*
Downstream fill - not submerged (usual, extreme) — *Tailwater only anticipated during overtopping (flood)*
Silt - submerged (usual, unusual, extreme)

If there is no significant tension at the downstream toe then the deflections will be sufficiently nominal to not cause additional strain in the downstream soil mass. Therefore at-rest earth pressure coefficient is deemed appropriate.

Load diagram:



The earth and silt pressures need to be calculated. The parameters to use in the calculations are as follows;

Crest	615.0 m		
Silt	578.0 m	Therefore depth of silt	1.0 m
EGL	577.0 m	Therefore depth of earth fill	4.5 m
Foundation	572.5 m		

Unit weight of earth fill	18	kN/m^3	(assuming recompacted embankment fill)
Unit weight of water	10	kN/m^3	
Submerged unit weight of fill	8	kN/m^3	
Angle of friction	32.5	degrees	
Ko	0.46	At-rest coefficient	
K_A	*0.30*	*Active coefficient*	*(not used)*
K_P	*3.32*	*Passive coefficient*	*(not used)*

Unit weight of silt	17	kN/m^3
Submerged unit weight of silt	7	kN/m^3
Angle of friction	20	degrees
Ko	0.66	At-rest coefficient

Calculate the earth pressures, forces and moments as shown on the diagram.
Centroid and d/s toe moments have the same lever arm therefore produce the same moment.

Silt - submerged (usual, unusual, extreme)

P1 (silt) 0.00 kN/m^2
P2 (silt) 4.61 kN/m^2 (K_{0silt} x submerged unit weight x depth of material)
The pressure distribution is triangular. Determine the forces and the moments:

Force	2.30	kN/m	(0.5*P2*depth of material)
Lever arm	4.83	m	
Moment	11.13	kNm/m	

Upstream fill - submerged (usual, unusual, extreme)

P3 (fill) 3.24 kN/m^2 (K_{0fill} x submerged unit weight silt x depth silt) *i.e. silt pressure distributed through the fill soil matrix*

P3 rectangle force	14.58	kN/m	(P3 x depth of fill)
P3 rectangle lever arm	2.25	m	(depth of fill / 2)
P3 rectangle moment	32.79	kNm/m	

P4 (fill) 19.90 kN/m^2 (K_{0fill} x submerged unit weight fill x depth fill) + P4

P4 triangle force	44.77	kN/m	(0.5 x P4 x depth of fill)
P4 triangle lever arm	1.5	m	(depth of fill / 3)
P4 triangle moment	67.15	kNm/m	

Therefore total:

Force	59.34	kN/m
Moment	99.94	kNm/m

Downstream fill - submerged (unusual) due to tailwater from an overtopping event

P5 (fill) 0.00 kN/m^2 *Bulk weight still used as tailwater hydrostatic is not considered*
P6 (fill) 37.48 kN/m^2 *If tailwater hydrostatic considered then use submerged unit weight*
The pressure distribution is triangular. Determine the forces and the moments:

Force	84.33	kN/m	Restoring force
Lever arm	1.50	m	
Moment	126.49	kNm/m	Restoring moment

Downstream fill - not submerged (usual, extreme) i.e. no tailwater

P5 (fill) 0.00 kN/m^2
P6 (fill) 37.48 kN/m^2
The pressure distribution is triangular. Determine the forces and the moments:

Force	84.33	kN/m	Restoring force
Lever arm	1.50	m	
Moment	126.49	kNm/m	Restoring moment

Determine the net force and new moment for each load case:

Usual:
Net force -22.68 kN/m
Net moment -15.42 kNm/m

Unusual:
Net force -22.68 kN/m
Net moment -15.42 kNm/m

Extreme:
Net force -22.68 kN/m
Net moment -15.42 kNm/m

Unusual (Flood) Hydrostatic Parameters

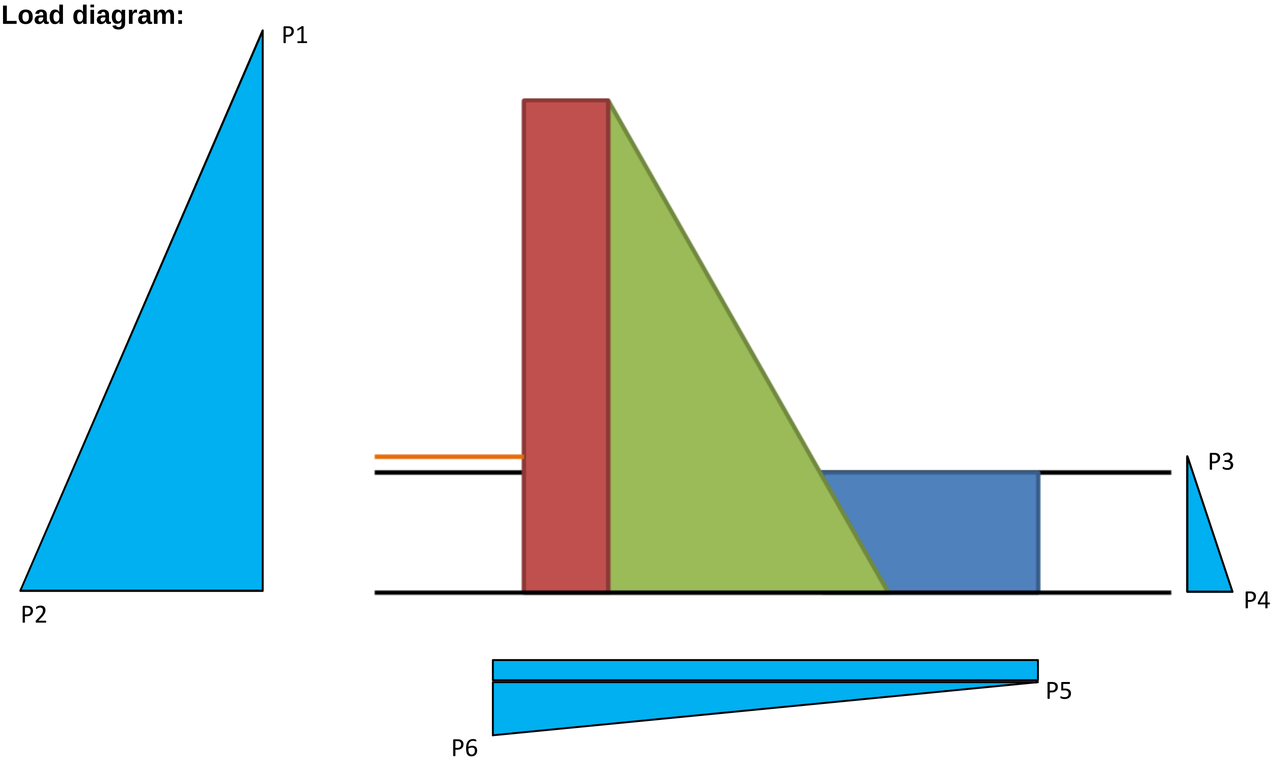
Input data

In the unusual load case, the hydrostatic (water) level is taken to be at Maximum Water Level. This is noted to be slightly conservative as the water flowing over the weir will be velocity head rather than static head.

With water flowing over the section there will be a tailwater. This will be considered for uplift calculations, however it is assumed that the tailwater does not offer a restoring force as it is being used in the energy dissipating process (USBR 1987)

Uplift is assumed to respond instantaneously to reservoir level, therefore the uplift pressures are increased. Due to the tailwater the pressure distribution is trapezoidal. The other uplift assumptions are still applicable.

Load diagram:



The hydrostatic pressures need to be calculated. The parameters to use in the calculations are as follows:

Unit Weight of Water	10	kN/m^3
Dam crest level (FSL)	615	m
EGL	577	m
Foundation level	572.5	m
Anticipated flood rise	5.4	m
Maximum water level	620.4	m
Anticipated tailwater depth	5	m
Tailwater level	582	m

Hydrostatic pressure:

In the unusual case the hydrostatic pressure occurs between MWL and foundation level.

On the upstream side:

P1	0.00	kN/m^2	
P2	479.00	kN/m^2	(hydrostatic depth x unit weight of water)

The sliding force is	11472.05	kN/m	(0.5 x P2 x hydrostatic depth)

The overturning moment depends on the lever arm about the centre and the front toe of the dam. The dam is assumed to have a level foundation so both moments will have the same lever arm.

Lever arm	15.97	m	
Overturning moment	183170.40	kNm/m	(sliding force x lever arm)

On the downstream side:

P3	0.00	kN/m^2	
P4	95	kN/m^2	(hydrostatic depth x unit weight of water)

At the downstream toe the tailwater is not considered eligible as a restoring force (see above)

Uplift pressure:

As stated previously, there is a drainage gallery. The effectiveness of the drainage is assumed to be 50%
Therefore reduction in uplift due to drainage is 0.5

This is not applicable to the hydrostatic pressure at the downstream toe

The length of the base upon which uplift pressures act 41.0 m

Pressure on the upstream side (P2)	479.00	kN/m^2
Revised for drainage (P2)	239.50	kN/m^2
Pressure on the downstream side (P4)	95.00	kN/m^2

Split the trapezoid into a rectangular and a triangular distribution and determine the uplift forces:

Rectangle:

P4	95.00	kN/m^2	
Hence uplift force is	3895.00	kN/m	(pressure x length of base)
Lever arm to centroid	0.00	m	
Moment about centroid	0.00	kNm/m	
Lever arm to d/s toe	20.50	m	
Moment about d/s toe	79847.50	kNm/m	

Triangle:

P6	144.50	kN/m^2	(difference in pressure i.e. revised P2 - P4)
Hence uplift force is	2962.25	kN/m	(0.5 x pressure x length of base)
Lever arm to centroid	6.83	m	
Moment about centroid	20242.04	kNm/m	
Lever arm to d/s toe	13.67	m	
Moment about d/s toe	40484.08	kNm/m	

Total uplift force	6857.25	kN/m
Total moment about centroid	20242.04	kNm/m
Total moment about downstream toe	120331.58	kNm/m

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Seismic - Horizontal Acceleration

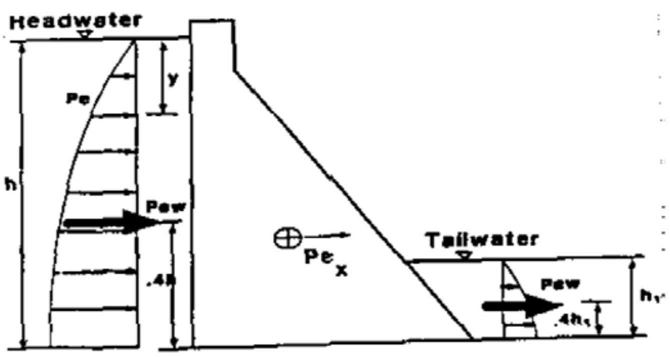
Input data

The horizontal acceleration is applied to four elements:
- Reservoir water
- Self weight of dam
- Silt
- Fill

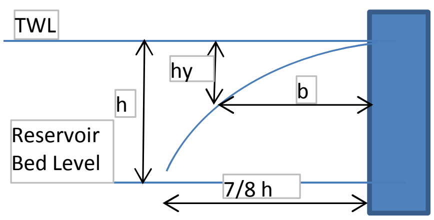
Reservoir water

Guidance on the effect of earthquakes on reservoir water was developed by Westergaard in 1933.

The seismic pressure distribution is a parabola as shown by USACE Gravity Dam Design:



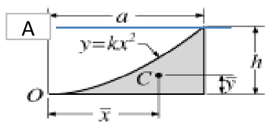
Simplified, the load diagram is:



b = 7/8 √(hy)

The shape is a parabola. The area and the centroid are given below: -

Parabolic spandrel



$\frac{3a}{4}$ $\frac{3h}{10}$ $\frac{ah}{3}$

In the above, the reservoir water is represented by the unshaded region.
To determine the moments we need to know the lever arm.

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For the rectangle:
Area = ah
Centroid x = a/2
y = h/2

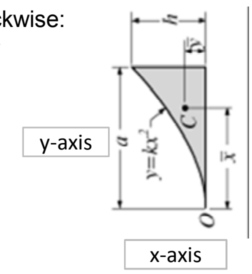
For the shaded area:
Area = ah/3
Centroid x = 3a/4
y = 3h/10

For the unshaded area:
Area = 2ah/3
Centroid: the moment of the rectangle about the origin should be the sum of the parts
ie. rectangle = shaded + unshaded

x = ah*(a/2) = (ah/3)*(3a/4) + (2ah/3)*x
x = 3a/8 About Point A 3a/8

y = ah*(h/2) = (ah/3)*(3h/10) + (2ah/3)*y
y = 3h/5 About Point A 2h/5

For this reservoir analysis, the above parabola diagram should be rotated 90 degrees anticlockwise:
Location of the centroid about the x axis is 2h/5 where h is the distance in to the reservoir
Location of the centroid about the y axis is 3a/8 where a is the height of the reservoir
****note h and a are used in this manner in relation to this diagram only****



Dam crest level (FSL)	615	m
Reservoir bed (EGL)	577	m
Foundation level	572.5	m
Depth of reservoir	38	m

Depth taken to reservoir bed. Additional water load taken in fill.

From the load diagram it can be seen that the extent of the parabola in to the reservoir is 7/8 x depth 33.25

The area of the parabola is 2ah/3 842.33 m²
The lever arm about the base (the y-axis) is 3*reservoir depth / 8 14.25 m

MCE ground acceleration: 0.24 g where g = 9.806 m/s^2

Apply a correction factor to get seismic design coefficients: 0.666667
Therefore horizontal seismic coefficient, k_h is 0.16 g
Therefore the vertical seismic coefficient, k_v is $0.5k_h$ 0.08 g

2/3MCE as recommended in Sectio 6 of BRE An Engineering Guide to Seismic Risk to Dams in UK

Density of water is 1000 kg/m^3

Therefore the force due to reservoir water using the Westergaard formula is

Reservoir water horizontal inertia force:	1321.59	kN/m

(MCE*density*area / 1000)

The associated moment is therefore:	18832.619	kNm/m

Self weight of dam

Unit weight of concrete	24	kN/m³
SI density of concrete	2447.5	kg/m^3

From previous calculations the dam is divided in to two elements:

W1	Height	42.50	m
	Width	5.20	m
	Lever arm	21.25	m above the base (Height / 2)
	Horizontal inertia force	848.64	kN/m
	Inertia moment about the base	18033.6	kNm/m

W2	Height	42.50	m
	Width	25.50	m
	Lever arm	14.17	m above the base (Height / 3)
	Horizontal inertia force	2080.8	kN/m
	Inertia moment about the base	29478	kNm/m

W3 comprises two components: $W3_{triangle}$ and $W3_{rectangle}$

$W3_{triangle}$	Height	4.5	m
	Width	2.7	m
	Lever Arm	3.00	m above the base (2*Height / 3)
Horizontal inertia force		23.328	kN/m
Inertia moment about the base		69.98	kNm/m

$W3_{rectangle}$	Height	4.5	m
	Width	10.3	m
	Lever Arm	2.25	m above the base (Height / 2)
Horizontall inertia force		177.984	kN/m
Inertia moment about the base		400.46	kNm/m

The total inertia force and moment due to self weight of dam

Self weight horizontal inertia force:	3130.752	kN/m

The associated moment is therefore:	47982.05	kNm/m

Silt

Note that silt is treated as a liquid here…

Unit weight of silt	17.00	
SI density of silt	1733.63	kg/m^3
The silt is submerged therefore revised density is	733.63	kg/m^3
Depth of silt	1.00	m
The extent of the associated parabola is 7/8h	0.875	m
The area of the associated parabola is 2ah/3	0.58	m^2/m
The lever arm about the base (the y-axis) is 3*depth / 8	0.375	m

Silt horizontal inertia force:	0.67	kN/m

The associated moment is therefore:	0.25	kNm/m

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Earth fill

Use the equations developed by Mononobe and Okobe to determine the seismic earth pressure acting on the dam.

$$P_{AE} = \tfrac{1}{2}\gamma H^2(1 - k_v)K_{AE} \qquad (8.4)$$

where K_{AE} is the active earth pressure coefficient with earthquake effect:

$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos\theta\cos^2\beta\cos(\delta + \beta + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}}\right]^2} \qquad (8.5)$$

$$\theta = \tan^{-1}\left(\frac{k_h}{1 - k_v}\right) \qquad (8.6)$$

Where…

$$k_h = \frac{\text{horiz. component of earthquake accel.}}{g}$$

$$k_v = \frac{\text{vert. component of earthquake accel.}}{g}$$

and g is acceleration due to gravity.

From equation 8.6 above, θ is 9.9 °

ϕ' =	32.50
δ =	21.67
β =	0.00
i =	0.00
θ =	9.87

φ - θ - β =	22.63
δ + β + θ =	31.53
δ + ϕ =	54.17
φ - θ - i =	22.63
i - β =	0.00

$\cos^2$ (φ - θ - β) =	0.85
$\cos^2$ β =	1.00
cos(δ + β + θ) =	0.85
sin(δ + ϕ) =	0.81
sin(φ - θ - i) =	0.38
cos(i - β) =	1.00
cos θ =	0.99

From equation 8.5, K_{AE} is 0.39

Unit weight of fill 18 kN/m^3
Depth of fill 4.5 m

From equation 8.4, P_{AE} is 66.03 kN/m

The lever arm above the base is one third of the fill depth 1.5 m

The associated moment is therefore: 99.04 kNm/m

The fill at the downstream toe has the same parameters and is restoring therefore -99.04 kNm/m

Seismic - Vertical Acceleration

Input data

There is a vertical upstream face therefore the vertical acceleration applies to one element only:
- Self weight of dam

Vertical acceleration will apply a vertical inertia load which can be taken to oppose the action of the various weights.

Self weight of dam

SI density of concrete 2447.5 kg/m^3

Vertical seismic coefficient 0.08 g where g = 9.806 m/s^2

From previous calculations the dam is divided in to two elements:

W1	Height	42.50	m
	Width	5.20	m
	Lever arm	28.1	m about the downstream toe
	Vertical inertia force	424.32	kN/m (MCE*density*area / 1000)
	Inertia moment about the base	11923.39	kNm/m
W2	Height	42.50	m
	Width	25.50	m
	Lever arm	17.00	m above the base (Height / 3)
	Vertical inertia force	1040.4	kN/m
	Inertia moment about the base	17686.8	kNm/m

W3 comprises two components: $W3_{triangle}$ and $W3_{rectangle}$

$W3_{triangle}$	Height	4.5	
	Width	2.7	
	Lever Arm	11.20	
	Vertical inertia force	11.664	kN/m
	Inertia moment about the base	130.64	kNm/m
$W3_{rectangle}$	Height	4.5	
	Width	10.3	
	Lever Arm	5.15	
	Vertical inertia force	88.992	kN/m
	Inertia moment about the base	458.31	kNm/m

The total inertia force and moment due to self weight of dam

Self weight vertical inertia force:	1565.376	kN/m
The associated moment is therefore:	30199.14	kNm/m

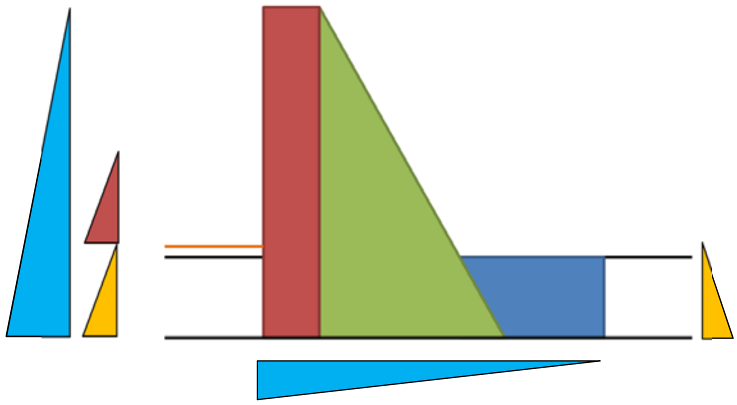
Usual Load Case

Input data

The usual load case considers the following:

Load	Usual
Dead load	Yes
Hydrostatic water load	Yes – FSL
Silt	Yes
Upstream earth	Yes - submerged
Downstream earth	Yes
Uplift	Yes - usual case
Earthquake	No

Load diagram:



From the previous calculations the following parameters have been obtained:

Total dam weight - restoring

Force:	19567.2	kN/m
Moment about centroid:	-164944.3	kNm/m
Moment about downstream toe:	-566071.9	kNm/m

Hydrostatic (usual) - destabilising

Force:	9031.3	kN/m
Moment about centroid:	127942.7	kNm/m
Moment about downstream toe:	127942.7	kNm/m

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Uplift (usual) - destabilising

Force:	4356.3	kN/m
Moment about centroid:	29767.7	kNm/m
Moment about downstream toe:	119070.8	kNm/m

Upstream fill (saturated) - destabilising

Force:	59.34	kN/m
Moment about centroid:	99.94	kNm/m
Moment about downstream toe:	99.94	kNm/m

Downstream fill - restoring

Force:	84.33	kN/m
Moment about centroid:	126.49	kNm/m
Moment about downstream toe:	126.49	kNm/m

Silt - destabilising

Force:	2.30	kN/m
Moment about centroid:	11.13	kNm/m
Moment about downstream toe:	11.13	kNm/m

Sliding failure

USBR Design of Small Dams states that the shear-friction (sliding) Factor of Safety is given by:

$$Q = \frac{CA + (\Sigma N + \Sigma U)\tan\phi}{\Sigma V} \qquad (6)$$

where:

C = unit cohesion,
A = area of section considered,
ΣN = summation of normal forces,
ΣU = summation of uplift forces,
$\tan\phi$ = coefficient of internal friction, and
ΣV = summation of shear forces.

The summation of the normal forces (weight of dam) is as reported above:
ΣN = 19567.2 kN/m

The summation of the uplift forces is as reported above:
ΣU = -4356.3 kN/m (net vertical force is therefore 15211.0 kN/m)

Foundation parameters:
The concrete/rock angle of friction ø' is 27.0 degrees (ø for rock foundation)
Therefore tanø 0.51

The summation of the shear forces (net horizontal) is hydrostatic + earth (u/s) + silt - earth (d/s):
ΣV = 9008.57 kN/m

The unit cohesion is taken to be 2.0 N/mm²

Working over an area:

Length	1000	mm
Width	41000	mm
Area	41000000	mm²

Hence the cohesive capacity (C x A) is 82000 kN

Therefore the FoS against sliding failure is **9.96** at concrete/mudstone interface

The required FoS for sliding in the usual load case is **3** **This is acceptable**

Overturning failure

About the downstream toe, the destabilising moment is generated by the hydrostatic, uplift, u/s earth and silt:
Destabilising moment 247124.6 kNm/m

About the downstream toe, the restoring moment is generated by the weight of the dam and the d/s earth:
Restoring moment -566198.4 kNm/m

The net moment is -319073.8 kNm/m i.e. restoring

The FoS against overturning at the downstream toe is **2.29** (note USBR does not consider a FoS for overturning)
(CIRIA 148 (UK) suggests FoS of 1.50)

The eccentricity of the force from the upstream toe is net moment / net vertical force (M/P):
e = 21.0 m

The dam is designed as an uncracked section. Therefore check for the presence of tension at the upstream end.

$$\sigma_z = \frac{\Sigma W}{A} \pm \frac{\Sigma M y}{I} \qquad (7)$$

where:

σ_z = normal stress on a horizontal plane,
ΣW = resultant vertical force from forces above the horizontal plane,
A = area of horizontal plane considered,
ΣM = summation of moments about the center of gravity of the horizontal plane,
y = distance from the neutral axis of the horizontal plane to where σ_z is desired, and
I = moment of inertia of the horizontal plane about its center of gravity.

ΣW resultant vertical force is as reported above: 15211.0 kN/m

The area of section is base width x unit width: 41.0 m^2

ΣM is the net moment about the centroid:

Destabilising moment is generated by hydrostatic, uplift, u/s earth and silt:	157821.49	kNm/m
Restoring moment is generated by dam weight and d/s earth:	-165070.81	kNm/m
ΣM	-7249.32	kNm/m

Z = 280.17 m^3

Therefore:

ΣW/A	371.00
ΣM/Z	-25.88

For all of the base to be in compression, ΣW/A ≥ ΣM/Z **Base in compression**

Determine resultant location

$$Resultant\ location = \frac{\Sigma M}{\Sigma V}$$

Distance of resultant force from the centroid is ΣM / ΣW = -0.48 m

Distance of middle third from the centroid is +/- base / 6 6.83 m **This is acceptable**

Foundation bearing pressure

Check the maximum foundation bearing pressure:

Using the above, the maximum bearing pressure is ΣW/A + ΣM/Z 345.12 kN/m^2

The acceptable bearing on the foundation is 4000 kN/m^2 **This is acceptable**

Maximum compressive strength

The bearing pressure can be converted into a compressive stress 0.35 N/mm^2

The limit for the concrete is 0.33f'c f'c where is the characteristic compressive strength of the concrete: 40 N/mm^2

Therefore the limit is 13.2 N/mm^2 **This is acceptable**

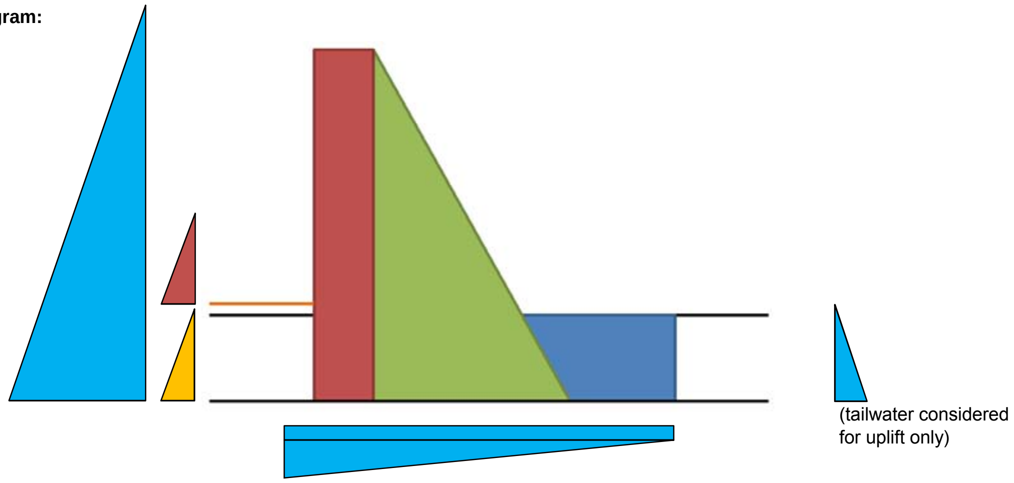
Unusual Load Case

Input data

The unusual load case considers the following:

Load	Unusual
Dead load	Yes
Hydrostatic water load	Yes – maximum water level
Silt	Yes
Upstream earth	Yes - submerged
Downstream earth	Yes - submerged
Uplift	Yes - flood tailwater
Earthquake	No

Load diagram:



From the previous calculations the following parameters have been obtained:

Total dam weight - restoring

Force:	19567.2	kN/m
Moment about centroid:	-164944.3	kNm/m
Moment about downstream toe:	-566071.9	kNm/m

Hydrostatic (flood) - destabilising

Force:	11472.1	kN/m
Moment about centroid:	183170.4	kNm/m
Moment about downstream toe:	183170.4	kNm/m

Uplift (flood) - destabilising

Force:	6857.3	kN/m
Moment about centroid:	20242.0	kNm/m
Moment about downstream toe:	120331.6	kNm/m

Upstream fill (saturated) - destabilising

Force:	59.34	kN/m
Moment about centroid:	99.94	kNm/m
Moment about downstream toe:	99.94	kNm/m

Downstream fill (saturated) - assume full erosion has occurred (concervative assumption)

Force:	0.00	kN/m
Moment about centroid:	0.00	kNm/m
Moment about downstream toe:	0.00	kNm/m

Silt - destabilising

Force:	2.30	kN/m
Moment about centroid:	11.13	kNm/m
Moment about downstream toe:	11.13	kNm/m

Sliding failure

USBR Design of Small Dams states that the shear-friction (sliding) Factor of Safety is given by:

$$Q = \frac{CA + (\Sigma N + \Sigma U)\tan\phi}{\Sigma V} \qquad (6)$$

where:

C = unit cohesion,
A = area of section considered,
ΣN = summation of normal forces,
ΣU = summation of uplift forces,
$\tan\phi$ = coefficient of internal friction, and
ΣV = summation of shear forces.

The summation of the normal forces (weight of dam) is as reported above:
ΣN = 19567.2 kN/m

The summation of the uplift forces is as reported above:
ΣU = -6857.3 kN/m (net vertical force is therefore 12710.0 kN/m)

Foundation parameters:
The concrete/rock angle of friction ø is 27.0 degrees (ø for rock foundation)
Therefore tanø 0.51

The summation of the shear forces (net horizontal) is hydrostatic + earth (u/s) + silt - earth (d/s):
ΣV = 11533.69 kN/m

The unit cohesion is taken to be 2.0 N/mm² (10% of Unconfined Compressive Strength of rock)

Working over an area:

Length	1000	mm
Width	41000	mm
Area	41000000	mm²

Hence the cohesive capacity (C x A) is 82000 kN

Therefore the FoS against sliding failure is **7.67** at concrete/mudstone interface

The required FoS for sliding in the usual load case is **3** **This is acceptable**

Overturning failure

About the downstream toe, the destabilising moment is generated by the hydrostatic, uplift, u/s earth and silt:
Destabilising moment 303613.1 kNm/m

About the downstream toe, the restoring moment is generated by the weight of the dam and the d/s earth:
Restoring moment -566071.9 kNm/m

The net moment is -262458.9 kNm/m i.e. restoring

The FoS against overturning at the downstream toe is **1.86** (note USBR does not consider a FoS for overturning)
(CIRIA 148 (UK) suggests FoS of 1.25)

The eccentricity of the force from the upstream toe is net moment / net vertical force (M/P):
e = 20.6 m

The dam is designed as an uncracked section. Therefore check for the presence of tension at the upstream end.

$$\sigma_z = \frac{\Sigma W}{A} \pm \frac{\Sigma M y}{I} \quad (7)$$

where:

σ_z = normal stress on a horizontal plane,
ΣW = resultant vertical force from forces above the horizontal plane,
A = area of horizontal plane considered,
ΣM = summation of moments about the center of gravity of the horizontal plane,
y = distance from the neutral axis of the horizontal plane to where σ_z is desired, and
I = moment of inertia of the horizontal plane about its center of gravity.

ΣW resultant vertical force is as reported above: 12710.0 kN/m

The area of section is base width x unit width: 41.0 m^2

ΣM is the net moment about the centroid:

Destabilising moment is generated by hydrostatic, uplift, u/s earth and silt:	203523.51	kNm/m
Restoring moment is generated by dam weight and d/s earth:	-164944.32	kNm/m
ΣM	38579.19	kNm/m

Z = 280.17 m^3

Therefore:

ΣW/A	310.00
ΣM/Z	137.70

For all of the base to be in compression, ΣW/A ≥ ΣM/Z **Base in compression**

ARUP	Job No. 225739	Sheet No.	Rev. Rev3
	Member/Location London		
Job Title Foxwood Dam	Drg. Ref.		
Calculation Unusual case - flood	Made by PB	Date 16/09/2014	Chd. DC

Determine resultant location

$$Resultant\ location = \frac{\Sigma M}{\Sigma V}$$

Distance of resultant force from the centroid is ΣM / ΣW = 3.04 m

Distance of middle half from the centroid is +/- base / 4 10.25 m **This is acceptable**

Foundation bearing pressure

Check the maximum foundation bearing pressure:

Using the above, the maximum bearing pressure is ΣW/A + ΣM/Z 447.70 kN/m^2

The acceptable bearing on the foundation is 4000 kN/m^2 **This is acceptable**

Maximum compressive strength

The bearing pressure can be converted into a compressive stress 0.45 N/mm^2

The limit for the concrete is 0.5f'c f'c where is the characteristic compressive strength of the concrete: 40 N/mm^2

Therefore the limit is 20 N/mm^2 **This is acceptable**

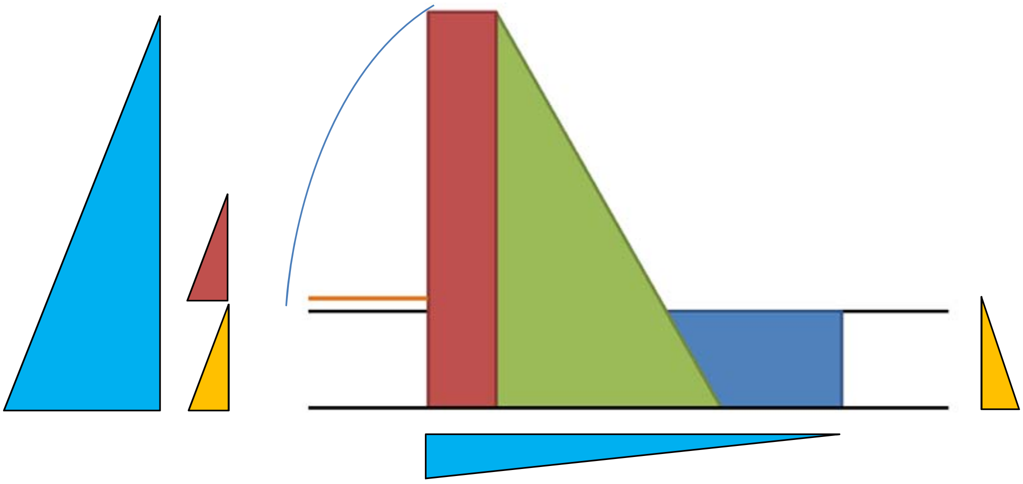
Extreme Load Case

Input data

The extreme load case considers the following:

Load	Extreme
Dead load	Yes
Hydrostatic water load	Yes – FSL
Silt	Yes
Upstream earth	Yes - submerged
Downstream earth	Yes
Uplift	Yes - usual case
Earthquake	Yes - MCE

Load diagram:



Total dam weight - restoring

Force:	19567.2	kN/m
Moment about centroid:	-164944.3	kNm/m
Moment about downstream toe:	-566071.9	kNm/m

Hydrostatic (usual) - destabilising

Force:	9031.3	kN/m
Moment about centroid:	127942.7	kNm/m
Moment about downstream toe:	127942.7	kNm/m

Uplift (usual) - destabilising

Force:	4356.3	kN/m
Moment about centroid:	29767.7	kNm/m
Moment about downstream toe:	119070.8	kNm/m

Upstream fill (saturated) - destabilising

Force:	59.34	kN/m
Moment about centroid:	99.94	kNm/m
Moment about downstream toe:	99.94	kNm/m

Downstream fill (unsaturated) - restoring

Force:	84.33	kN/m
Moment about centroid:	126.49	kNm/m
Moment about downstream toe:	126.49	kNm/m

Silt - destabilising

Force:	2.30	kN/m
Moment about centroid:	11.13	kNm/m
Moment about downstream toe:	11.13	kNm/m

Seismic - horizontal reservoir water - destabilising

Force:	1321.59	kN/m
Moment about centroid:	18832.62	kNm/m
Moment about downstream toe:	18832.62	kNm/m

Seismic - horizontal concrete - destabilising

Force:	3130.75	kN/m
Moment about centroid:	47982.05	kNm/m
Moment about downstream toe:	47982.05	kNm/m

Seismic - horizontal silt - destabilising

Force:	0.67	kN/m
Moment about centroid:	0.25	kNm/m
Moment about downstream toe:	0.25	kNm/m

Seismic - horizontal fill - destabilising

Force:	66.03	kN/m
Moment about centroid:	99.04	kNm/m
Moment about downstream toe:	99.04	kNm/m

Seismic - vertical concrete - destabilising

Force:	1565.38	kN/m
Moment about centroid:	30199.14	kNm/m
Moment about downstream toe:	30199.14	kNm/m

Sliding failure

USBR Design of Small Dams states that the shear-friction (sliding) Factor of Safety is given by:

$$Q = \frac{CA + (\Sigma N + \Sigma U) \tan \phi}{\Sigma V} \qquad (6)$$

where:

C = unit cohesion,
A = area of section considered,
ΣN = summation of normal forces,
ΣU = summation of uplift forces,
$\tan \phi$ = coefficient of internal friction, and
ΣV = summation of shear forces.

The summation of the normal forces (weight of dam) is as reported above:
ΣN = 19567.2 kN/m

The summation of the uplift forces is as reported above:
ΣU = -4356.3 kN/m

There is also the vertical concrete inertia force:
-1565.376 kN/m

(net vertical force is therefore 13645.6 kN/m)

Foundation parameters:
The concrete/rock angle of friction ø is 27.0 degrees (ø for rock foundation)
Therefore tanø 0.51

The summation of the shear forces (net horizontal) is hydrostatic + earth (u/s) + silt + seismic - earth (d/s):
ΣV = 13527.61 kN/m

The unit cohesion is taken to be 2.0 N/mm² (10% of Unconfined Compressive Strength of rock)

Working over an area:

Length	1000	mm
Width	41000	mm
Area	41000000	mm²

Hence the cohesive capacity (C x A) is 82000 kN

Therefore the FoS against sliding failure is **6.58** at concrete/mudstone interface

The required FoS for sliding in the usual load case is **1** **This is acceptable**

Overturning failure

About the downstream toe, the destabilising moment is generated by the hydrostatic, uplift, u/s earth, silt and seismic:
Destabilising moment 344237.7 kNm/m

About the downstream toe, the restoring moment is generated by the weight of the dam and the d/s earth:
Restoring moment -566198.4 kNm/m

The net moment is -221960.7 kNm/m i.e. restoring

The FoS against overturning at the downstream toe is **1.64** (note USBR does not consider a FoS for overturning)
(CIRIA 148 (UK) suggests FoS of 1.10)

The eccentricity of the force from the upstream toe is net moment / net vertical force (M/P):
e = 16.3 m

The dam is designed as an uncracked section. Therefore check for the presence of tension at the upstream end.

$$\sigma_z = \frac{\Sigma W}{A} \pm \frac{\Sigma My}{I} \qquad (7)$$

where:

σ_z = normal stress on a horizontal plane,
ΣW = resultant vertical force from forces above the horizontal plane,
A = area of horizontal plane considered,
ΣM = summation of moments about the center of gravity of the horizontal plane,
y = distance from the neutral axis of the horizontal plane to where σ_z is desired, and
I = moment of inertia of the horizontal plane about its center of gravity.

ΣW resultant vertical force is as reported above: 13645.6 kN/m

The area of section is base width x unit width: 41.0 m^2

ΣM is the net moment about the centroid:

Destabilising moment is generated by hydrostatic, uplift, u/s earth, silt and seismic:	254934.59	kNm/m
Restoring moment is generated by dam weight and d/s earth:	-165070.81	kNm/m
ΣM	89863.78	kNm/m

Z = 280.17 m^3

Therefore:

ΣW/A	332.82
ΣM/Z	320.75

For all of the base to be in compression, ΣW/A ≥ ΣM/Z **Base in compression**

Determine resultant location

$Resultant\ location = \frac{\Sigma M}{\Sigma V}$

Distance of resultant force from the centroid is ΣM / ΣW =	6.59 m	
Resultant must be within base i.e. base/2	20.5 m	This is acceptable

Foundation bearing pressure

Check the maximum foundation bearing pressure:

Using the above, the maximum bearing pressure is ΣW/A + ΣM/Z	653.57 kN/m^2	
The acceptable bearing on the foundation is allowable x 1.5	6000 kN/m^2	This is acceptable

Maximum compressive strength

The bearing pressure can be converted into a compressive stress 0.65 N/mm^2

The limit for the concrete is 1.0f'c f'c where is the characteristic compressive strength of the concrete: 40 N/mm^2

Therefore the limit is 40 N/mm^2 This is acceptable

Calculation summary

Usual case:

	Acceptable	Calculated
Sliding	3	9.96
Resultant location	Middle third	This is acceptable
Foundation bearing pressure	< allowable	This is acceptable
Maximum compressive stress	0.33f'c	This is acceptable

Note, Base in compression

Unusual case:

	Acceptable	Calculated
Sliding	2	7.67
Resultant location	Middle half	This is acceptable
Foundation bearing pressure	< allowable	This is acceptable
Maximum compressive stress	0.5f'c	This is acceptable

Note, Base in compression

Extreme case:

	Acceptable	Calculated
Sliding	1	6.58
Resultant location	Within base	This is acceptable
Foundation bearing pressure	< 1.5 x allowable	This is acceptable
Maximum compressive stress	1.0f'c	This is acceptable

Note, Base in compression

APPENDIX B: GEOTECHNICAL DESIGN PARAMETERS

ARUP

JOB TITLE	Foxwood Dam Feasibility Design
JOB NUMBER	225739
MADE BY	RG
CHECKED BY	
DATE	10/12/2014
Description of spreadsheet	Geotechnical Design Parameters
Sheet Number prefix	
Member/Location	RG / London
Drawing Reference	
Filename	\\global.arup.com\africa\South Africa\Johannesburg\Jobs\IN_Projects\Projects\225739-00

CONTENTS OF SPREADSHEET

Sheet	Description
Cover	
Notes	
Summary	Summary of parameters
Ground Model	Ground Model
Rock Param	Rock Parameters
Triax	Triaxial Test Analysis
Permeability	Permeability test summary
Filter Design	Filter Design
Rip Rap Design	Rip Rap Design

AUTHORISATION OF LATEST VERSION

Type and method of check		
Signatures & dates:	Made by	
	Checked	

REVISIONS Current Revision

Rev.	Date	Made by	Checked	Description

ARUP	Job: 225739	S	R
	Member/Location: RG / London		
Job Title: Foxwood Dam Feasibility Design	Drg.		
Calculation: Summary of parameters	Mad: RG	Date: 10/12/2014	Chd.

Balfour Mudstone

Design Parameter	Basis	Value	Unit
Unconfined Compressive Strength (UCS)	6 UCS tests (35.5MPa to 210 MPa)	40	MPa
Phi'	Assumed from Tomlinson 7th Ed. Table 2.2	27°	
c'	Correlated from UCS & RQD (Tomlinson Section 2.3.6) to 4MPa. Cripps and Taylor 1981 suggest lower bound of 2MPa for Coal Mesaures Mudstone with UCS of 9 to 103MPa.	2	MPa
Young's Modulus for Settlement	Correlated from UCS & RQD	300	MPa
Permeability	Correlated from Packer test Lugeon Values	3 x 10-6	m/s
Permeability below 27m depth	Correlated from Packer test Lugeon Values	1×10^{-7}	m/s
Grouted zone Permeability	Ciria C514, Section 6.3, lower limit permeability of rock mass grouting	1×10^{-7}	m/s
Bulk Density	From 6 UCS tests (25.6 kN/m^3 to 27kN/m^3)	26	kN/m^3
Allowable Bearing Cap	Using Tomlinson table 2.3 for cemented Mudstone.	4	MPa

Alluvial Deposits

Design Parameter	Basis	Value	Unit
Cu	Assumed with correlation from LL	90	kPa
Phi'	Correlated from Plasticity Index using BS8002 (c'=0)	31°	
Young's Modulus for Settlement	No data - assumed	5	MPa
Permeability	3 remoulded falling head tests	1×10^{-7}	m/s
Bulk Density	Average proctor compaction from 17 tests at centreline	17.5	kN/m^3

Homogeneous Embankment Fill

Asssumed from borrow pits D1, D2 and centreline, although all borrow pits show similar grading and plasticity.

Design Parameter	Basis	Value	Unit
Cu	Assumed with correlation from LL	90	kPa
Phi'	3 Triaxial Test (c' = 0)	32.5°	
Permeability	3 remoulded falling head tests	5×10^{-8}	m/s
Bulk Density	Average proctor compaction from 36 tests	18	kN/m^3
Embankment Settlement	BS6031 Code of practice for earthworks	1% of Embankment height	

Filter to Homogeneous Embankment Fill

Design Parameter	Basis	Value	Unit
Permeability	Requirment for filter design	1×10^{-4}	m/s
Bulk Density	Assumed on guidance from BS8002	21	kN/m^3
Phi'	Assumed	30°	

ARUP	Job 225739	S	R
	Member/Location RG / London		
Job Title: Foxwood Dam Feasibility Design	Drg.		
Calculation: Ground Model	Mad RG	Date 10/12/2014	Chd.

Embankment Section Ground Model

Section through embankment chosen for stability and seepage check as highest and most critical.
Section elevations Provided by JB 19/8/2014 - TYPICAL CROSS SECTIONS 1 to 3 FOR UK.pdf

Section midway between BH4a and BH3a:

	Top Depths of strata (m)		Design Depth
	BH3A	BH4A	
Clayey SILT	0	0	0
SILT with cobbles & boulders	4.5	7.9	
MUDSTONE	10	10.8	10.4

As geotechnical information below trial pit depth is limited, and the granular content of the lower superficial layer is unclear, all superficial deposits have been classed as alluvium for feasibility design.

Gravity Dam Section Ground Model

Gravity Dam founded on Mudstone as per Seismic Section and BHs Bh1A and BH2A.

	Inclined Depths of strata (m)		Top Depths of strata (m)		Design Depth
	BH1A	BH2A	BH1A	BH2A	
Sandy SILT		0		0	0
Slightly clayey SILT with cobbles & boulders	0	9.4	0	6.6	
MUDSTONE	4.5	19.7	3.2	13.9	8.6

2m cover of alluvium included as worst likely case in BH1A for seepage analysis

Reservoir Basin Valley Side Stability Model

Section locations for natural ground stability checks chosen for steepness and proximity to dam

Cross Sectio TPs refuse at 3m, assumed alluvium to 6m depth
Cross Sectio 1.5m of colluvium over siltstone / sandstone bedrock - conservatively assumed to be mudstone

Ptoential materials within foudnation excavations

Borehole	Depth of Alluvial fill (m)	Comments
BH1A	0	All cobbles
BH2A	5	possibly deeper
BH3A	4.5	
BH4A	7.9	
BH5A	0.6 to 4.2	depending on cobble content
TH1 to TH15	1.6 to 3.4	Excavator refusals in alluvium.

RQD

At dam formation level (~15m below ground level) the following RQD levels were recorded:

BH3A	75 to 100%		
BH2A	0 to 80%		
BH1A	100%	**RQD assumed as**	**75%**

Unconfined Compressive Strength (UCS)

Borehole	Description	Depth	UCS (MPa)	
BH1A	Moderately weathered mudstone	6.5	35.5	shallow
BH2A	Slightly weathered mudstone	22.8	42.9	
BH3A	Slightly weathered mudstone	10.8	149.1	
BH4A	Moderately weathered mudstone	14	111.7	
BH5A	Slightly weathered mudstone	7.3	155.2	
	UCS of Mudstone at 15m depth =		**40**	

Borehole	Description	Depth	UCS (MPa)
BH6A	Slightly weathered **sandstone**	11.4	210.1

Mass strength for bearing and stability (c and phi')

Correlated from UCS & RQD (Tomlinson Section 2.3.6)

Phi' from table 2.2 = **27 degrees**

c = 0.1 * UCS = 4 MPa

Cripps and Taylor 1981 suggest lower bound of 2MPa for Coal Mesaures Mudstone with UCS of 9 to 103MPa.

c = **2 MPa**

Young's Modulus

Tomlinson Eq. 2.54 Em = j * Mr * quc

RQD =	75%	
J =	0.5	
Mr =	150	
quc =	40	MPa
Em =	3000	MPa

Data in Cripps and Taylor (1981) suggest on the order of 100 to 500 MPa for unweathered Jurrasic Mudstones.

Em = 300 MPa

Presumed Bearing Capacity

Using Tomlinson table 2.3 for cemented Mudstone:

Property	Rating	Basis
Strength =	moderately strong	25-50MPa as per Hoek ch11 table 2
Discontinuity Spacing	600 to 1000mm	Core photographs
Presumed allowable bearing		4000 to 6000 kPa

Note: Above allowable bearing pressure are for 50mm settlement, less settlment would be tollerated by the weir. Bearing pressures must not exceed the UCS of rock if the joints are tight.

Allowable bearing capactiy **4000 kPa**

Permeability

from Quinones-Rozo 2010

Lugeon Range	Classification	Hydraulic Conductivity Range (cm/sec)	Condition of Rock Mass Discontinuities	Reporting Precision (Lugeons)
<1	Very Low	$< 1 \times 10^{-5}$	Very tight	<1
1-5	Low	1×10^{-5} - 6×10^{-5}	Tight	± 0
5-15	Moderate	6×10^{-5} - 2×10^{-4}	Few partly open	± 1
15-50	Medium	2×10^{-4} - 6×10^{-4}	Some open	± 5
50-100	High	6×10^{-4} - 1×10^{-3}	Many open	± 10
>100	Very High	$> 1 \times 10^{-3}$	Open closely spaced or voids	>100

	Representative Lugeon Value						
Depth (m)	BH1A*	BH2A*	BH3A	BH4A	BH5A	All	k (m/s)
6.0 - 11.0	37	No tests - soft soil			0	0	1 x 10-7
11.0 - 17.0	0	39	0	20	37	37	5 x 10-6
17.0 - 22.0	0	56	10	16	2	20	3 x 10-6
22.0 - 27.0	Depth not attaine	Depth not attained	21	14	22	20	3 x 10-6
27.0 - 30.0	Depth not attaine	Depth not attained	1	22	3	20	3 x 10-6

*angled hole

References

Cripps and Taylor 1981. The engineering properties of mudrocks. Quarterly Journal of Engineering Geology and Hydrogeology. V14. p325

Tomlinson 2001. Foundation Design and Construction. 7th Edition.

Remoulded Consolidated Undrained Triaxial tests

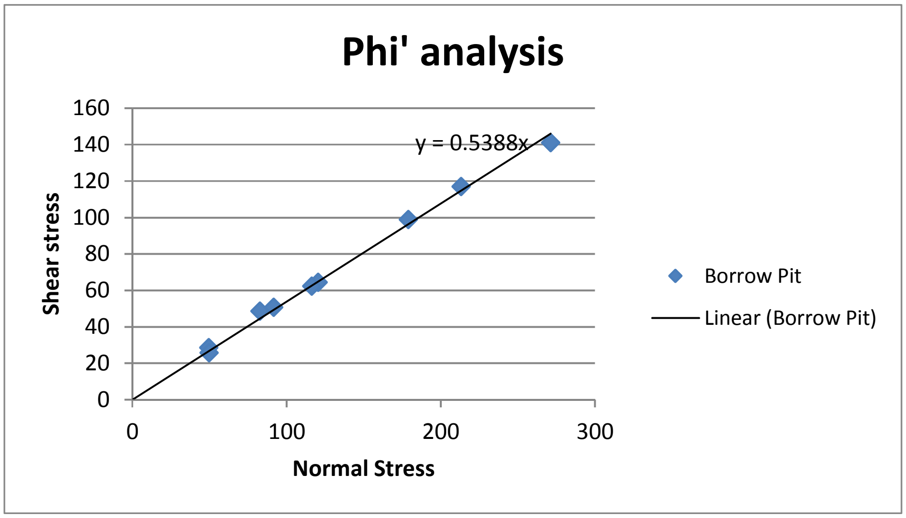
The following correlates all triaxial testing for embankment fill materials to obtain average phi'

Borehole	Depth (m)	(s1+s3)/2	(s1-s3)/2	s1	s3
D1-11	0.4	82.65	48.65	131.3	34
		120.5	64.5	185	56
		213.1	117.1	330.2	96
D1-19	0.8	49.55	28.55	78.1	21
		91.7	50.7	142.4	41
		178.85	98.85	277.7	80
D2-5	1.4	49.75	25.75	75.5	24
		116.4	62.4	178.8	54
		271.15	141.15	412.3	130

From chart:

Tan α' = 0.5388 φ' = 0.56901202 radians
a' = 0

φ' = 32.60 degrees
c' = 0.00 kN/m^2



Remoulded borrow pit samples

TRIAL HOLE	SAMPLE	DEPTH (m)	DESCRIPTION	DENSITY (Kg/m³)	PERMEABILITY (m/s)
D1-11	GT429/D1-11/FH	0.4	Clayey silt	1491	1.47E-08
D1-19	GT429/D1-19/FH	0.8	Clayey silt	1690	7.68E-08
D2-5	GT429/D2-5/FH	1.4	Slightly clayey silt	1859	7.36E-09
				Average	**3.00E-08**

Remoulded Alluvium Samples

TRIAL HOLE	SAMPLE	DEPTH (m)	DESCRIPTION	DENSITY (Kg/m³)	PERMEABILITY (m/s)
TH2	GT429/TH2/FH	0.8	Slightly clayey silt	1754	5.43E-08
TH4	GT429/TH4/FH	1.5	Slightly clayey silt	1721	<1E-10
TH6	GT429/TH6/FH	1.9	Silt	1957	2.14E-07
				Average	**9.00E-08**

Particle size of filter material against Homogeneous Embankment Fill

Sherard et al 1984 (as quoted in BRE guide and US Department of Interior Oct. 1991):

Based on the above, Sherard et al. (1984b) state that sand or gravelly sand filters with D_{15} = 0.5 mm or smaller will safely control and seal concentrated leaks through most dispersive clays with d_{85} larger than about 0.03 mm. Sand filters with D_{15} = 0.2 mm or smaller are conservative for the very finest dispersive clays. For clays having similar particle size distribution, whether dispersive or nondispersive, the required filters are the same.

where:

D_{15} = particle size in the filter of which 15 percent are smaller, by dry mass of soil
d_{85} = particle size of the base soil of which 85 percent are smaller, by dry mass of soil

The filter should be noncohesive to be effective when cracks form. If it is not, the filter could sustain an open crack and fail to protect the cracked core. Similar design criteria can be used if a geotextile is used for the filter element.

98% of Trial Pit samples had a d_{85} larger than 0.2mm. Therefore recommended D_{15} of 0.5mm is acceptable for the sand filter.

Sherard and Dunnigan (1989) also have the following criteria for materials with fines content (passing 0.075mm) of 40-85%:

$$D_{15} < 0.7\text{mm}$$

Uniformity of filter material

In addition, the uniformity of the filter should be specified in accordance with Ciria C551 Section 5.6.2

In addition, a uniformity criterion (also called a geometrically tight criterion) for the filter itself is often specified, to ensure that the finer particles of the filter are not removed through the voids between the coarser particles. This is particularly important where hydraulic loadings are high, such as occur in turbulent flow conditions.

For uniformity: $\dfrac{d_{60}}{d_{10}} \leq 10$

Permeability of filter material

Briddle (2008) provides figure 8 below which suggests that the filter needs a permeability less than 1 x 10^{-3} m/s to retain the embankment material with fines content of 40-60%. Therefore a design permeability of 1 x10^{-4} m/s is recommended.

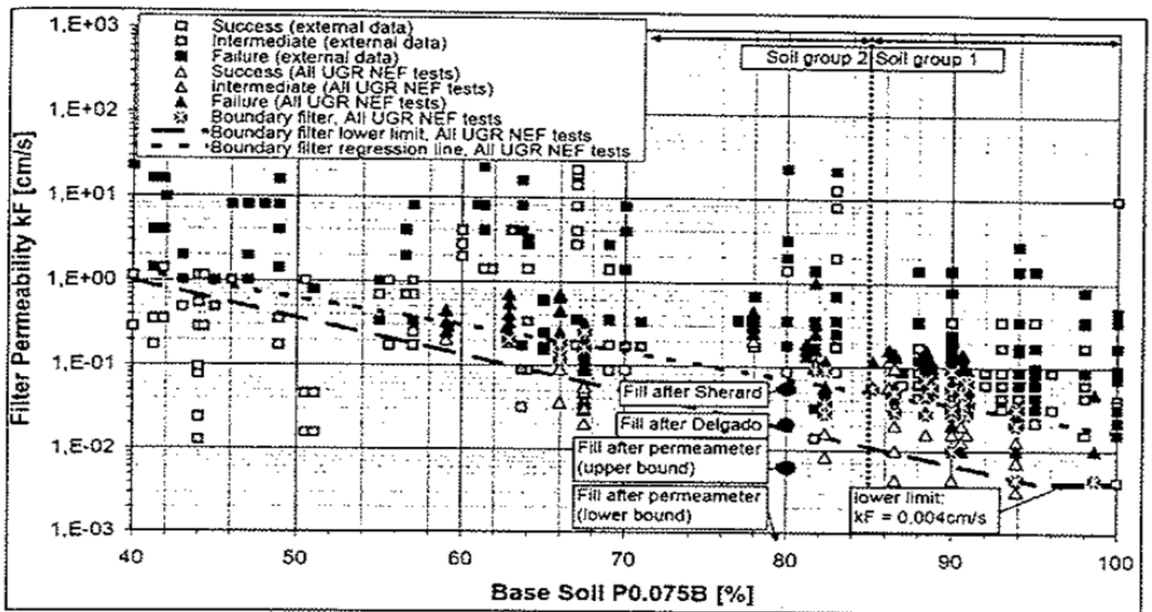


Figure 8: Delgado relationship between % base soil passing 0.075 mm sieve and filter permeability [3]

Bridle, R. Assessing the vulnerability of a typical British embankment dam to internal erosion. In; *Ensuring reservoir safety into the future* 2008 Thomas Telford

ARUP	Job 225739	S	R
	Member/Location RG / London		
Job Title: Foxwood Dam Feasibility Design	Drg.		
Calculation: Rip Rap Design	Mad RG	Date 10/12/2014	Chd.

CIRIA C683 Rock Manual

Hudson 1959 method - no damage, K_D=3.5, H=H_s (p. 564)

Parameter	Symbol	Equation	Unit	Value	Comment
Rock density	ρr		kg/m³	2700	Assumed Dolerite rock density from *Engineering Geology* (Bell 2007)
Wave height	H		m	2.30	1:100 yrs from freeboard calc
Stability coefficient	KD			3.50	For rough, angular, randomly placed armourstone in two layers on a breakwater trunk for *breaking waves on the foreshore*
Density of water	ρw		kg/m³	1000.00	
Relative density of stone	Δ	$\rho_r / \rho_w - 1$		1.70	
Slope angle	α		°	14.04	1 in 4
cotα				4.00	cotα = 1 / tanα
Median weight of armourstone	W_{50}	Hudson formula: $W_{50} = (\rho_r g H^3) / (K_D \Delta^3 \cot\alpha)$	N	4685.34	
Median mass of armourstone	M_{50}		kg	477.61	
Median stone diameter	$D_{n50\ armour}$	$(M_{50}$/Pr)^1/3	m	0.56	p. 107 of Rock Manual

Dn50 of filter layer

Parameter	Symbol	Equation	Unit	Value	Comment
D_{n50} of filter layer	$D_{n50\ filter}$	$D_{n\ armour}/D_{n\ filter} = 2.0$	m	0.28	See Fig 5.39 page 568

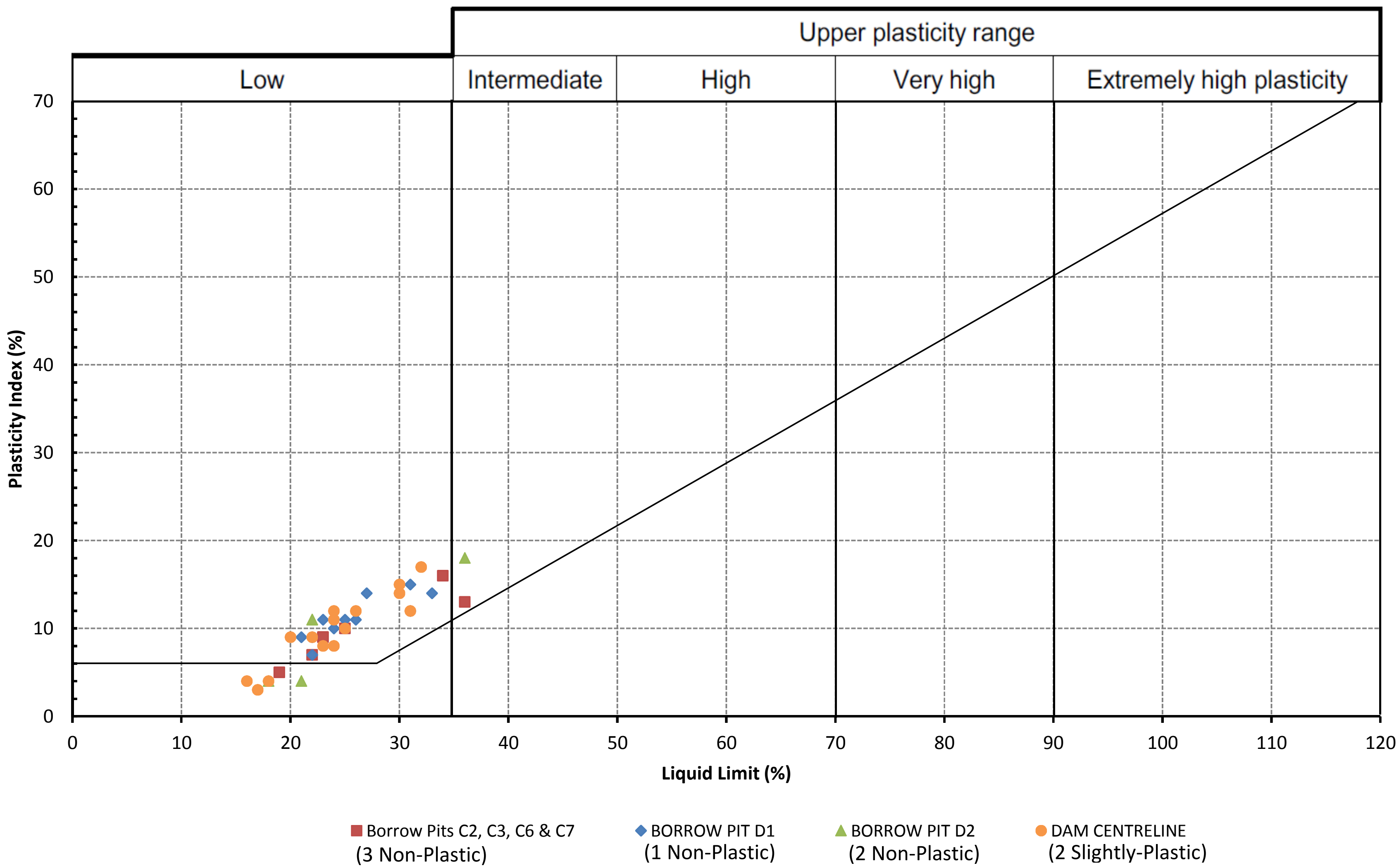
Layer thickness

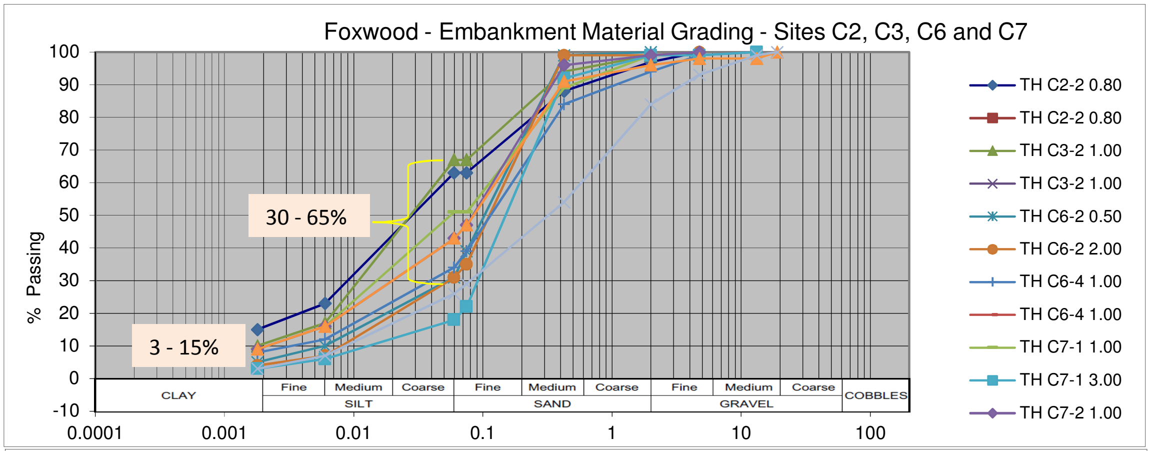
Parameter	Symbol	Equation	Unit	Value	Comment
Thickness of armour layer		2 x Dn50armour	m	1.12	See Fig 5.39 page 568
Thickness of filter layer		1.5 x Dn50armour	m	0.84	See Fig 5.39 page 568

As a check determine Ns

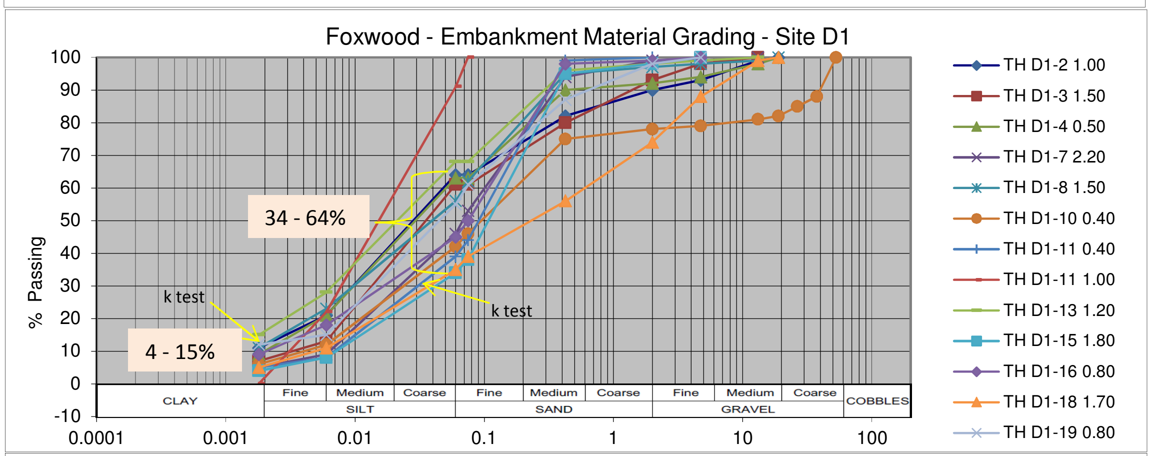
Parameter	Symbol	Equation	Unit	Value	Comment
Stability number	Ns	$H_s/\Delta D_{n50}$		2.41	1<Ns<4 therefore statically stable, see page 559

Foxwood Embankment - Plasticity Chart

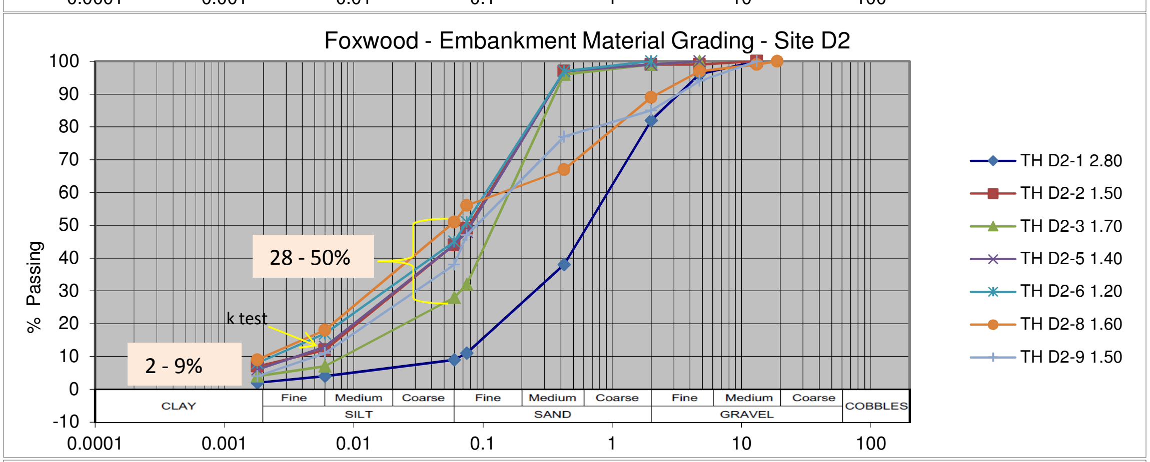




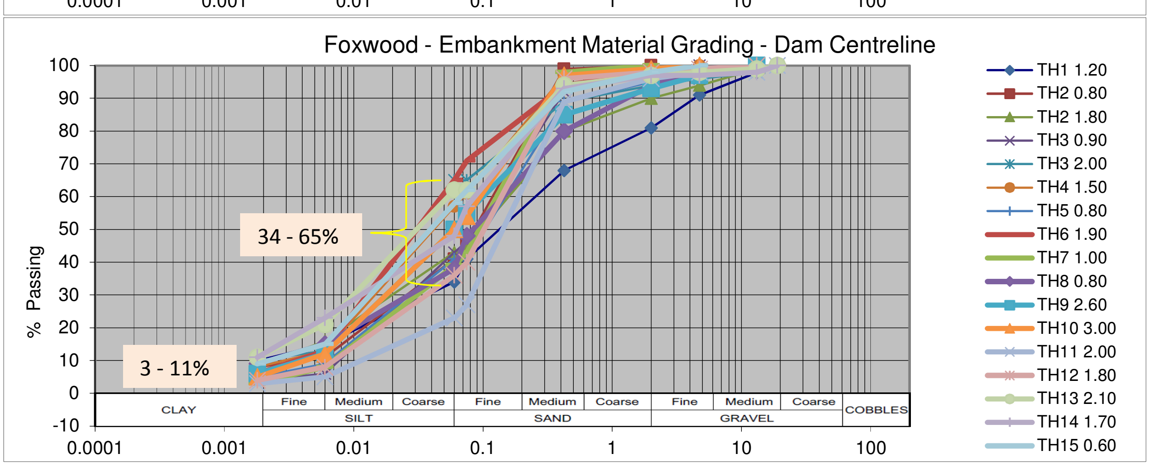
Foxwood - Embankment Material Grading - Sites C2, C3, C6 and C7
% Passing
30 - 65%
3 - 15%
CLAY
Fine
Medium
Coarse
SILT
SAND
GRAVEL
COBBLES
0.0001
0.001
0.01
0.1
1
10
100
TH C2-2 0.80
TH C2-2 0.80
TH C3-2 1.00
TH C3-2 1.00
TH C6-2 0.50
TH C6-2 2.00
TH C6-4 1.00
TH C6-4 1.00
TH C7-1 1.00
TH C7-1 3.00
TH C7-2 1.00



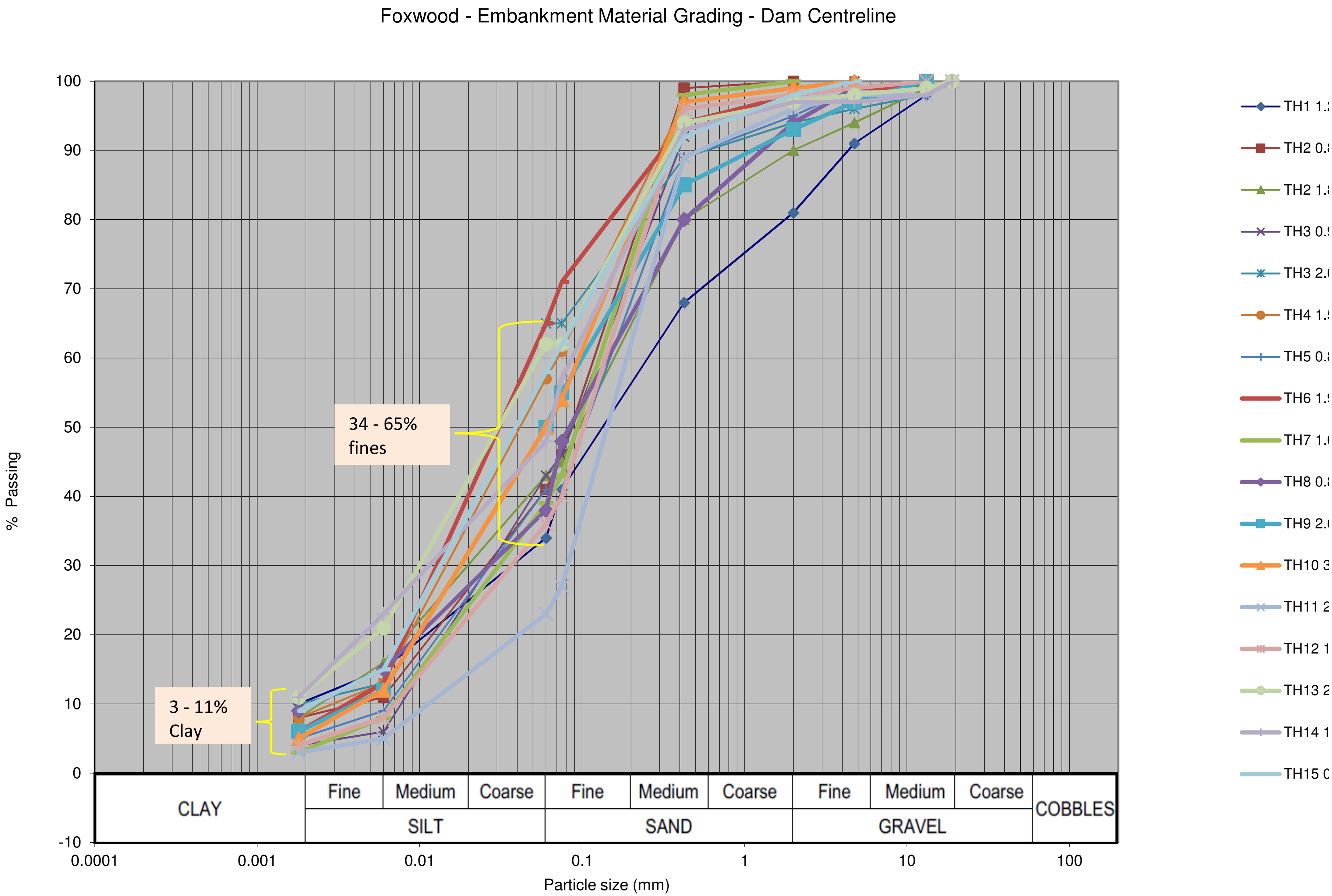
Foxwood - Embankment Material Grading - Site D1
% Passing
34 - 64%
4 - 15%
k test
k test
CLAY
Fine
Medium
Coarse
SILT
SAND
GRAVEL
COBBLES
TH D1-2 1.00
TH D1-3 1.50
TH D1-4 0.50
TH D1-7 2.20
TH D1-8 1.50
TH D1-10 0.40
TH D1-11 0.40
TH D1-11 1.00
TH D1-13 1.20
TH D1-15 1.80
TH D1-16 0.80
TH D1-18 1.70
TH D1-19 0.80



Foxwood - Embankment Material Grading - Site D2
% Passing
28 - 50%
2 - 9%
k test
CLAY
Fine
Medium
Coarse
SILT
SAND
GRAVEL
COBBLES
TH D2-1 2.80
TH D2-2 1.50
TH D2-3 1.70
TH D2-5 1.40
TH D2-6 1.20
TH D2-8 1.60
TH D2-9 1.50



Foxwood - Embankment Material Grading - Dam Centreline
% Passing
34 - 65%
3 - 11%
CLAY
Fine
Medium
Coarse
SILT
SAND
GRAVEL
COBBLES
TH1 1.20
TH2 0.80
TH2 1.80
TH3 0.90
TH3 2.00
TH4 1.50
TH5 0.80
TH6 1.90
TH7 1.00
TH8 0.80
TH9 2.60
TH10 3.00
TH11 2.00
TH12 1.80
TH13 2.10
TH14 1.70
TH15 0.60


Foxwood - Embankment Material Grading - Dam Centreline
% Passing
Particle size (mm)
34 - 65% fines
3 - 11% Clay
CLAY
Fine
Medium
Coarse
SILT
Fine
Medium
Coarse
SAND
Fine
Medium
Coarse
GRAVEL
COBBLES
TH1 1.
TH2 0.
TH2 1.
TH3 0.
TH3 2.
TH4 1.
TH5 0.
TH6 1.
TH7 1.
TH8 0.
TH9 2.
TH10 3
TH11 2
TH12 1
TH13 2
TH14 1
TH15 0

APPENDIX C: SLOPE STABILITY DESIGN REGISTER

ARUP

JOB TITLE	Foxwood Dam Feasibility Design
JOB NUMBER	225739
MADE BY	RG
CHECKED BY	
DATE	23/09/2014
Description of spreadsheet	Slope Stability Run Register
Sheet Number prefix	
Member/Location	RG / London
Drawing Reference	
Filename	\\global.arup.com\africa\South Africa\Johannesburg\Jobs\IN_Projects\Projects\225739-00

CONTENTS OF SPREADSHEET

Sheet	Description
Cover	
Notes	
Slope Register	Slope Register
Assumptions	Design Assumptions
Run Images	Run Images
Sheet2	

AUTHORISATION OF LATEST VERSION

Type and method of check		
Signatures & dates:	Made by	
	Checked	

REVISIONS Current Revision

Rev.	Date	Made by	Checked	Description

SLOPE/W Outputs **Bold Text are the latest design files**

Run #	Location	Description	PWP	Slip Method	Analysis	Combination	FoS	Tagret FoS
Slope_1	Embankment	Stability of upstream slope 1:3	Drawdown	Grid and radius	Drained	Global FoS	1.24	1.3
Slope_2	Embankment	Stability of downstream slope 1:2.5	MWL - 620.4	Grid and radius	Drained	Global FoS	1.60	1.5
Slope_3	Embankment	Stability of upstream slope - construction		Grid and radius	Undrained	Global FoS	1.01	1.3
Slope_4 - Not undertaken		Stability of downstream slope - construction		Grid and radius	Undrained	Global FoS		1.3
Slope_5 - Not undertaken		Stability of upstream slope - OBE	MWL - 620.4	Grid and radius	Drained	Seismic OBE		1.1
Slope_6	Embankment	Stability of downstream slope - OBE	MWL - 620.4	Grid and radius	Drained	Seismic OBE	1.40	1.1
Slope_7		Stability of upstream slope - MDE	TWL - 615	Grid and radius	Drained	Seismic MDE		1
Slope_8	Embankment	Stability of downstream slope - MDE	TWL - 615	Grid and radius	Drained	Seismic MDE	0.91	<1
Slope_9	Embankment	As Run 8 with gwl 1m below surface at toe	TWL - 615	Grid and radius	Drained	Seismic MDE	0.97	<1
		UPSTREAM SLOPE SLACKENED to 1:4						
Slope_10	**Embankment**	**Stability of upstream slope 1:4**	Drawdown	Grid and radius	Drained	Global FoS	1.40	1.3
Slope_11		Not used						
Slope_12	**Embankment**	**Stability of upstream slope - construction**		Grid and radius	Undrained	Global FoS	1.30	1.3
Slope_13	Embankment	Stability of downstream slope - construction		Grid and radius	Undrained	Global FoS	1.19	1.3
		DOWNSTREAM SLOPE SLACKENED to 1:3 (+ berms)						
Slope_14	**Embankment**	**As Run 13 with 1:3 d/s slope**		Grid and radius	Undrained	Global FoS	1.29	1.3
Slope_15	**Embankment**	**Stability of downstream slope**	MWL - 620.4	Grid and radius	Drained	Global FoS	1.94	1.5
Slope_16	**Embankment**	**Stability of upstream slope - OBE**	MWL - 620.4	Grid and radius	Drained	Seismic OBE	1.73	1.1
Slope_17	**Embankment**	**Stability of downstream slope - OBE**	MWL - 620.4	Grid and radius	Drained	Seismic OBE	1.66	1.1
Slope_18	**Embankment**	**Stability of upstream slope - MDE**	TWL - 615	Grid and radius	Drained	Seismic MDE	0.74	<1
Slope_19	**Embankment**	**Stability of downstream slope - MDE**	TWL - 615	Grid and radius	Drained	Seismic MDE	1.02	<1
Slope_20	**Embankment**	**Stability of upstream slope**	TWL - 615	Grid and radius	Drained	Global FoS	2.50	1.5
Slope_21	Embankment	Stability of upstream slope - 16%g	TWL - 615	Grid and radius	Drained	Global FoS	1.00	1
Slope_22	Embankment	As Run 18 with 1:5 u/s slope - MDE	TWL - 615	Grid and radius	Drained	Global FoS	0.82	<1
Slope_30	**Valley side 4**	**Section 4 on drawdown**	Drawdown	Grid and radius	Drained	Global FoS	1.30	1.3
Slope_31	**Valley side 5**	**Section 5 on drawdown**	Drawdown	Grid and radius	Drained	Global FoS	0.90	1.3

Parameters used in slope stability analysis

Drained Paramters

Material	weight above & below GWL	Condition	phi'	c'	k	yo	cu/p
Alluvium	17.5	Drained - linear strength	31	0	0	0	0
Mudstone	26	Drained - linear strength	27	2000	0	0	0
Embankment Fill	18	Drained - linear strength	32.5	0	0	0	0
Filter	18	Drained - linear strength	32	0	0	0	0
Rockfill	20	Drained - linear strength	40	0	0	0	0

Undrained Parameters

Material	weight above & below GWL	Condition	phi'	c	k	yo	cu/p
Alluvium	17.5	Drained - linear strength		90	0	0	0
Mudstone	26	Drained - linear strength	27	2000	0	0	0
Embankment Fill	18	Drained - linear strength		90	0	0	0
Filter	18	Drained - linear strength	32	0	0	0	0
Rockfill	20	Drained - linear strength	40	0	0	0	0

Design Basis

Max Water level (MWL) of 620.4
Top Water Level (TWL) of 615
OBE: 0.05g
MDE: 0.24g (as per the GI report, max credible earthquake in the SA)

For ground model build up of the embankment section see Geotechnical Parameters calculation

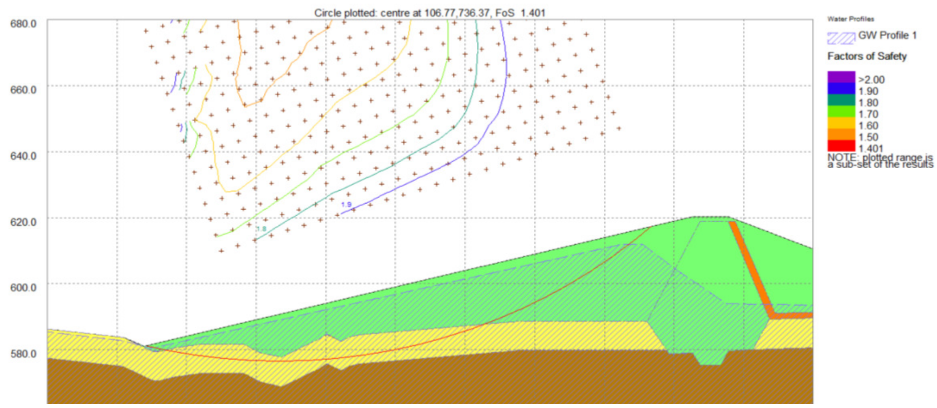
Valley slope sections groud model

Section locations: ACAD-C3D_GX_CROSS SECTION_ locations.pdf
Topography provided by M Hilton 29/08/2014 based on steep slopes opposite to the dam:
Cross Section 4_.xls TPs refuse at 3m, assumed alluvium to 6m depth
Cross Section 5_.xls 1.5m of colluvium over siltstone / sandstone bedrock - conservatively assumed to be mudstone

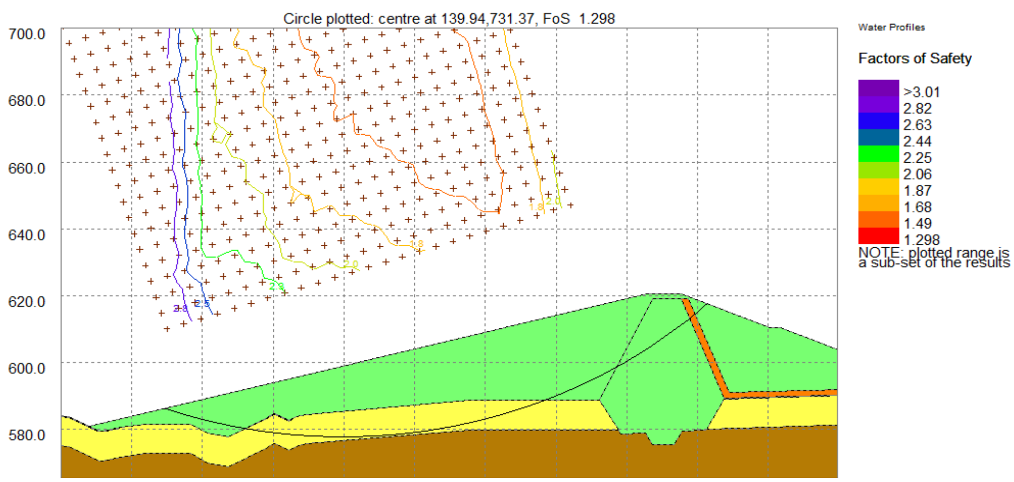
Relevant BHs: BH Map.pdf

ARUP	Job	S	R
	225739		
	Member/Location	RG / London	
Job Title: Foxwood Dam Feasibility Design	Drg.		
Calculation: Run Images	Mad RG	D 23/09/2014	C

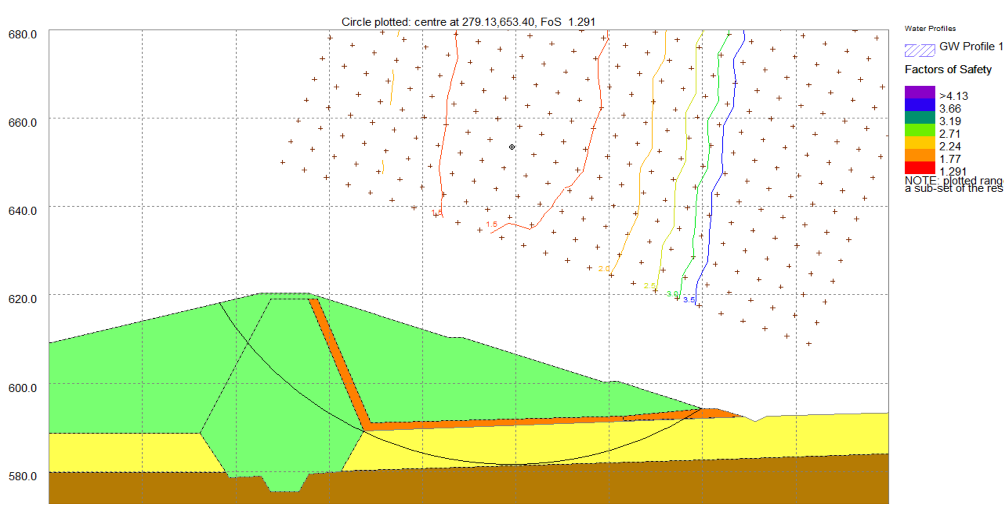
Slope Run 10



Slope Run 12

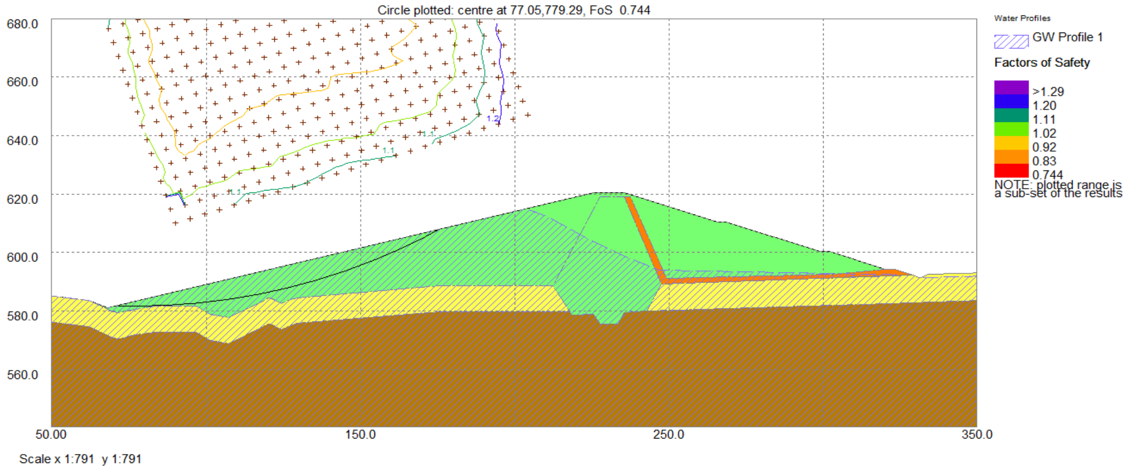


Slope Run 14

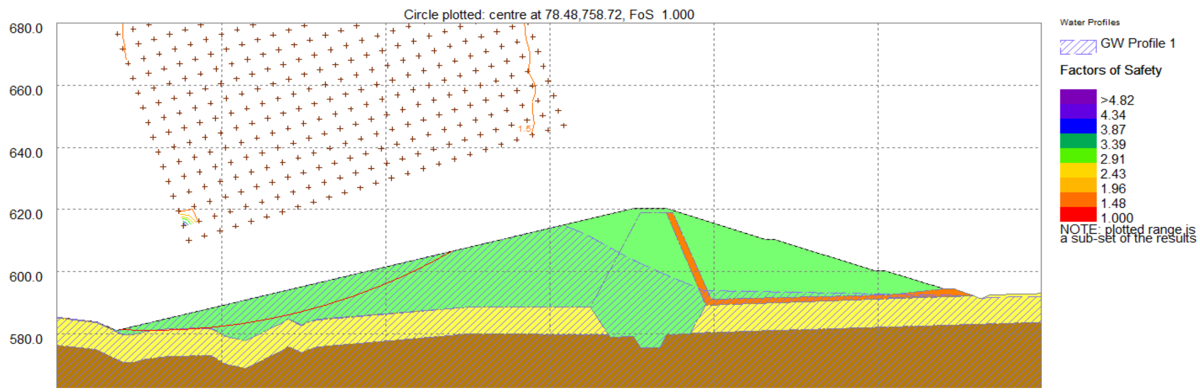


ARUP	Job 225739	S	R
	Member/Location RG / London		
Job Title Foxwood Dam Feasibility Design	Drg.		
Calculation Run Images	Mad RG	D 23/09/2014	C

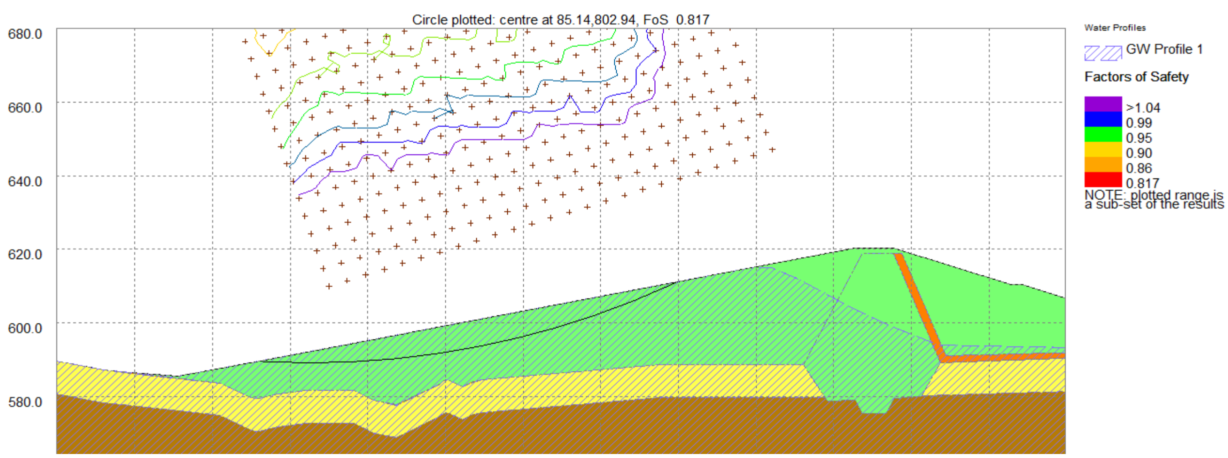
Run 18



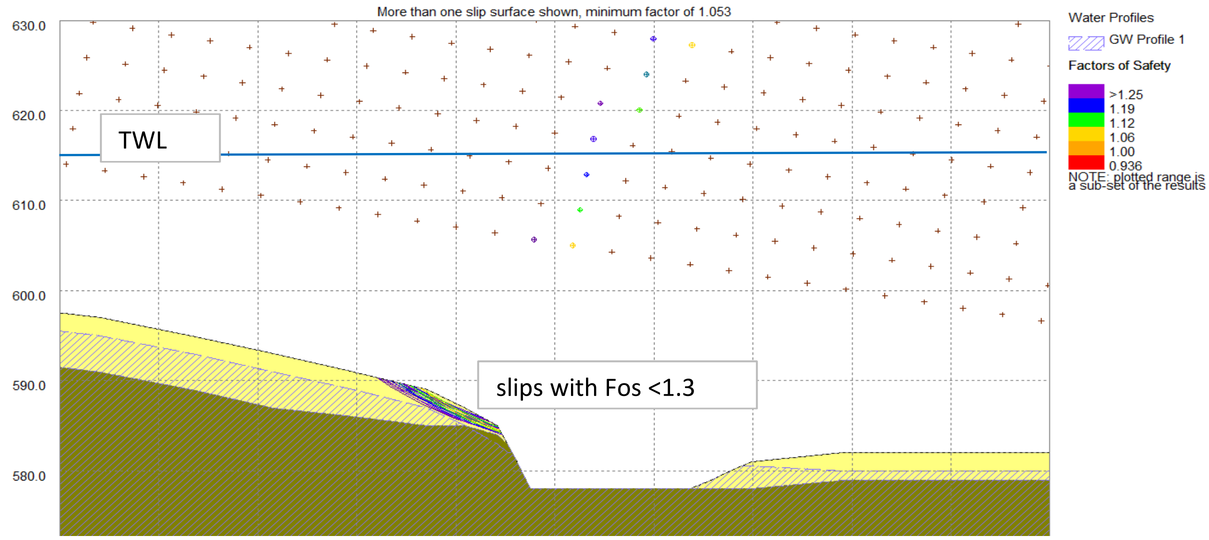
Run 21



Run 22

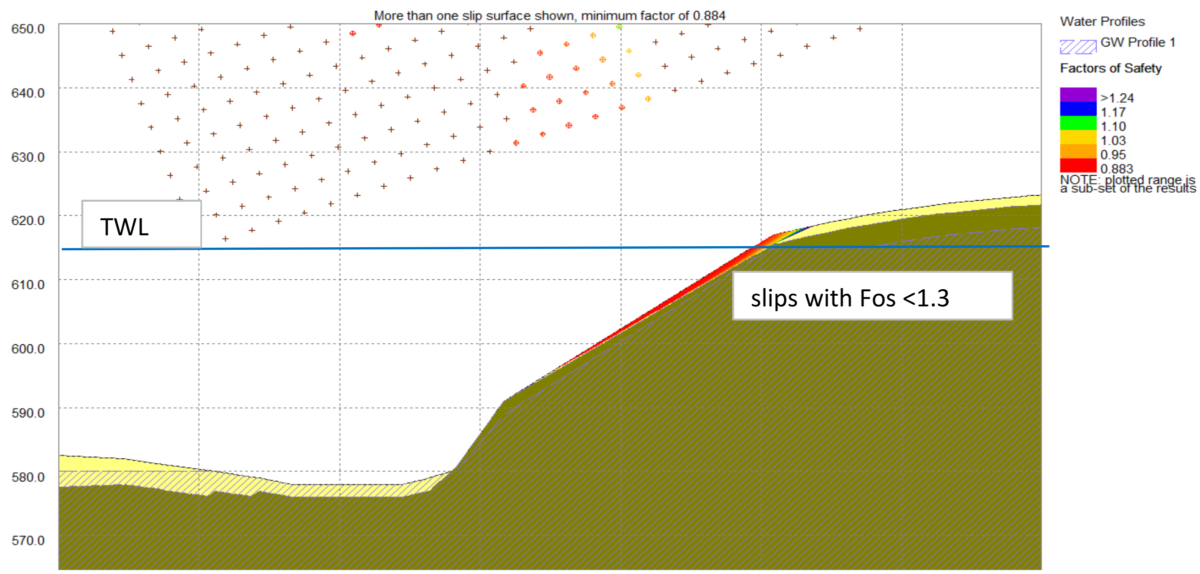


Run 30



More than one slip surface shown, minimum factor of 1.053
630.0
620.0
610.0
600.0
590.0
580.0
TWL
slips with Fos <1.3
Water Profiles
GW Profile 1
Factors of Safety
>1.25
1.19
1.12
1.06
1.00
0.936
NOTE: plotted range is a sub-set of the results

Run 31



More than one slip surface shown, minimum factor of 0.884
650.0
640.0
630.0
620.0
610.0
600.0
590.0
580.0
570.0
TWL
slips with Fos <1.3
Water Profiles
GW Profile 1
Factors of Safety
>1.24
1.17
1.10
1.03
0.95
0.883
NOTE: plotted range is a sub-set of the results

APPENDIX D: PDISP SETTLEMENT DESIGN REGISTER

ARUP

JOB TITLE	Foxwood Dam Feasibility Design
JOB NUMBER	225739
MADE BY	RG
CHECKED BY	
DATE	10/12/2014
Description of spreadsheet	Foxwood pdisp calc register
Sheet Number prefix	
Member/Location	
Drawing Reference	
Filename	\\global.arup.com\africa\South Africa\Johannesburg\Jobs\IN_Projects\Projects\225739-00

CONTENTS OF SPREADSHEET

Sheet	Description
Cover	
Notes	
Calc Register	Calc Register
Model Build Up	Model Build Up
Run Images	Run Images
Sheet2	

AUTHORISATION OF LATEST VERSION

Type and method of check

Signatures & dates: Made by

Checked

REVISIONS Current Revision

Rev.	Date	Made by	Checked	Description

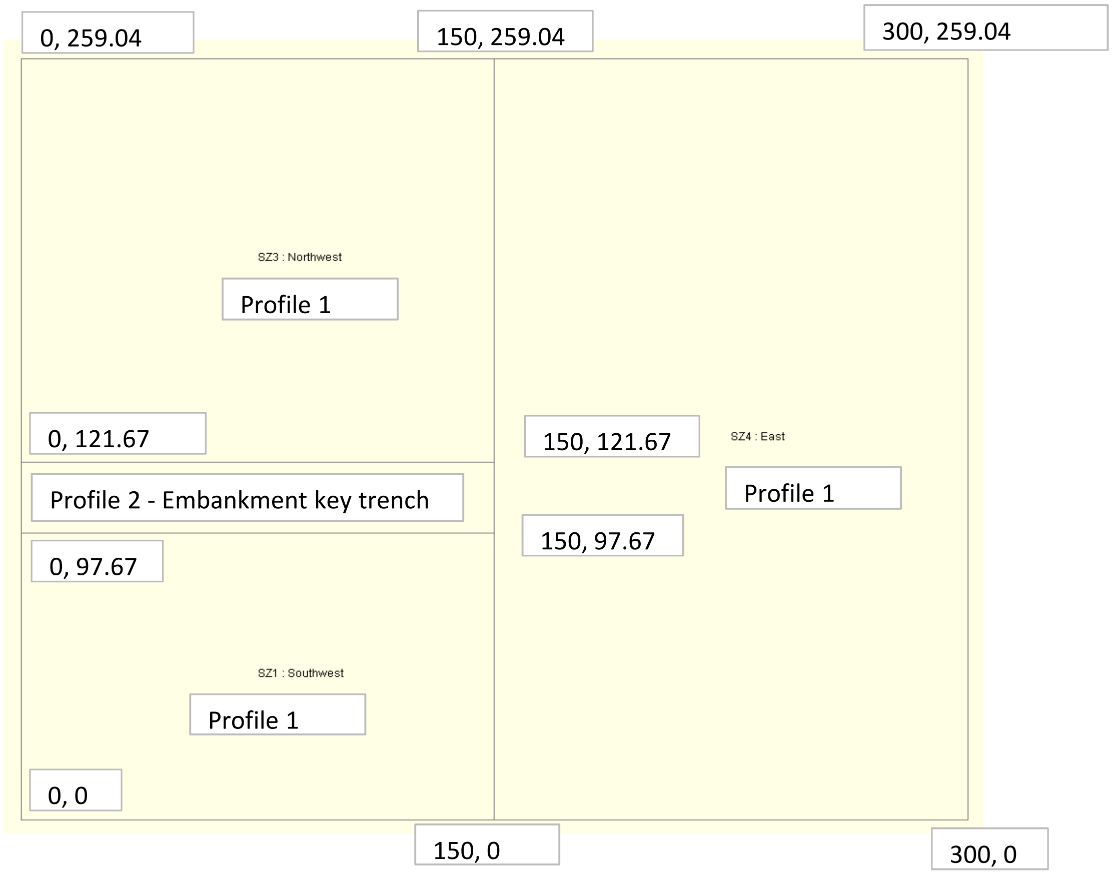
ARUP	Jo	S	Rev.
	225739		
	Member/Location		
Job Title: Foxwood Dam Feasibility Design	Dr		
Calculation: Calc Register	Ma RG	D 10/12/2014	C

Settlement analysis has been carried out in order to establish the scale of settlement expected around the embankment and concrete gravity dam interface. Modelling has been undertaken in Oasys pdisp.

pdisp Outputs

Run #	Date	Description	Comments	File name
pdisp_1	09/09/2014	Model of wall - embankment interface	950mm max settlement	Pdisp 1.pdd
	Em of Mudstone changed from 3000 to 300MPa following L S-G Comments and Review of Cripps and Taylor			
pdisp_2	09/09/2014	Model of wall - embankment interface	1000mm max settlement	Pdisp 2.pdd
	Location of wall-embnakment interface following lengthening of spillway and modelling of interface wall			
pdisp_3	**08/12/2014**	**Model of relocated wall-embankment interface following lengthening of spillway**		Pdisp 3.pdd

Soil Zone Plan Coordinates



Soil Zone Profiles

z = 0m at existing groud level

Profile 1 (Natural groud)

Material	Level at top (m)	Youngs mod (kN/m2)	Poisson's Ratio	Basis
Colluvium	0	5000	0.2	No data - assumed
Mudstone	-8.8	300000	0.2	Correlated from UCS & RQD

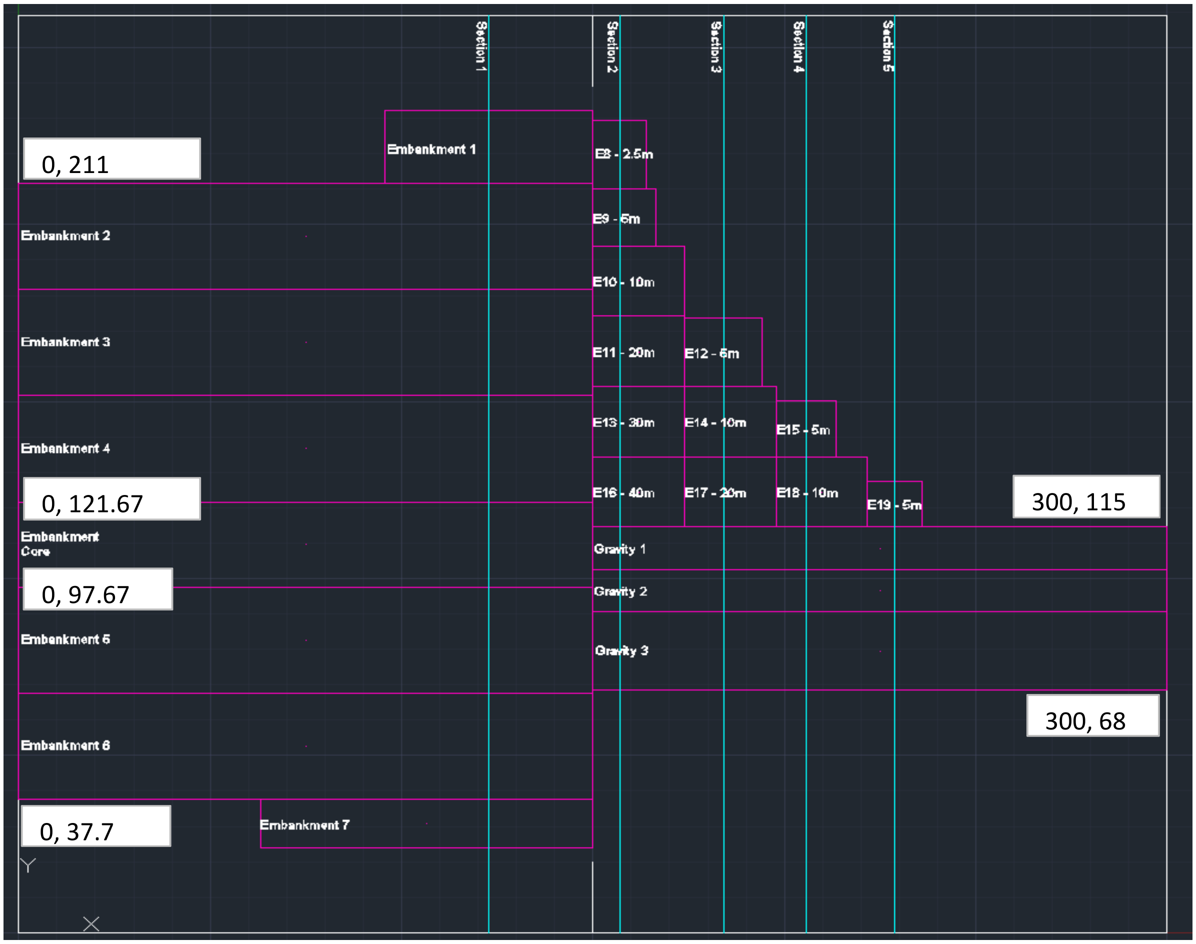
Profile 2 (Key trench)

Material	Level at top (m)	Youngs mod (kN/m2)	Poisson's Ratio	Basis
Trench Fill	0	10000	0.2	No data - assumed
Mudstone	-12.88	300000	0.2	Correlated from UCS & RQD

Rigid boundary set at -30m
Unit weight of Embankment Fill = 18kN/m3
Unit weight of Concrete = 24kN/m3

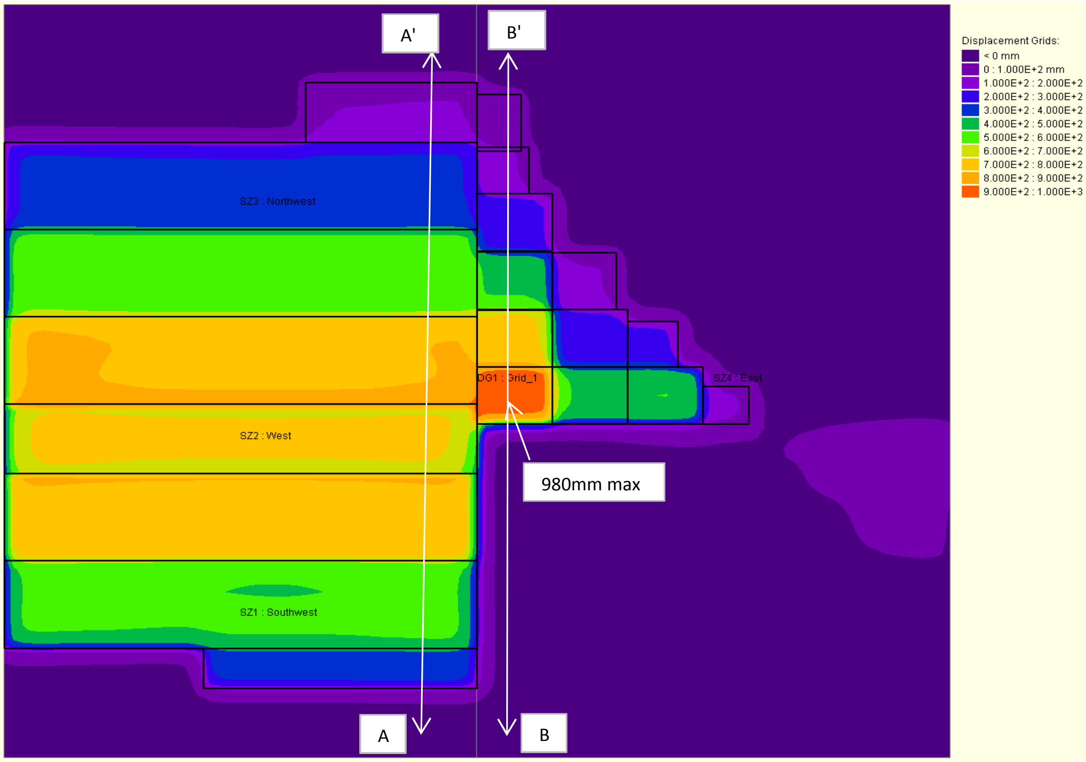
ARUP	Job 225739	S	R
	Member/Location		
Job Title: Foxwood Dam Feasibility Design	Drg.		
Calculation: Model Build Up	Mad RG	D 10/12/2014	C

Load Plan Coordinates

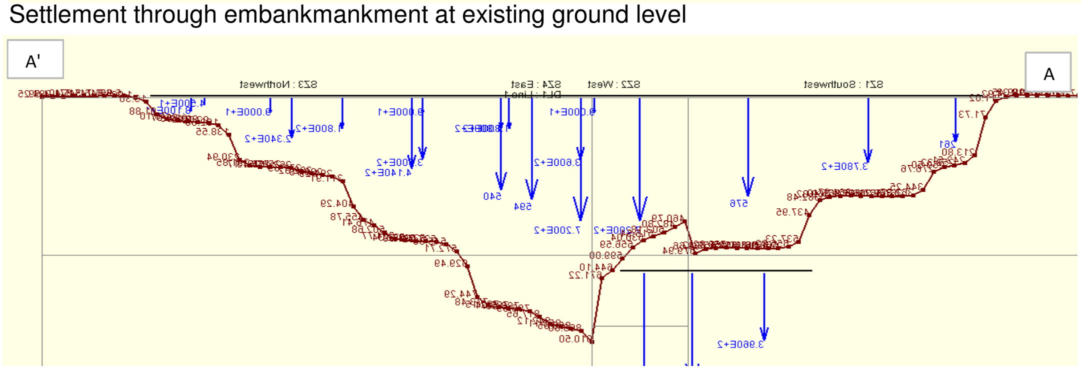


Load Name	Level (m)	Run 1&2 Pressure kN/m2	Run 3 Pressure kN/m2	Run 3 height (m)
Embankment_Co	0	720	559.8	31.1
Embankment_1	0	81	0	0
Embankment_2	0	234	81	4.5
Embankment_3	0	414	234	13
Embankment_4	0	594	414	23
Embankment_5	0	576	414	23
Embankment_6	0	378	234	13
Embankment_7	0	261	81	4.5
Gravity_1	-9.88	1164	1164	49
Gravity_2	-9.88	660	660	28
Gravity_3	-9.88	396	396	17
Embankment_8	0	45	0	0
Embankment_9	0	90	0	0
Embankment_10	0	180	90	5
Embankment_11	0	360	90	5
Embankment_12	0	90	0	0
Embankment_13	0	540	270	15
Embankment_14	0	180	0	0
Embankment_15	0	180	0	0
Embankment_16	0	720	450	25
Embankment_17	0	360	270	15
Embankment_18	0	360	90	5
Embankment_19	0	90	0	0

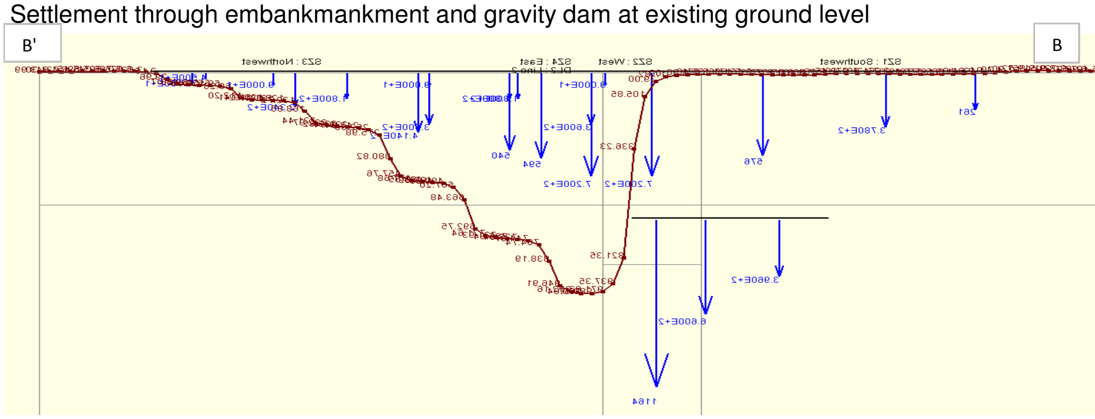
Run 1



Settlement through embankmankment at existing ground level

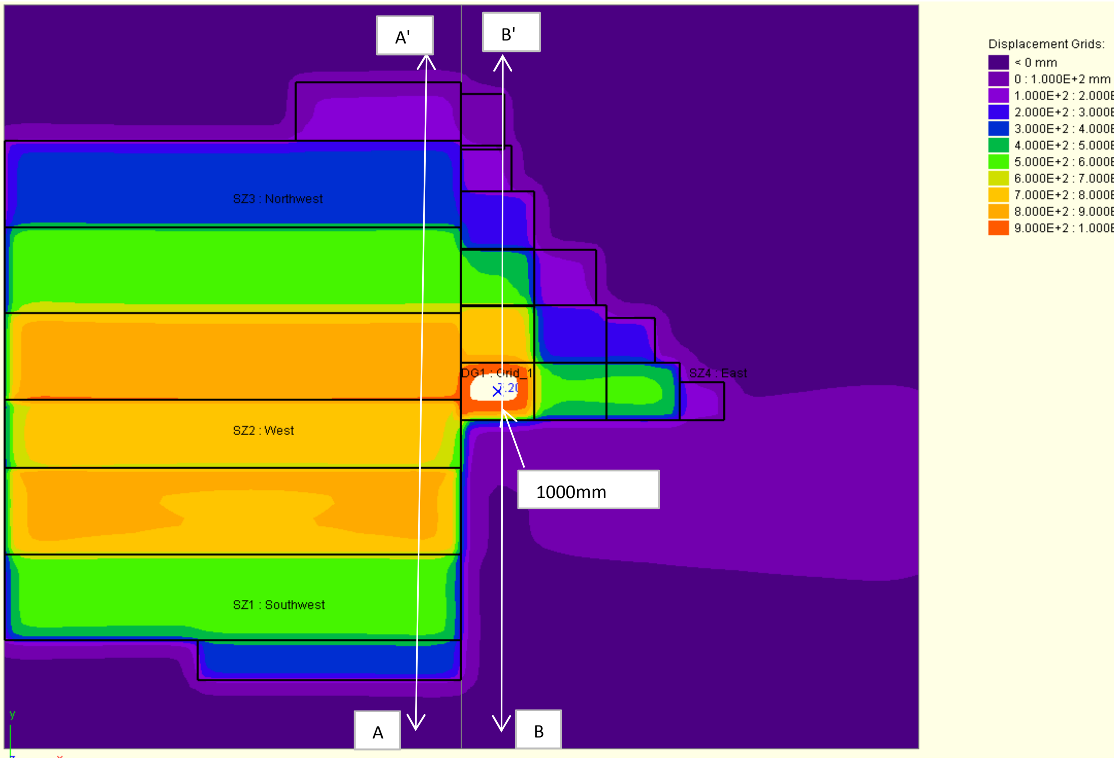


Settlement through embankmankment and gravity dam at existing ground level

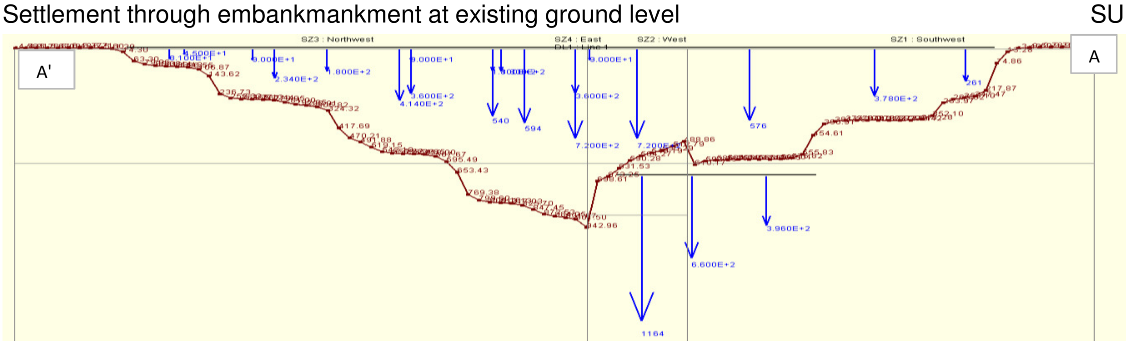


ARUP	Job: 225739	S	R
	Member/Location		
Job Title: Foxwood Dam Feasibility Design	Drg.		
Calculation: Run Images	Mad: RG	D: 10/12/2014	C

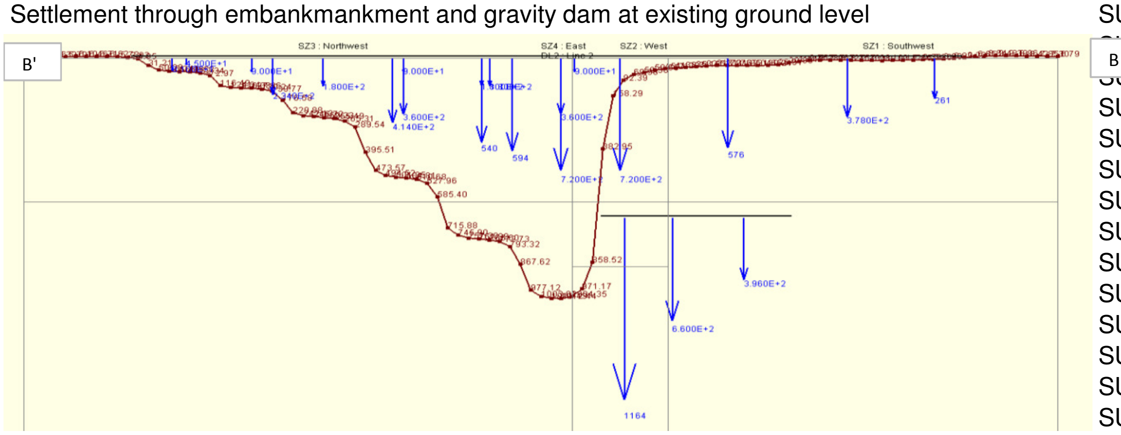
Run 2



Settlement through embankmankment at existing ground level

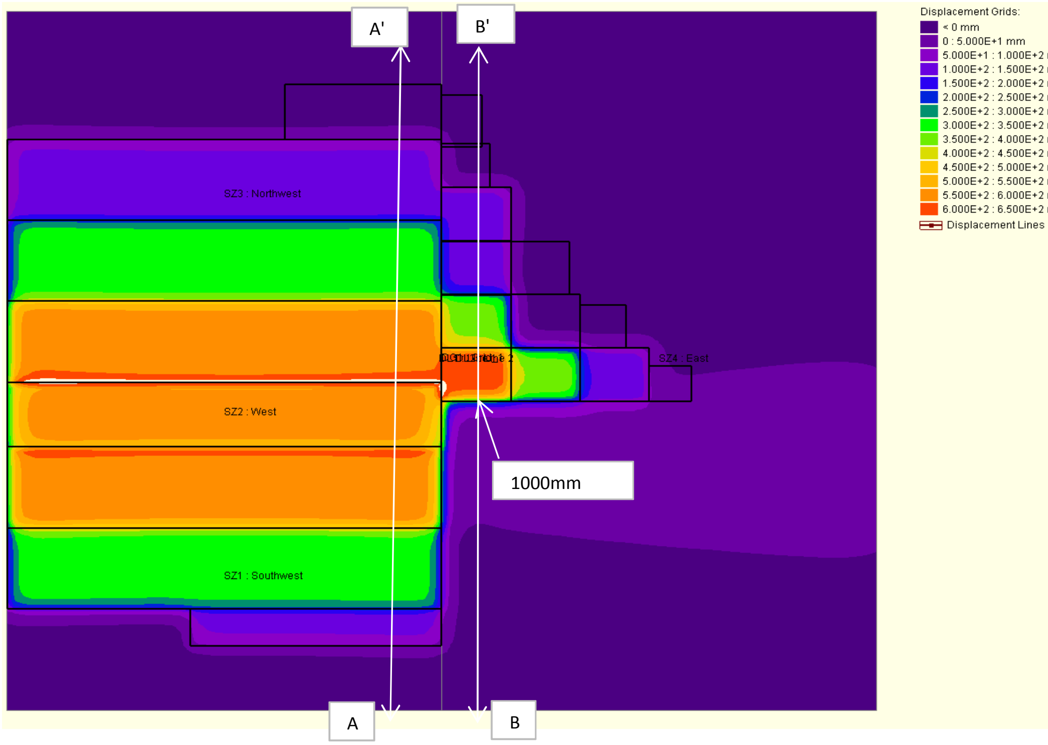


Settlement through embankmankment and gravity dam at existing ground level

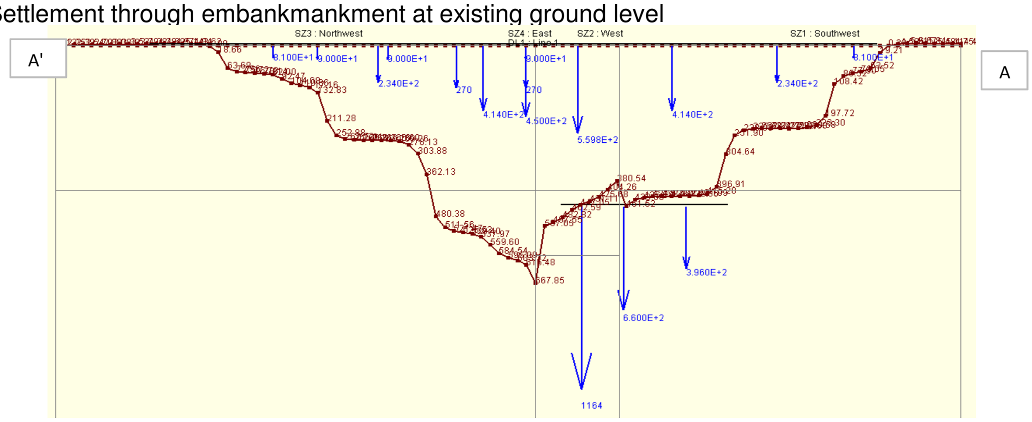


ARUP	Job	S	R
	225739		
	Member/Location		
Job Title: Foxwood Dam Feasibility Design	Drg.		
Calculation: Run Images	Mad RG	D 10/12/2014	C

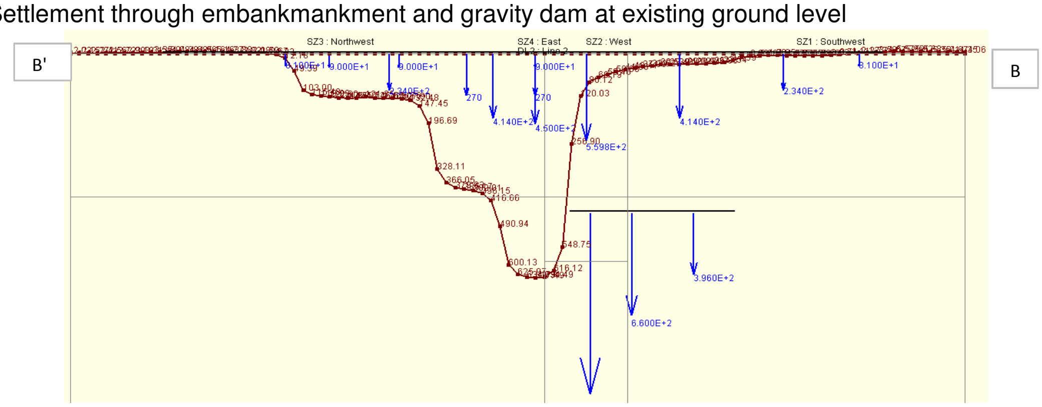
Run 3



Settlement through embankmankment at existing ground level



Settlement through embankmankment and gravity dam at existing ground level



APPENDIX E: SEEP DESIGN RUN REGISTER

ARUP

JOB TITLE	Foxwood Dam Feasibility design
JOB NUMBER	225739
MADE BY	RG
CHECKED BY	
DATE	12/09/2014
Description of spreadsheet	Seep Analysis Run Register
Sheet Number prefix	
Member/Location	
Drawing Reference	
Filename	\\global.arup.com\africa\South Africa\Johannesburg\Jobs\IN_Projects\Projects\225739-00

CONTENTS OF SPREADSHEET

Sheet	Description
Cover	
Notes	
Seep Register	Seep Run Register
Assumptions	Design Assumptions
Run Images	Run Images
Ground Coordinate	Ground Coordinates

AUTHORISATION OF LATEST VERSION

Type and method of check		
Signatures & dates:	Made by	
	Checked	

REVISIONS Current Revision

Rev.	Date	Made by	Checked	Description

ARUP	Jo	S	Rev.
	225739		
	Member/Location		
Job Title: Foxwood Dam Feasibility design	Dr		
Calculation: Seep Run Register	Ma RG	D 12/09/2014	C

Bold text are files which include the final design

Run #	Location	Description	Analysis	Head	Comments
Seep Run 1	Embankment	Rockfill + core	Steady State	615m (TWL)	
Seep Run 2	Embankment	Earthfill Embankment	Steady State	615m (TWL)	
Seep Run 3	Embankment	Earthfill Embankment vert. drain	Steady State	615m (TWL)	
Seep Run 4	**Not used**				
Seep Run 5	Embankment	As Run 3 to see MWL profile for slope	Steady State	620.4 (MWL)	
	EMBANKMENT PROFILE CHANGED FOLLOWING SLOPE ANANLYSIS. 1:4 u/s, 1:3 d/s. inlcined drain				
Seep Run 6	**Embankment**	**Earthfill Embankment**	Steady State	615m (TWL)	
Seep Run 7	Embankment	As Run 6 to see MWL profile for slope	Steady State	620.4 (MWL)	
Seep Run 8	**Embankment**	**As Run 6 with 10m grout cut off**	Steady State	615m (TWL)	
Seep Run 9	**Embankment**	**As Run 9 with 27m grout to lower k mudst**	Steady State	615m (TWL)	
Seep Run 10	Gravity Dam	15m grout cut off	Steady State	615m (TWL)	outflow 3x10-7m^3/sec per m^2 of rock
					high exit gradients through alluvium
Seep Run 11	**Gravity Dam**	**20m grout cut off to lower k mudstone**	Steady State	615m (TWL)	outflow 9x10-8m^3/sec per m^2 of rock
					acceptable exit gradients through alluvium

Section Location	**Basis**
Embankment	Highest embankment section
Gravity Dam	Highest Gravity section

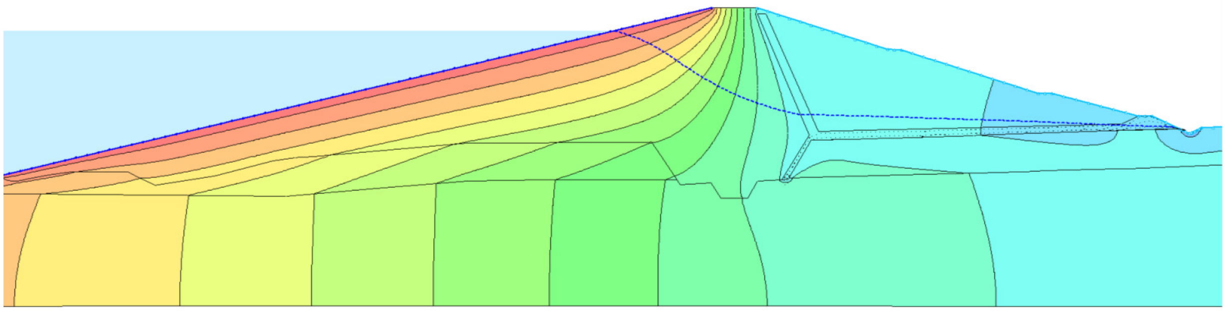
Ground Profile

See Geotechnical Parameters Calcs

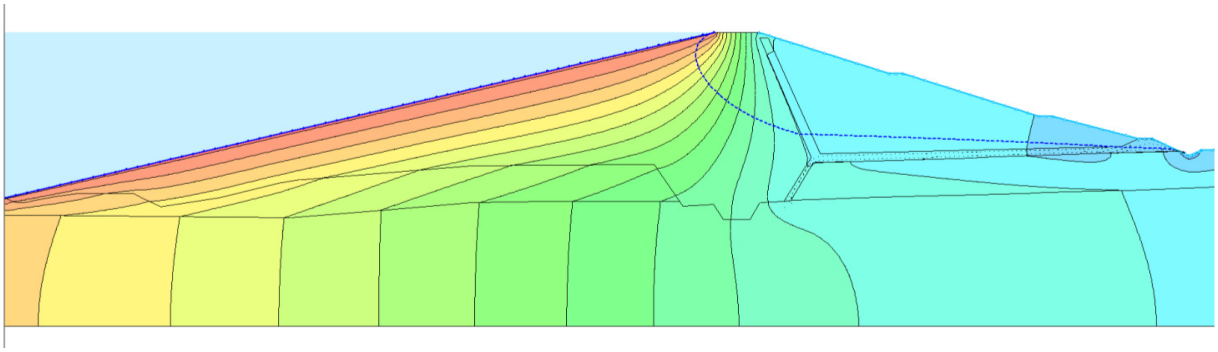
Parameters used in seep analysis

Colour	Material	Saturadted Conductivity	Conductivity R	Sat. Vol Water content	Mv
pale Yellow	Alluvium	1×10^{-7}	1	0.2	0
Brown	Mudstone	3×10^{-6}	1	0.2	0
pale green	Embankment Fill	5×10^{-8}	1	0.2	0
orange	Filter	1×10^{-4}	1	0.2	0
Pink	Grouted Mudstone	1×10^{-7}	1	0.2	0

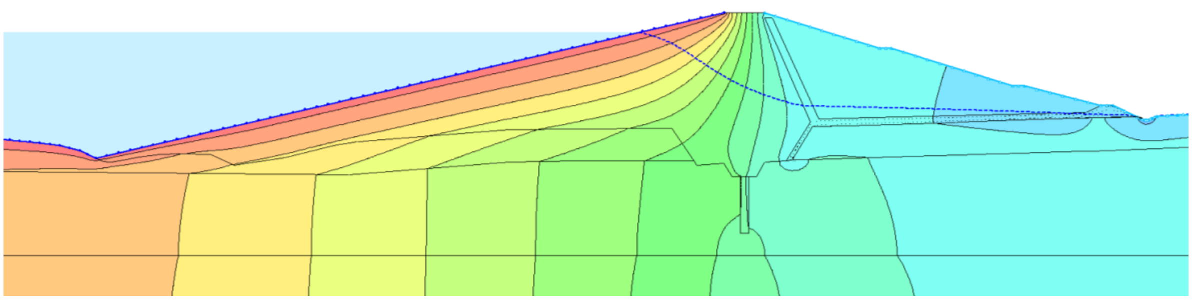
Seep Run 6



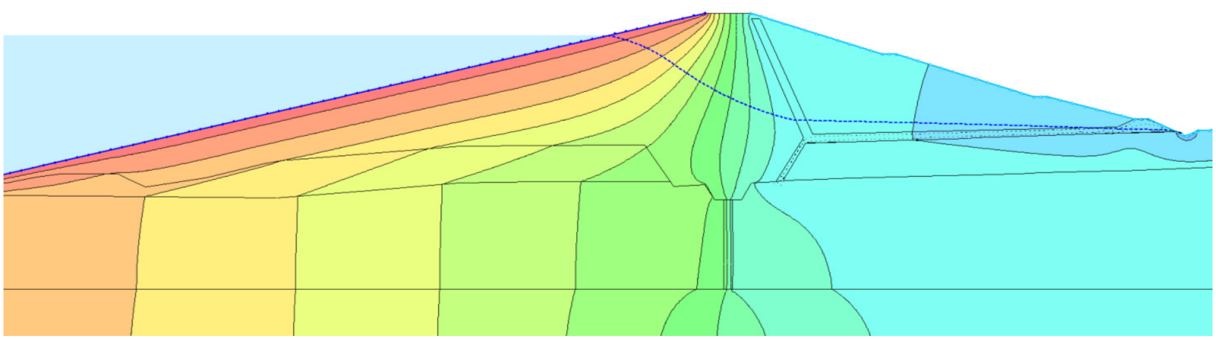
Seep Run 7



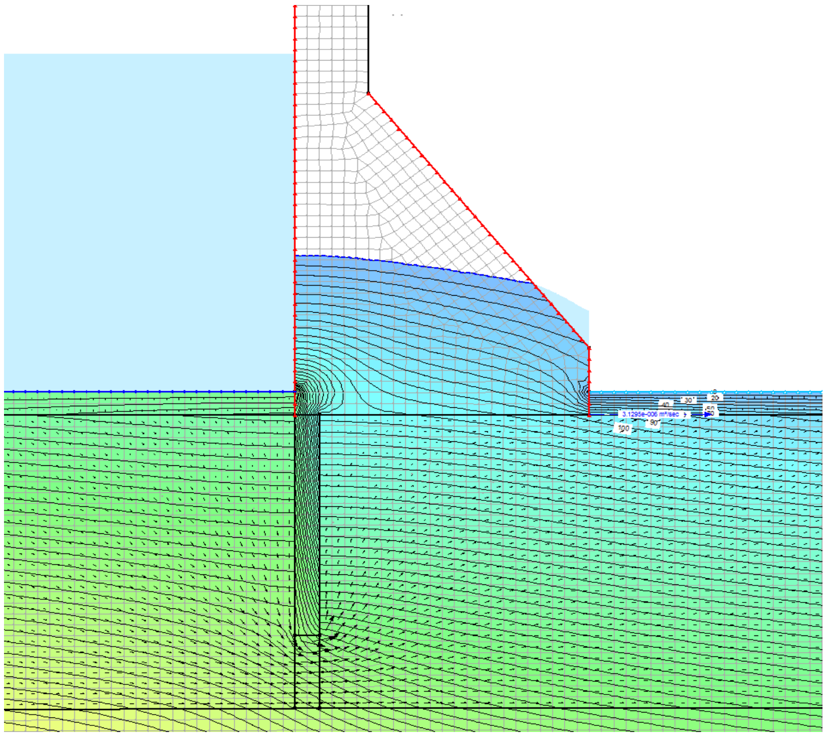
Seep Run 8



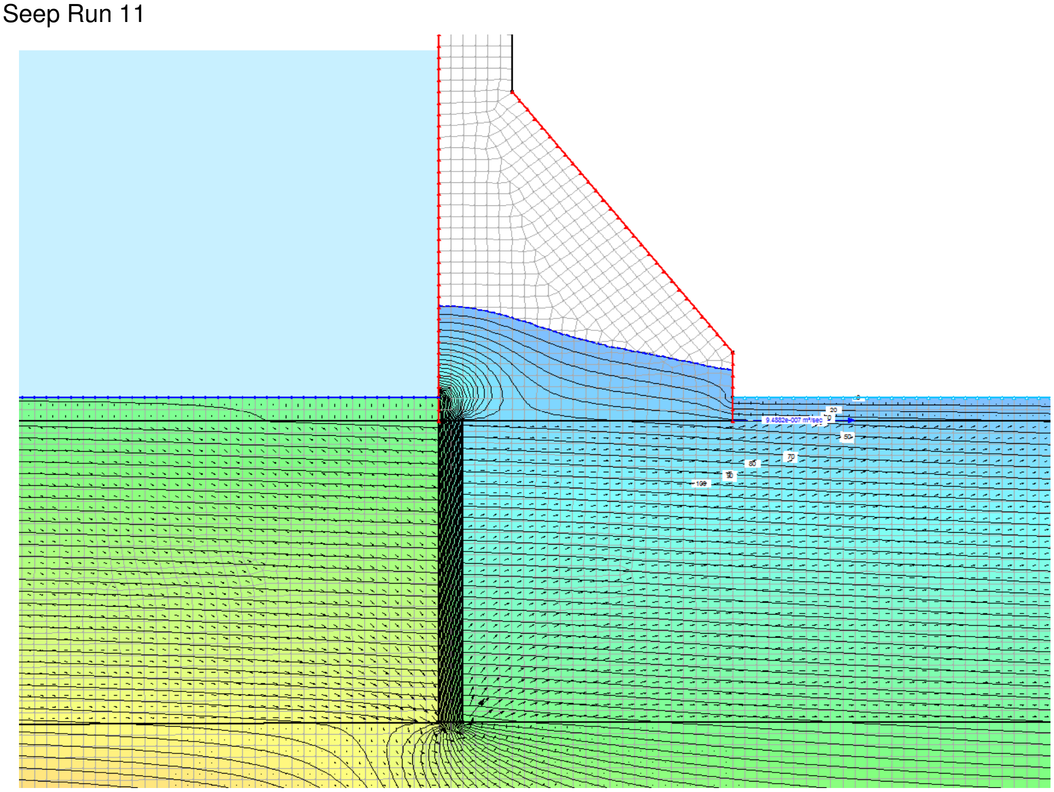
Seep Run 9



Seep Run 10



Seep Run 11



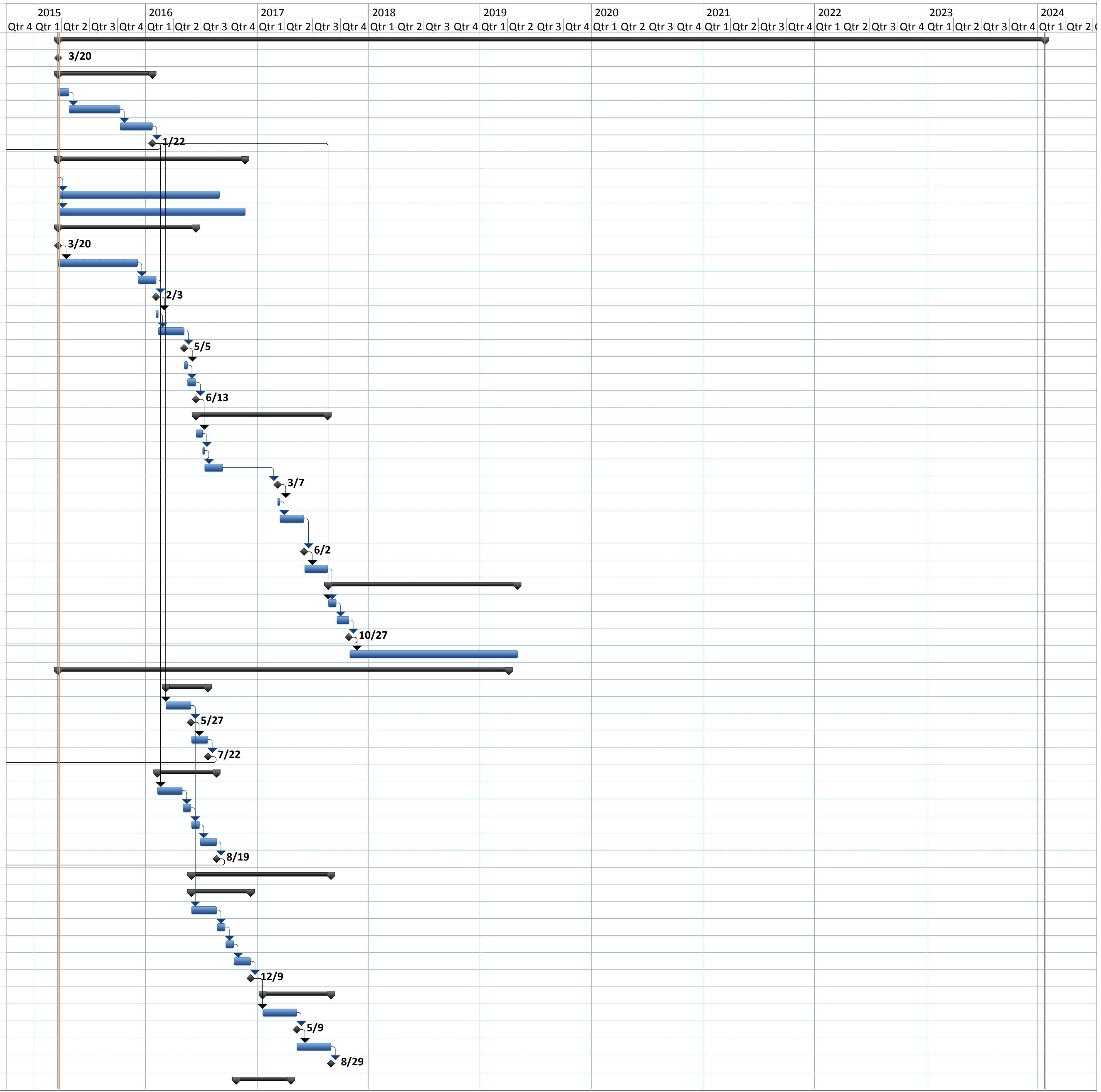
APPENDIX F: DESIGN DRAWINGS

	Site Investigation
225739-GEO-0601	Geotechnical Investigation key Plan
225739-GEO-0602	Seismic Survey Layout
	Dam Works
225739-DAM-0901	Concrete Dam Wall Combined Elevation and Section
225739-DAM-0902	Site Layout (access road, lay-down etc)
225739-DAM-0903	Bulk Pipeline to Adelaide WTW - Plan & Profile
	Earthfill Dam Works
225739-DAM-1001	Dam Plan
225739-DAM-1002	Earthfill Dam Wall Typical Cross-sections and Details
	Foundations
225739-DAM-1101	Cut Off Trench Plan, Longitudinal Section and Details
225739-DAM-1102	Consolidation Grouting
225739-DAM-1103	Grout Curtain and Foundation Treatment - Sheet 1
225739-DAM-1104	Grout Curtain and Foundation Treatment - Sheet 2
225739-DAM-1105	Grout Curtain and Foundation Treatment - Sheet 3
225739-DAM-1106	Grout Curtain and Foundation Treatment - Sheet 4
	Gravity Concrete Dam & Spillway
225739-DAM-1201	Spillway & Concrete Gravity Dam Wall General Layout and Details
225739-DAM-1202	Spillway Excavation
225739-DAM-1203	Rockfill Bolster Return Channel
	Intake / Outlets
225739-DAM-1301	Inlet / Outlet Works General Arrangement & Details

APPENDIX G: CONSTRUCTION PROGRAMME

FOXWOOD DAM IMPLEMENTATION

ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
1		**DAM DEVELOPMENT PROJECT**	**2160 days**	**Mar 20 '15**	**Jan 25 '24**	
2		Project Start	0 days	Mar 20 '15	Mar 20 '15	
3		**Record of Implementation Decision (RID)**	**205 days**	**Mar 20 '15**	**Jan 22 '16**	
4		Draft RID	25 days	Mar 20 '15	Apr 23 '15	
5		DWA Review / Comment Period on RID	6 mons	Apr 24 '15	Oct 8 '15	4
6		Finalise RID	60 days	Oct 9 '15	Jan 22 '16	5
7		Issue RID	0 days	Jan 22 '16	Jan 22 '16	6
8		**Environmental Impact Assessment**	**421 days**	**Mar 20 '15**	**Nov 21 '16**	
9	✓	Environmental Scoping	1 day	Mar 20 '15	Mar 20 '15	
10		Further Public Participation	18 mons	Mar 23 '15	Aug 29 '16	9
11		EIA	21 mons	Mar 23 '15	Nov 21 '16	9
12		**Environmental Authorisation**	**306 days**	**Mar 20 '15**	**Jun 13 '16**	
13		Submit Application to DEA - Single Authorisation	0 days	Mar 20 '15	Mar 20 '15	
14		DEA Evaluation	186 days	Mar 20 '15	Dec 4 '15	13
15		DWA Respond to DEA Enquiry	27 days	Dec 7 '15	Feb 3 '16	14
16		Submit Application to DEA - Single Authorisation	0 days	Feb 3 '16	Feb 3 '16	15
17		DEA Acknowledgement	5 days	Feb 4 '16	Feb 10 '16	16
18		DEA Evaluation / Appeal Period	61 days	Feb 11 '16	May 5 '16	17
19		DEA Approve Authorisation	0 days	May 5 '16	May 5 '16	18
20		DWA Request Clarification on Conditions	7 days	May 6 '16	May 16 '16	19
21		DEA Evaluation	20 days	May 17 '16	Jun 13 '16	20
22		DEA Approve Ammendments to Enviro Authorisation	0 days	Jun 13 '16	Jun 13 '16	21
23		**DWA Approval Process**	**292 days**	**Jun 14 '16**	**Aug 18 '17**	
24		Obtain Stakeholder Support (WUA, SANParks, DM)	15 days	Jun 14 '16	Jul 4 '16	22
25		Prepare Ministerial Submission Secttion 109 Notice	5 days	Jul 5 '16	Jul 11 '16	24
26		Internal Approval Process for Ministerial Submission	44 days	Jul 12 '16	Sep 9 '16	25
27		Ministerial Approval	0 days	Mar 7 '17	Mar 7 '17	26FS+110 days
28		Prepare NWA S110 Notice	5 days	Mar 8 '17	Mar 14 '17	27
29		Internal Approval Process to Publish NWA Section110 Notice	58 days	Mar 15 '17	Jun 2 '17	28
30		Publish NWA Section 110 Notice	0 days	Jun 2 '17	Jun 2 '17	29
31		Public Comment Period	55 days	Jun 5 '17	Aug 18 '17	30
32		**Ministerial Approval to Appoint PSPs**	**410 days**	**Aug 21 '17**	**May 2 '19**	
33		Preparation of Submission	20 days	Aug 21 '17	Sep 15 '17	7,31
34		Pre-Approval & Submit to Minister	30 days	Sep 18 '17	Oct 27 '17	33
35		Ministerial Approval	0 days	Oct 27 '17	Oct 27 '17	34
36		Water Licence Application	18 mons	Oct 30 '17	May 2 '19	35
37		**Environmental Authorisation Conditions**	**988 days**	**Mar 20 '15**	**Apr 4 '19**	
38		**Pre-Construction EMP**	**100 days**	**Mar 7 '16**	**Jul 22 '16**	
39		Preparation of EMP	60 days	Mar 7 '16	May 27 '16	7FS+30 days
40		Submit to DEA	0 days	May 27 '16	May 27 '16	39
41		DEA Evaluation	40 days	May 30 '16	Jul 22 '16	40
42		Approval by DEA	0 days	Jul 22 '16	Jul 22 '16	41
43		**Appoint ECO**	**140 days**	**Feb 8 '16**	**Aug 19 '16**	
44		Prepare Tender for ECO	60 days	Feb 8 '16	Apr 29 '16	7FS+10 days
45		Advertised Tender	20 days	May 2 '16	May 27 '16	44
46		Approve Evaluation Panel	20 days	May 30 '16	Jun 24 '16	45
47		Evaluate Tender & DBAC Approval	40 days	Jun 27 '16	Aug 19 '16	46
48		Appoint ECO	0 days	Aug 19 '16	Aug 19 '16	47
49		**Construction EMP (Phase 2 & 3)**	**310 days**	**May 30 '16**	**Aug 29 '17**	
50		**Appoint PSP**	**140 days**	**May 30 '16**	**Dec 9 '16**	
51		Prepare Tender for PSP	60 days	May 30 '16	Aug 19 '16	39
52		Advertised Tender	20 days	Aug 22 '16	Sep 16 '16	51
53		Approve Evaluation Panel	20 days	Sep 19 '16	Oct 14 '16	52
54		Evaluate Tender & DBAC Approval	40 days	Oct 17 '16	Dec 9 '16	53
55		Appoint PSP	0 days	Dec 9 '16	Dec 9 '16	54
56		**Preparation of EMP**	**160 days**	**Jan 18 '17**	**Aug 29 '17**	
57		Prepare EMP Documentation	80 days	Jan 18 '17	May 9 '17	55FS+10 days
58		Submit to DEA	0 days	May 9 '17	May 9 '17	57
59		DEA Evaluation	80 days	May 10 '17	Aug 29 '17	58
60		Approval by DEA	0 days	Aug 29 '17	Aug 29 '17	59
61		**Plant Rescue**	**112 days**	**Oct 24 '16**	**Apr 20 '17**	



Project: Foxwood Dam
Date: Mar 25 '15

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

FOXWOOD DAM IMPLEMENTATION

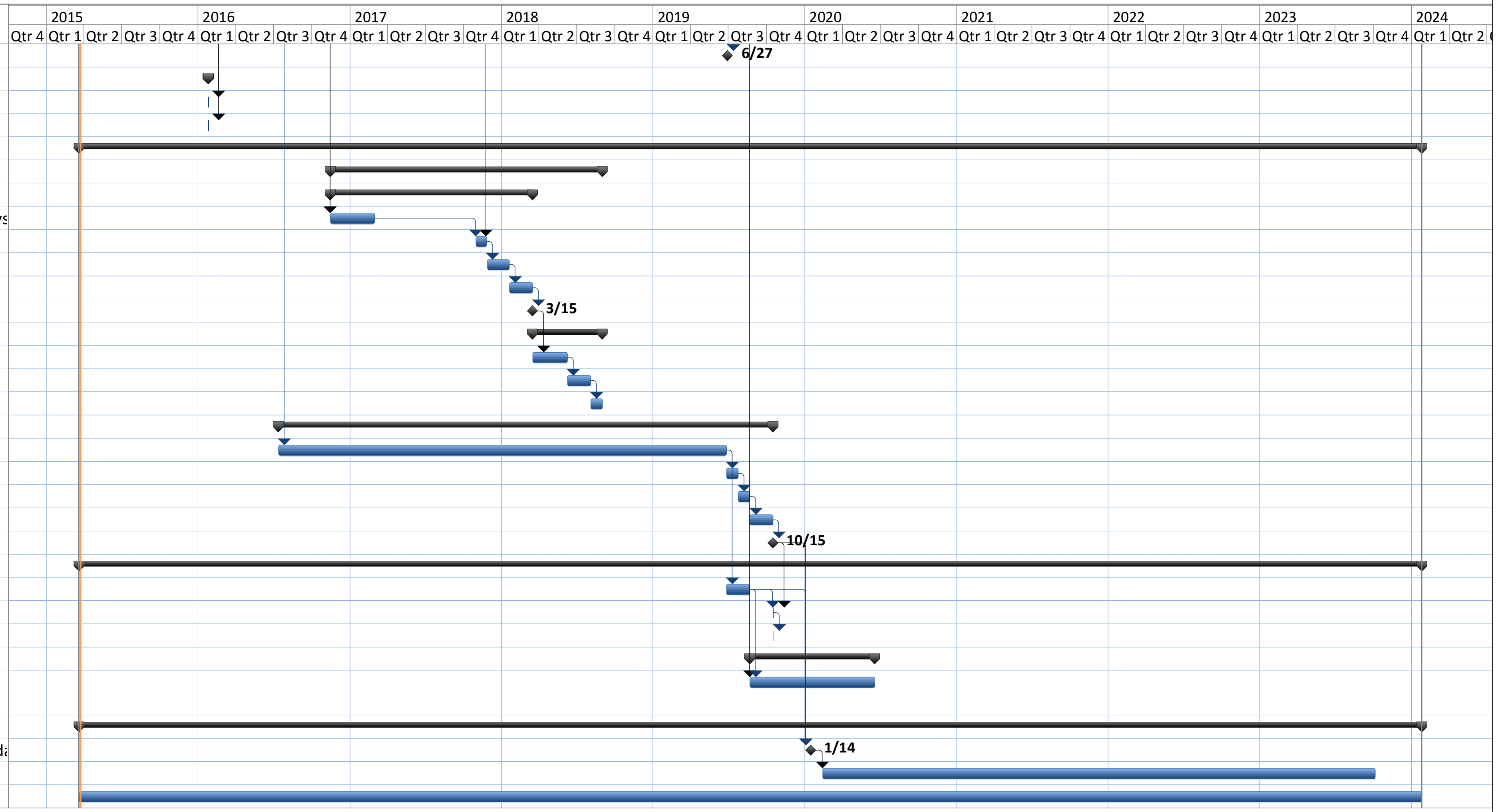
ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
62		Search RDL Floral Species	80 days	Oct 24 '16	Mar 7 '17	48FS+45 days
63		Obtain Permit to Remove Species	0 days	Mar 7 '17	Mar 7 '17	62
64		Plant Rescue in Priority Area	30 days	Mar 8 '17	Apr 18 '17	63
65		Plant Rescue in Dam Basin	1 day	Apr 19 '17	Apr 19 '17	64
66		Prepare Habitat Rehabilitation Plan	1 day	Apr 20 '17	Apr 20 '17	65
67		**Archeological Work**	**240 days**	**Apr 13 '18**	**Apr 4 '19**	
68		**Appoint PSP**	**140 days**	**Apr 13 '18**	**Oct 25 '18**	
69		Prepare Tender for Archeological Work	60 days	Apr 13 '18	Jul 5 '18	35FS+100 days,7
70		Advertised Tender	20 days	Jul 6 '18	Aug 2 '18	69
71		Approve Evaluation Panel	20 days	Aug 3 '18	Aug 30 '18	70
72		Evaluate Tender & DBAC Approval	40 days	Aug 31 '18	Oct 25 '18	71
73		Appoint PSP	0 days	Oct 25 '18	Oct 25 '18	72
74		**Implementation**	**100 days**	**Oct 26 '18**	**Apr 4 '19**	
75		Archeological Work Priority Area	20 days	Oct 26 '18	Nov 22 '18	73
76		Archeological Work - Dam Basin	80 days	Nov 23 '18	Apr 4 '19	75
77		**Grave Relocation**	**988 days**	**Mar 20 '15**	**Apr 4 '19**	
78		**Appoint PSP**	**140 days**	**Apr 13 '18**	**Oct 25 '18**	
79		Prepare Tender for Archeological Work	60 days	Apr 13 '18	Jul 5 '18	7,35FS+100 days
80		Advertised Tender	20 days	Jul 6 '18	Aug 2 '18	79
81		Approve Evaluation Panel	20 days	Aug 3 '18	Aug 30 '18	80
82		Evaluate Tender & DBAC Approval	40 days	Aug 31 '18	Oct 25 '18	81
83		Appoint PSP	0 days	Oct 25 '18	Oct 25 '18	82,106
84		**Implementation**	**100 days**	**Oct 26 '18**	**Apr 4 '19**	
85		Obtain Permit from SAHRA	20 days	Oct 26 '18	Nov 22 '18	83,7,106
86		Grave Relocation in Priority Area	20 days	Nov 23 '18	Jan 10 '19	85,106
87		Grave Relocation in Dam Basin / Conservation	60 days	Jan 11 '19	Apr 4 '19	86,108
88		**Operational of EMP**	**1 day**	**Mar 20 '15**	**Mar 20 '15**	
89		Appoint PSP & Prepare EMP	1 day	Mar 20 '15	Mar 20 '15	
90		Submit to DEA	1 day	Mar 20 '15	Mar 20 '15	
91		DEA Evaluation	1 day	Mar 20 '15	Mar 20 '15	
92		Approval by DEA	1 day	Mar 20 '15	Mar 20 '15	
93		**Land Matters**	**637 days**	**Mar 20 '15**	**Oct 12 '17**	
94		**Dam Footprint and Borrow Pits**	**637 days**	**Mar 20 '15**	**Oct 12 '17**	
95		Purchase Line Demarcation	20 days	Mar 20 '15	Apr 16 '15	
96		Prepare Land Schedules	20 days	Apr 17 '15	May 14 '15	95
97		**Appoint Evaluator**	**210 days**	**Apr 18 '16**	**Feb 28 '17**	
98		Prepare Tender Documentation	130 days	Apr 18 '16	Oct 14 '16	7FS+60 days
99		Advertised Tender	20 days	Oct 17 '16	Nov 11 '16	98
100		Approve Evaluation Panel	20 days	Nov 14 '16	Dec 9 '16	99
101		Evaluate Tender & DBAC Approval	40 days	Jan 4 '17	Feb 28 '17	100
102		Appoint PSP	0 days	Feb 28 '17	Feb 28 '17	101
103		**Valuation Investigation**	**140 days**	**Mar 29 '17**	**Oct 10 '17**	
104		Prepare Valuation Report	60 days	Mar 29 '17	Jun 20 '17	102FS+20 days
105		Evaluation of Report	60 days	Jun 21 '17	Sep 12 '17	104
106		Expropriate Dam Footprint & Borrow Pits	0 days	Oct 10 '17	Oct 10 '17	105FS+20 days
107		**Dam Basin Properties**	**2 days**	**Oct 11 '17**	**Oct 12 '17**	
108		Acquire Dam Basin Properties	1 day	Oct 11 '17	Oct 11 '17	106
109		Caretaker Agreements	1 day	Oct 12 '17	Oct 12 '17	108
110		**Infrastructure**	**843 days**	**Jan 25 '16**	**Jun 27 '19**	
111		**Appoint PSP**	**583 days**	**Jan 25 '16**	**Jun 7 '18**	
112		Prepare Tender Documentation	60 days	Jan 25 '16	Apr 15 '16	7
113		Advertised Tender	20 days	Feb 16 '18	Mar 15 '18	35FS+60 days,11
114		Approve Evaluation Panel	20 days	Mar 16 '18	Apr 12 '18	113
115		Evaluate Tender & DBAC Approval	40 days	Apr 13 '18	Jun 7 '18	114
116		Appoint PSP	0 days	Jun 7 '18	Jun 7 '18	115
117		**Tender Design & Appointment of Contractor**	**240 days**	**Jul 6 '18**	**Jun 27 '19**	
118		**Prepare Tender Documentation**	**160 days**	**Jul 6 '18**	**Mar 7 '19**	**116FS+20 days**
119		Realignment of Road R344	160 days	Jul 6 '18	Mar 7 '19	
120		Reinstate Bulk Water Supply to Adelaide Dam	160 days	Jul 6 '18	Mar 7 '19	
121		Advertised Tender	20 days	Mar 8 '19	Apr 4 '19	118
122		Approve Evaluation Panel	20 days	Apr 5 '19	May 2 '19	121
123		Evaluate Tender & DBAC Approval	40 days	May 3 '19	Jun 27 '19	122

Project: Foxwood Dam
Date: Mar 25 '15

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

FOXWOOD DAM IMPLEMENTATION

ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
124		Appoint PSP	0 days	Jun 27 '19	Jun 27 '19	123
125		**Public Utilities**	**1 day**	**Jan 25 '16**	**Jan 25 '16**	
126		Eskom Services	1 day	Jan 25 '16	Jan 25 '16	7
127		Telkom Services	1 day	Jan 25 '16	Jan 25 '16	7
128		**Dam Design & Construction**	**2160 days**	**Mar 20 '15**	**Jan 25 '24**	
129		**Foundation & Borrow Pits Investigation**	**433 days**	**Nov 14 '16**	**Aug 30 '18**	
130		**Appoint PSP**	**313 days**	**Nov 14 '16**	**Mar 15 '18**	
131		Prepare Tender Documentation	60 days	Nov 14 '16	Feb 28 '17	42,48FS+60 days
132		Advertised Tender	20 days	Oct 30 '17	Nov 24 '17	35,131
133		Approve Evaluation Panel	20 days	Nov 27 '17	Jan 18 '18	132
134		Evaluate Tender & DBAC Approval	40 days	Jan 19 '18	Mar 15 '18	133
135		Appoint PSP	0 days	Mar 15 '18	Mar 15 '18	134
136		**Investigation**	**120 days**	**Mar 16 '18**	**Aug 30 '18**	
137		Foundation & Borrow Pits Investigation	60 days	Mar 16 '18	Jun 7 '18	135
138		Prepare Report	40 days	Jun 8 '18	Aug 2 '18	137
139		Evaluation of Report	20 days	Aug 3 '18	Aug 30 '18	138
140		**Tender Design & Appointment of Contractor**	**800 days**	**Jul 12 '16**	**Oct 15 '19**	
141		Prepare Tender Documentation	720 days	Jul 12 '16	Jun 25 '19	25
142		Advertised Tender	20 days	Jun 26 '19	Jul 23 '19	141
143		Approve Evaluation Panel	20 days	Jul 24 '19	Aug 20 '19	142
144		Evaluate Tender & DBAC Approval	40 days	Aug 21 '19	Oct 15 '19	143
145		Appoint PSP	0 days	Oct 15 '19	Oct 15 '19	144
146		**Dam Safety Issues**	**2160 days**	**Mar 20 '15**	**Jan 25 '24**	
147		Consider & Issue Licence to Construct	40 days	Jun 26 '19	Aug 20 '19	141
148		Consider & Issue Licence to Impound	1 day	Oct 16 '19	Oct 16 '19	147,145
149		Completion Report & Certificate	1 day	Oct 17 '19	Oct 17 '19	148
150		**Final Dam Design & Supervision**	**200 days**	**Aug 21 '19**	**Jun 16 '20**	
151		Final Design	200 days	Aug 21 '19	Jun 16 '20	7,147
152		**Dam Construction**	**2160 days**	**Mar 20 '15**	**Jan 25 '24**	
153		Site Establishment	0 days	Jan 14 '20	Jan 14 '20	147,145FS+50 da
154		Construction	45 mons	Feb 12 '20	Oct 4 '23	153FS+20 days
155		Irrigation Scheme Development	108 mons	Mar 20 '15	Jan 25 '24	



Project: Foxwood Dam
Date: Mar 25 '15

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

APPENDIX H: BILL OF QUANTITIES

Item No	Description	Amount (ZAR)	Comment
1	PRELIMINARY & GENERAL	*239 411 545*	*30% of item 2-12*
2	WATER CONTROL-RIVER DIVERSION	*5 118 848*	
3	DRILLING & GROUTING	*65 895 189*	
	Earthfill	*5 772 591*	
	Concrete Gravity	*60 122 598*	
4	GRAVITY SPILLWAY	*434 835 032*	
5	GRAVITY NOC	*26 515 352*	
6	EARTHFILL EMBANKMENT	*105 196 437*	
9	OUTLET WORKS	*64 306 681*	
	Concrete Works	21 204 550	
	Mechanical Equipment	39 102 131	
	Structural Steelwork	1 750 000	
	Electrical Equipment	2 250 000	
10	INSTRUMENTATION	*7 500 000*	Provisional Sum
11	Miscellaneous 10% & Landscaping 2.5%	*88 670 942*	(12.5% of cost (excl P&G))
	DAM CONSTRUCTION (incl. P&G)	***1 167 651 897***	

12	TOTAL DAM CONSTRUCTION (incl. associated infrastructure)		
	1 MAR composite dam structure	*1 167 651 897*	
	Dam Access Road	*9 412 689*	(incl 10% contingency)
	Bulk water pipeline and pumpstation	*8 887 960*	(incl 10% contingency)
	Gauging Weir & other DWS hydrology structures	*5 451 000*	
	Relocation of R344 (MR00639)	*126 599 941*	(incl 10% contingency)
	Relocation of water supply canal	*20 400 000*	
	Land matters - land costs	*10 239 625*	
	Land matters - fixed improvements	*25 764 000*	
	Graves relocation	*0 300 000*	
	Eskom relocation cost	*2 200 000*	
	Telkom relocation cost	*0 500 000*	
	Environmental management	*5 000 000*	
Sub-total A	GRAND TOTAL	***1 382 407 112***	

	Contingencies	*207 361 067*	(15% of Sub total A)
Sub-total B	TOTAL DAM CONSTRUCTION (incl contingency)	***1 589 768 179***	
	Professional Fees	*238 465 227*	(15% of Sub-total B)
Sub-total C	TOTAL COST (incl design fees)	***1 828 233 406***	
	VAT	*255 952 677*	(14% of Sub-total C)
	TOTAL DAM COSTS	***2 084 186 082***	

Potential Cost	**Relocation of MR00368**	***311 595 811***	**(incl 10% contingency & VAT)**

FOXWOOD DAM

SECTION 1: PRELIMINARY AND GENERAL

	Dam Type and Size			**Composite 1MAR**	
item No	Description	Unit	Rate	Quantity	Amount R
1,1	**PRELIMINARY & GENERAL** 30% of Total Construction Cost	Lump Sum	30%		239 411 545
	TOTAL TO SUMMARY				239 411 545

FOXWOOD DAM					
SECTION 2: RIVER DIVERSION					
	Dam Type and Size			**Composite 1MAR**	
Item No	Description	Unit	Rate	Quantity	Amount R
2,1	**RIVER DIVERSION**				
2.1.1	Temporary coffer dam	m^3	102	25 000	2 550 000
	Install temporary diversion box culvert and closure	Sum	1 500 000	1	1 500 000
	Excavation hard	m^3	263	4 064	1 068 848
	TOTAL TO SUMMARY				5 118 848

FOXWOOD DAM
SECTION 3: DRILLING AND GROUTING (EARTHFILL SECTION)

	Dam Type and Size			**Composite 1MAR**	
Item No	Description	Unit	Rate	Quantity	Amount R
3,1	GENERAL				
3.1.1	Initial set-up for drilling and grouting equipment	set	500 000	1	500 000
3.1.2	Maintain drilling and grouting equipment	months	50 000	24	1 200 000
3,2	DRILLING AND GROUTING				
3.2.1	Curtain grouting (dam wall and ogee spillway)				
	(a) Set up including collar:				
	(i) Primary holes spaced at 6m c/c	No	670	34	23 003
	(ii) Secondary holes spaced at 3m c/c	No	670	33	22 333
	(iii) check holes (5%)	No	3 740	3	12 654
	(b) Drill and grout in down hole stages of 6m:				
	(i) 48mm dia. holes at depths of 0m-20m	m	270	1 353	365 400
	(ii) 48mm dia. Holes at depths of 20m-40m	m	270	1 020	275 400
	(iii) 48mm dia.holes at depths of 40m-60m	m	510	69	35 020
3.2.2	Consolidation grouting				
	(a) Set up including collar for 38mm dia. holes spaced at 3m c/c				
	(i) below cut off trench	No	1 490	133	198 667
	(b) Drill and grout in down hole stages of 3m:	m	375	800	300 000
3.2.3	Dry grout materials				
	(a) Rapid Hardening Portland Cement (RHPC) 70%	t	4 760	45	216 326
	(b) Slagment/ Flyash (30%)	t	1 400	19	27 268
3.2.4	Sleeves				
	(a) 75mm dia. PVC sleeve pipes	m	656	21	13 739
3,3	WATER PRESSURE TESTING				
3.3.1	Water pressure tests (in stages)				
	(a) Curtain grout holes	No	2 625	407	1 068 375
	(b) Check holes	No	2 625	17	44 406
	(c) Consolidation grout holes	No	2 625	133	350 000
3,4	Concrete grout cap				
	(a) 25/38 concrete	m³	1 750	640	1 120 000
	TOTAL TO SUMMARY				5 772 591

FOXWOOD DAM
SECTION 3: DRILLING AND GROUTING (SPILLWAY SECTION)

	Dam Type and Size			**Composite 1MAR**	
Item No	Description	Unit	Rate	Quantity	Amount R
3	**DRILLING AND GROUTING**				
3,1	GENERAL				
3.1.1	Initial set-up for drilling and grouting equipment	set	500 000	1	500 000
3.1.2	Maintain drilling and grouting equipment	months	50 000	24	1 200 000
3,2	Drilling and Grouting				
3.2.1	Curtain grouting (dam wall and ogee spillway)				
	(a) Set up including collar:				
	(i) Primary holes spaced at 6m c/c	No	670	44	29 703
	(ii) Secondary holes spaced at 3m c/c	No	670	43	29 033
	(iii) check holes (5%)	No	3 740	4	16 394
	(b) Drill and grout in down hole stages of 6m:				
	(i) 48mm dia. holes at depths of 0m-20m	m	270	1 753	473 400
	(ii) 48mm dia. Holes at depths of 20m-40m	m	270	1 320	356 400
	(iii) 48mm dia.holes at depths of 40m-60m	m	510	89	45 220
3.2.3	Consolidation grouting				
	(a) Set up including collar for 38mm dia. holes spaced at 3m c/c				
	(i) below ogee spillway	No	985	986	970 954
	(ii) below Concrete non-overflow sections	No	985	7 097	6 990 943
	(iii) below Rockfill non-overflow sections	No	985		
	(b) Drill and grout in down hole stages of 3m:	m	375	48 499	18 187 075
3.2.4	Dry grout materials				
	(a) Rapid Hardening Portland Cement (RHPC) 70%	t	4 760	724	3 447 128
	(b) Slagment/ Flyash (30%)	t	1 400	310	434 512
3.2.5	Sleeves				
	(a) 75mm dia. PVC sleeve pipes	m	656	334	218 925
3.2.6	Foundation Drainage from Gallery				
	(a) Setup	No	985	83	82 083
	(b) Drill:				
	(i) 102mm dia. holes at depths of 0m-20m	m	1 150	1 667	1 916 667
	(ii) 102mm dia. Holes at depths of 20m-40m	m	1 380	1 667	2 300 000
	(iii) 102mm dia.holes at depths of 40m-60m	m	1 590	167	265 000
3,3					
3.3.1	Water pressure tests (in stages)				
	(a) Curtain grout holes	No	2 625	527	1 383 375
	(b) Check holes	No	2 625	22	57 531
	(c) Consolidation grout holes	No	2 625	8 083	21 218 254
	TOTAL TO SUMMARY				60 122 598

FOXWOOD DAM
SECTION 4: GRAVITY DAM SPILLWAY

	Dam Type and Size			**Composite 1MAR**	
Item No	Description	Unit	Rate	Quantity	Amount R
4,1	**Gravity Spillway**				
4.1.1	Site preparation				
	(a) Clearing and grubbing including disposal of material	ha	35 200	1	31 228
	(b) Stockpiling of topsoil	m³	60	2 661	158 492
4.1.2	Excavations				
	(a) Overbuden (soft and burdens)	m³	51	141 562	7 247 973
	(b) Excavations in rock (controlled blasting)	m³	263	8 621	2 267 389
4.1.3	Foundation preparation				
	(a) Rock foundation for concrete	m²	110	8 872	975 883
4.1.4	Formwork (including joints)				
	(a) Class F2 (smooth) to permanently exposed surfaces				
	(i) Plain	m²	340	20 304	6 903 487
	(ii) Curved	m²	450	1 080	485 965
	(b)Class F1 (rough) to hidden surfaces				
	(i) Plain	m²	280	10 234	2 865 402
4.1.5	Surface Finish				
	(a) Class U3 (smooth) to permanently exposed surfaces	m²	29	9 759	283 006
	(b) Class UI (rough) to hidden surfaces	m²	19	488	9 027
4.1.6	Waterstops				
	(a)PVC waterstops 250mm wide retaining walls)	m			
	(b) Rubber waterstops 355mm wide (Concrete wall)	m	580	19 489	11 303 537
4.1.7	Construction Joints				
	(a) Formed designated joints in Concrete wall	m²	360	10 234	3 684 088
4.1.8	Drainage				
	Supply and install permanent drains complete:				
	(a) 75mm drainage holes (3m c/c @ 5m depth)	m	688	400	275 200
4.1.9	Anchor bars				
	(a) Y32 dia. 5m into rock @ 1.5m c/c	No	425	533	226 667
4.1.10	Reinforcement				
	(a) High tensile reinforcement bars in concrete				
	(i) Spillway crest (inculding bridge deck and piers) (100kg/m3)	t	11 700	4 498	52 622 388
	(ii) Spillway Apron (60kg/m3)	t	11 700	1 465	17 135 820
	(iii) Gallery concrete (gallery: 20kg/m2)	t	11 700	900	10 524 478
4.1.11	Concrete				
	(a)in-situ concrete:				
	(i) Class 20/38 skin concrete	m³			
	(ii) Class 25/38 -structural(bridge/crest)	m³	1 750	44 976	78 708 700
	(iii) Class 15/53 - Mass or roller compacted	m³	1 100	194 446	213 890 435
	(iv) class 20/19 -Dental + flow	m³	1 200	8 872	10 645 992
	(v) Bedding mortar	m³	1 450	444	643 195
	(b) Precast elements:				
	(i) Gallery arches	m³	12 695	156	1 980 658
4.1.12	Graded Bolster Rockfill				
	(a) Rock for rip-rap (Medium)	m²	202	32 540	6 556 860
	(b) Rock for rip-rap (Large)	m²	273	10 847	2 961 163
	TOTAL TO SUMMARY				434 835 032

FOXWOOD DAM
SECTION 5: GRAVITY DAM Non-Overflow Crest

	Dam Type and Size			**Composite 1MAR**	
Item No	Description	Unit	Rate	Quantity	Amount R
5,1	**Gravity Dam NOC**				
5.1.1	Site preparation				
	(a) Clearing and grubbing including disposal of material	ha	35 200	2	54 413
	(b) Stockpiling of topsoil	m³	60	275	16 402
5.1.2	Excavations				
	(a) Overbuden (soft and burdens)	m³	51	19 293	987 793
	(b) Excavations in rock (controlled blasting)	m³	263	981	257 945
5.1.3	Foundation preparation				
	(a) Rock foundation for concrete	m²	110	15 458	1 700 412
5.1.4	Formwork (including joints)				
	(a) Class F2 (smooth) to permanently exposed surfaces				
	(i) Plain	m²	340	2 712	921 994
	(b) Class F1 (rough) to hidden surfaces				
	(i) Plain	m²	280	1 522	426 117
5.1.5	Surface Finish				
	(a) Class U3 (smooth) to permanently exposed surfaces	m²	29	15 458	448 290
5.1.6	Waterstops				
	(a)PVC waterstops 250mm wide retaining walls)	m			
	(b) Rubber waterstops 355mm wide (Concrete wall)	m	580	251	145 756
5.1.7	Construction Joints				
	(a) Formed joints in Concrete wall	m²	360	1 522	547 865
5.1.8	Drainage				
	Supply and install permanent drains complete:				
	(a) 75mm drainage holes (3m c/c @ 5m depth)	m	688		
5.1.10	Concrete				
	(a)in-situ concrete:				
	(i) Class25/19 -structural	m³	1 750	919	1 607 916
	(ii) Class 15/53 - Mass	m³	1 100	15 458	17 004 119
	(iii) class 20/19 -Dental	m³	1 200	918	1 101 756
	(iv) Bedding mortar	m³	1 450	46	66 564
	(b) Precast elements:				
	(i) Gallery arches	m³	12 695	97	1 228 008
	(iii) Training walls				
	TOTAL TO SUMMARY				26 515 352

FOXWOOD DAM
SECTION 6: EARTHFILL SECTION

	Dam Type and Size			**Composite 1MAR**	
Item No	Description	Unit	Rate	Quantity	Amount R
6,1	**EARTHFILL EMBANKMENT**				
6.1.1	Site preparation				
	(a) Clearing and grubbing including disposal of material	ha	35 200	6	224 846
6.1.2	Excavations under dam footprint				
	(a) Soft	m^3	51	44 341	2 270 271
	(b) Excavate cutoff trench (hard)	m^3	263	5 544	1 458 035
	(b) Extra over in soft (hard)	m^3	212	10 493	2 222 502
6.1.3	Earthfill from borrow areas				
	(i) Embankment earthfill	m^3	51	703 607	35 883 964
6.1.4	Foundation treatment				
	(a) Treatment of joints and irregularities using:				
	(i) Slush grout (below cut off trench)	m^2	945	1 200	1 134 000
6.1.5	Fill and backfill				
	Transport from borrow area, place and compact:				
	(a)Earthfill materials				
	(i) Placed in 1m thick layers in embankment	m^3	68	703 607	47 845 285
	(ii) Impervious fill placed in 250mm layers	m^3	68	64 732	4 401 802
	(iii) Filter sand placed in 1000mm layers	m^3	150	26 583	3 987 524
	(iv) Rock toe	m^3	300	3 696	1 108 800
	(b)Crest capping	m^3	68	16 638	1 131 377
	(c)Dupal surface treatmant	m^2	122	1 200	146 400
	(d)Precast concrete kerb	m^3	12 695	18	228 510
	(e)Cut off trench	m^3	125	240	30 000
6.1.7	Stormwater drain				
	(b) Precast elements:				
	(i) Excavated channel drain (concrete lined)	m^3	12 695	142	1 800 405
	(ii) Discharge chute	m^3	12 695	104	1 322 717
					105 196 437

FOXWOOD DAM

SECTION 9: Outlet works

	Dam Type and Size			**Composite 1MAR**	
Item No	DESCRIPTION	Unit	Rate	Quantity	Amount R
9,1	**Concrete Works**				
9.1.1	Site preparation (included in dam footprint & diversion)				
	(a) Clearing and grubbing including disposal of material	ha	35 200	0,01	406
	(b) Stripping and removal of unsuitable material	m³	60	35	2 076
9.1.2	All excavations for outlet structurre and pipe encasement (included in dam footprint & diversion)				
	(a) Overburden (soft and boulders)	m³	138	1 153	159 162
	(b) Excavations in rock (controlled blasting)	m³	263	231	60 666
9.1.3	Foundation preparation (included in dam footprint & diversion)				
	(a) rock foundation for concrete	m²	110	115,34	12 687
9.1.4	Formwork				
	(a) Class F3 (smooth) to permanently exposed surfaces				
	(i) Plain	m²	340	6 773	2 302 740
	(ii) Curved	m²	450	839	377 550
	(b) Class F1 (rough) to hidden surfaces				
	(i) Plain	m²	280		
9.1.5	Joints				
	(a) Formed vertical construction joints at 8m c/c in retaining walls	m²	630	130	81 601
	(b) Designated joints	m²	630		
9.1.6	Waterstops				
	(a) PVC waterstops 250mm wide	m	370	118	74 183
	(b) Rubber waterstops 355mm wide	m	580		
9.1.7	Anchor bars				
	(a) Y25 dia. 4m into rock at 1,5m c/c	No	420	52	1 839 552
	(b) Y25 dia. 2,5 m into rock and concrete at 1,5m c/c	No	690		
9.1.8	Reinforcement				
	(a) High tensile reinforcement bars in concrete				
	(i) In-situ concrete	t	11 700	482	5 643 068
	(ii) Precast concrete	t	11 700	1,76	20 534
9.1.9	Concrete				
	(a) In-situ concrete:				
	(i) Class 30/19 Structural concrete	m³	1 980	4823	9 549 807
	(ii) Class 25/38	m³	1 750	482	844 049
9.1.10	Surface finish				
	(a) Class U3 (smooth) to permanently exposed surfaces	m²	29	231	6 689
	(b) Class U1 (rough) to hidden surfaces	m²	19	368	6 983
9.1.11	Precast concrete				
	(a) Trash rack panels	m³	12 695	18	222 797,25
	TOTAL CARRIED FORWARD				21 204 550

FOXWOOD DAM

SECTION 9: Outlet works

	Dam Type and Size			**Composite 1MAR**	
Item No	DESCRIPTION	Unit	Rate	Quantity	Amount R
9,2	**MECHANICAL EQUIPMENT**				
9.2.1	Supply and installation of pipework and fittings				
	(c) 1 000mm NB lined stainless steel outlet pipes through conduit	m	67 299	16	1 076 776
9.2.2	Supply and installation of valves and fittings				
	(a) Coated and lined mild steel sleeve valves (ii) 1 000mm DN electrically actuatedbottom discharge valve	No	2 447 218	2	4 894 437
	(b) Butterfly valves (i) 1 000mm DN electrically actuated butterfly valve	No	2 141 316	10	21 413 160
9.2.3	Supply and installation of fines screens service/emergency gates, grapple beams, embedded and built-in parts				
	(a) Stainless steel				
	(i) Fine screens	No	152 951	8	1 223 609,15
	(iv) Built-in parts	t	45 885	1,0	43 664,49
	(i)Embedded parts	t			
	(ii)Service/emergency gate	No	4 588 534	2	9 177 068,59
	(iii)Screen & gate grapple beams	No	36 708	2	73 416,55
9.2.4	Supply and installation of winches, hoist and cranes				
	(a) Design, furnish and install 30 t overhead gantry crane in emergency gate control room	Sum			
	(b)Design, furnish and install 10t overgead gantry crane in valve outlet house	Sum			1 200 000
	TOTAL CARRIED FORWARD				39 102 131
9,3	**STRUCTURAL STEELWORK**				
9.3.1	Furnish and install miscellaneous structural steelwork, including corroison protection (i.e. ladders, floor grantings, walkways, platforms, handrails etc)	Lump Sum			1 750 000
	TOTAL CARRIED FORWARD				1 750 000
9,4	**ELECTRICAL EQUIPMENT**				
9.4.1	Design, furnish and install electrical equipment	Lump Sum			2 250 000
	TOTAL CARRIED FORWARD				2 250 000
	TOTAL TO SUMMARY				64 306 681

FOXWOOD DAM
SECTION 11: INSTRUMENTATION

	Dam Type and Size			**Composite 1MAR**	
Item No	DESCRIPTION	Unit	Rate	Quantity	Amount R
11,1	**INSTRUMENTATION**				
	(a) Design, furnish and install instrumentation	Lump Sum			7 500 000
	TOTAL TO SUMMARY				7 500 000

FOXWOOD DAM

SECTION 12: MISCELLANEOUS

	Dam Type and Size			**Composite 1MAR**	
Item No	DESCRIPTION	Unit	Rate	Quantity	Amount R
12,1	**Landscaping**				
12.1.1	Landscaping and rehabilitation (2.5% +10% miscellaneous - of construction cost excl. P&Gs)	Lump Sum			88 670 942
	TOTAL TO SUMMARY				88 670 942

APPENDIX I: DWS COMMENTS ON DRAFT FOXWOOD DAM DESIGN REPORT

water & sanitation

Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

Enq: J Van Zyl	VanZylJ@dwa.gov.za	☎	(012) 336-8478
Date: 23 January 2015			(012) 336-6994

Chief Director: Options Analysis

Private Bag X313

PRETORIA

0001

Attention: Ms. S. Van Jaarsveld

RE: REVIEW OF THE DRAFT FOXWOOD FEASIBILITY DAM DESIGN REPORT

With reference to the email dated 12 December 2014 to the Acting Director Civil Engineering requesting a review of the Foxwood Dam Feasibility Design Report. The following comments were compiled in order to assist you on engineering technical decisions during the design phase, as well as differentiating between what could be refined at the next level of the design and what needs to be resolved and decided at this level of the study, before finalising your report. On establishing a realistic cost estimate to reduce the possibility of large cost increase PFMA should be used.

A. Technical Comments

1. Stilling Basin

- The report uses three different levels for the stilling basin apron, the highest is 11 m higher than bed level. To remedy the return flow to the river, a rock bed channel is proposed. Such a channel is not sustainable. The stilling basin apron must be at bed level throughout to hold the hydraulic jump in position. A fan shaped excavation with the bed equal to riverbed is required downstream.

2. Spillway

- The above matter will influence the optimised spillway length. It is suggested that the optimal length will be in the order of 170 m to 190 m, due to the fan shaped excavation and the deep foundation level for the concrete gravity wall. The spillway should be limited as far as possible to the river section. This will also do away with the requirement of an expensive return channel back towards the river. The spillway length can be reduced based on higher unit discharges, e.g. in a similar fashion as the approach used at De Hoop Dam; (Please note minutes of meeting held on 26/03/2014; attached)

- With a reduced spillway length, the costly concrete retaining wall currently shown to retain the earth fill, can be done away with , and the tongue wall can be used to wrap the earthfill around at the embankment slopes used; The reduced spillway length results in a saving in this manner as well. The NOC will be approximately 1m higher, but this is unlikely to offset the optimised spillway length materially.

3. Outlet Works

- The advantages with regards to river diversion and construction time of having the outlet works situated on the R/B versus the L/B should be studied in further detail. This could be done at the next phase of the tender design.

- The outlet works are normally on the critical path, therefore the use of a diversion tunnel/culvert at the bottom of the outlet works in such a fashion that the pipes could only be installed once the diversion is closed off, is not a good idea. It will delay completion of the outlet works.

- If DWS are the owner of the dam, based on departmental operational requirements, a twin outlet pipe stack will be required, i.e. a duplicate system (Please note minutes of meeting held on 26/03/2014; attached)

4. Design

- The Department (DWS) does have certain operational standards and minimum design criteria for instance on the emergency gate arrangement, but this could be incorporated once a tender design is engaged with.

- The South African rivers, especially in this area, have high silt loads, and this should be considered in the loads. In the absence of silt calculations, for this level of analysis, the DWS adopts a silt load criteria of 1/3 of the dam wall height.

- The DWS would rather make use of comprehensive internal filters when placing dispersive soils, than stabilizing with gypsum for instance. Changing the chemistry of the fill influence the elastic properties is costly and also difficult to control during fill placement.

- The downstream slope will be flattened during the tender design, due to the fact that allowable tensile stresses have not been used as a criteria for the feasibility design.

- The design criteria for the concrete gravity structures should include allowable tensile stresses as a criteria.

- Using RCC to construct the mass gravity structures, in order to limit the heat of hydration, will result in a reduced specified compressive strength of 15 MPa, instead of the value used of 40 MPa.

5. Geotechnical Investigations

- The borrow pit investigations need to properly classify the fill materials in order to prevent claims and construction delays once a contractor is appointed. It is suggested that large-scale test pitting be done along the proposed foundations for the detailed design investigations.

6. Hydrological

- It is recommended that a figure of ***5 218 m^3/s*** be adopted for the SEF and that Figures 2 and 3 in the Report on Flood Hydrology and Figure 3 in the Feasibility Dam Design Report be represented in a logarithmic scale. (Refer to attached detailed comment)

B. General Comments

- The option with the road over the dam wall should be re- considered. An additional study with various options is required at this level due to environmental implications. DWS is not in favour of building across our dam spillways;
- Tables on page 4 and 62 : use the metric system of notation to prevent confusion, i.e. space as a thousand separator instead of commas;
- Font quality for the above Tables to be improved for better reading;
- 10 % contingencies for this level of estimate are quite low. It is suggested to increase to at least 15 - 20 %;
- 25 % as P+G's is towards the low side, recommend 30 - 40 %
- There is no indication of the volume of the mass and/or RCC volumes in the table on dam statistics;

ACTING DIRECTOR: CIVIL ENGINEERING

DATE: 23/01/2015

Cc: 1) A. Thobejane

2) N. van Deventer

3) J. van Zyl

4) N. Sithole

Attachment: 1) Minutes of meeting held on 26/03/2014

2) Nandoni Dam Brochure

3) Hydrology Review and Comment

Feasibility Study of Foxwood Dam (WP 10580)
Dam Options Design Review

water affairs
Department:
Water Affairs
REPUBLIC OF SOUTH AFRICA

Project title	Feasibility Study for Foxwood Dam (WP 10580)	Job number 225739-00	
Meeting name and number	Foxwood Dam Options Design Review 1/14	File reference	
Location	DWA, Pretoria	Time and date 10:30 hrs 26 March 2014	
Purpose of meeting	Review Dam Design Process and Option Selection		
Present	DWA: S van Jaarsveld, K van Deventer, J van Zyl, P Pyke, M Nugumo, S Mthethwa Arup: J Hampton, C Watermeyer, A Simmans		
Apologies			
Circulation	Those present Arup: J Bristow, M Tindale		

Action

1.1 Welcome & Introduction

SvJ welcomed everyone to the meeting and presented the purpose of the meeting, which is to arrive at a consensus on the way forward for determining the dam size selection in order to complete the conceptual design.

JH provided an historical overview of the development plans for Foxwood Dam and progress of the design services. The following have been issued;

- Inception Report
- Preliminary Report
  - The following studies have been completed;
- Geotechnical
- Hydrological
- Environmental reserve

Outstanding is the completion of Alternative Water Supplies

1.2 Water Needs

JH explained that Foxwood Dam water demands were primarily for projected water demands of Adelaide and for an agricultural development.

The population growth, based on the latest census figures of Adelaide was

Prepared by	J Hampton
Date of circulation	
Date of next meeting	

in decline. The current supply is run of river off river storage. This system is currently being refurbished. Based this studies hydrological analysis, this system is sufficient for current demand and could be further enhanced with an increase in storage capacity of the existing off river dam.

It was noted that development of the Dam would have a social economic impact on Adelaide, and could be the catalyst for reversal of population and economic decline of Adelaide by attracting further developments.

It was noted that the design of the Dam to date had been based on a yield of 1 MAR, and that this could support agricultural development of approximately 2,000ha. JH stressed, that the agricultural development would be subject to:

- Unit cost of water
- Government policy on irrigation scheme development; management; beneficiation and subsidy qualification. This currently not clear.

This has an impact on the Dam size selection, and therefore Arup required guidance on the way forward for Dam sizing.

The possibility of resolving the uncertainty of size is by making provision, in the design, for future dam raising. This would affect the dam type selection.

1.3 Dam Options

JH discussed the various merits of dam options that had been considered; Earthfill, Rockfill, Concrete Gravity. The conclusions that were being reached are as follows;

- Earthfill has been eliminated on account of quantity and quality of suitable materials. Coupled with the materials balance from the excavated spillway.
- Concrete Gravity is potentially cost effective for a dam size up to 1.5MAR.
- Rockfill is potentially cost effective for a dam size from 1.5 MAR upwards.

It was raised that a 2 MAR dam may be developed at marginal additional cost.

1.4 Discussion Points and Considerations

1.4.1 Cost Analysis

A decision could not be taken yet as to the appropriate size of the Dam without a cost and sensitivity analysis being undertaken, resulting in a unit price of water.

1.4.2 Dam Option

DWA indicated that the capacity of the Dam should not be less than 1.0MAR.

DWA indicated that a Composite Dam and Rolled Concrete Dam options should be considered.

DWA asked how the Dam site was arrived at. Arup stated that DWA had indicated the location of the site based on previous studies.

DWA flagged that the PI for a clay core, indicated marginal suitability. Consequently the quality and quantity of suitable clay was uncertain, and a potential risk.

Arup remarked that filter material would need to be manufactured, and a potential dolerite quarry was 6km from the dam site.

1.4.3 Other Matters

DWA stated that a diversion tunnel for a Rock Fill dam should cater for a 1 in 20 year event.

The spillway width could be allowed to cater for up to 40t/s/m which would reduce the spillway width to 150m.

DWA stated that a side channel spillway should be considered as an alternative.

DWA stated that the outlet works should cater for twin duplicated system, and be multi-level.

DWA commented on the concrete dam spillway steps at the crest, should be recessed and not protrude above the ogee line.

DWA stated that they preferred that the access road river crossing **not** be combined with the dam wall.

DWA stated that the hydro potential of 180kw will not be pursued by DWA / Eskom, but may be of interest to potential local IPP. Allowance for a such an off take connection should incorporated within the outlet system

A 1:50 year sediment allowance to be allowed for in the dead storage allowance.

The SDF flood with full freeboard would apply to where over topping of the of the wall is feasible.

1.5 Gauging Weir

DWA to supply details as to gauging requirements downstream of the Dam and possibly upstream.

1.6 Conclusion

A cost benefit analysis is required to achieve a determination of dam type and size.

DWA will then sign off on this determination for the feasibility study dam.

1.7 Closure

The meeting closed at 1230hrs

Flood Hydrology Comments

Safety Evaluation Flood (SEF)

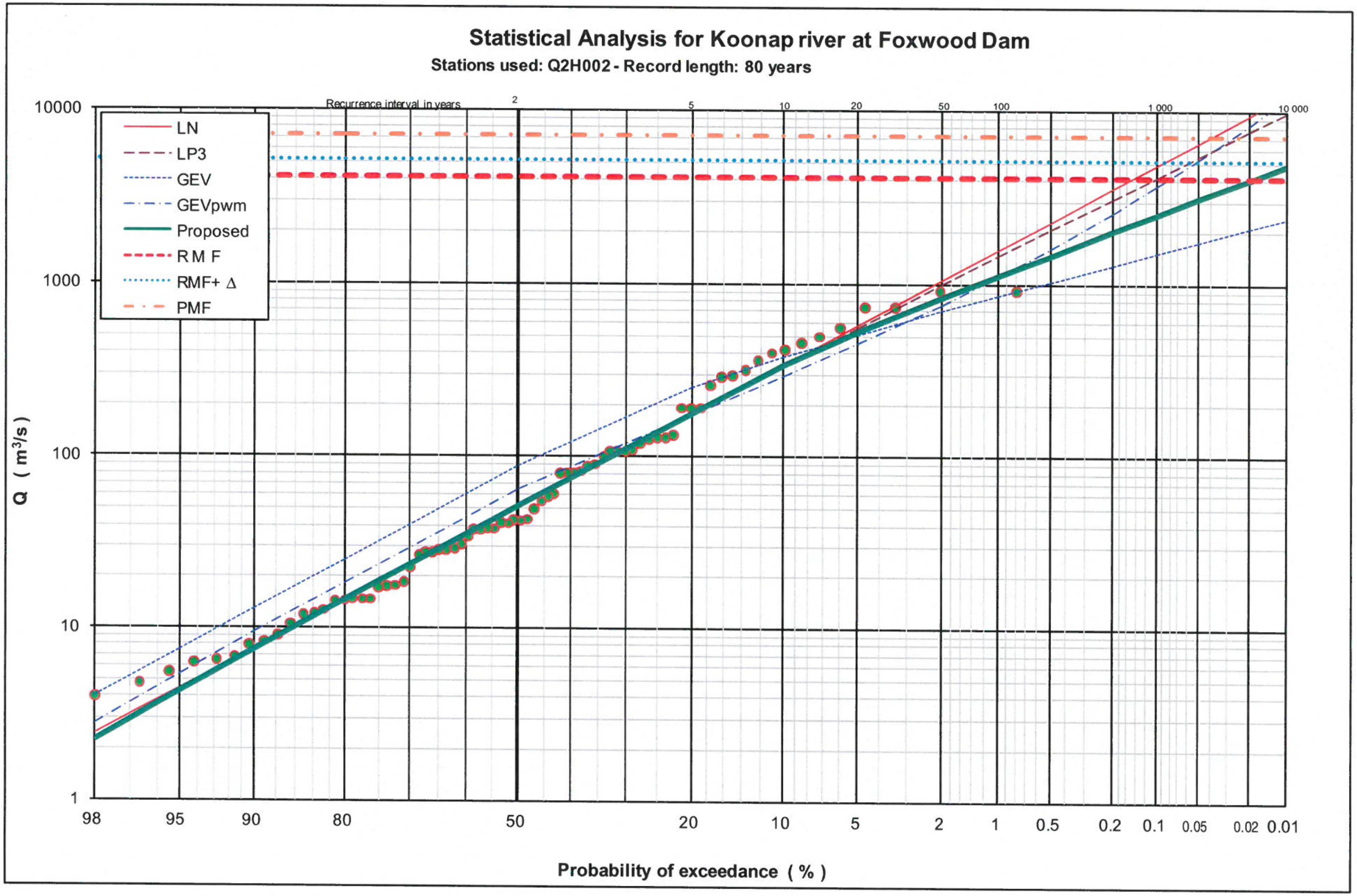
In South Africa meteorologists have not concerned themselves with Probable Maximum Precipitation (PMP) estimation like most overseas countries (used as input in the calculation of Probable Maximum Flood (PMF)). The only established PMP estimation procedure, for South Africa, is given in Report 1/72 of the University of the Witwatersrand's Hydrological Research Unit (HRU 1/72). Envelope curves for regions experiencing similar extreme point rainfalls for South Africa were developed. These envelope curves, as well as the extreme rainfall regions, need some serious revision. More than 30 years of additional data is now available. And this approach has a serious drawback (due to a lack of adequate data), in that it has to resort to not yet verified or unverifiable hypothesis and average rainfall coefficients.

The alternative approach to determine the maximum flood in South Africa is the use the Regional Maximum Flood (RMF). RMF estimates do not represent any specific return period, but from experience RMF estimates are considered to be associated with return period in the range of 1 000 to 10 000 years and is considered conservative as the upper envelope method.

According to the SANCOLD Guideline for a Category III Dam, the SEF is determined by adding a category to the adopted Kovacs K factors (RMF + Δ). Analysing the flow peaks in Annexure A in the Feasibility Study of Foxwood Dam: Report on Flood Hydrology. Assuming and accepting that the approach used to extent the Discharge Tables (DTs) is engineering sound. Because is a well know fact that extending DTs is not a straightforward excise, is complex in nature and needs sound engineering judgements based on structural and surrounding conditions. A good fit on the statistical analysis according to the combination of LP III (whole probability of exceedance (PE) range) and GEV_{mm} (from 5% PE), coincide closely to the RMF + Δ (K = 5.4) when extrapolated to the 1:10 000 year return period level and therefore it is recommended that a figure of ***5 218 m³/s*** be adopted for the SEF. (See the figure below)

Probability Analysis: Graphic Representation

A visual comparison between the data and the estimated probability distributions is fundamental. The graphical representation of the data, on log-probability paper, enables the analyst to compare the data with the results from the probability analysis. A standard practice is to use a logarithmic scale and is recommended that Figures 2 and 3 in Report on Flood Hydrology and Figure 3 in Feasibility Dam Design Report as well be represented in a logarithmic scale.



Statistical Analysis for Koonap river at Foxwood Dam
Stations used: Q2H002 - Record length: 80 years
Recurrence interval in years
2
5
10
20
50
100
1 000
10 000
10000
1000
100
10
1
Q (m³/s)
LN
LP3
GEV
GEVpwm
Proposed
R M F
RMF+ Δ
PMF
98
95
90
80
50
20
10
5
2
1
0.5
0.2
0.1
0.05
0.02
0.01
Probability of exceedance (%)

Your ref
Our ref 225739/JAH
File ref JAH/9/2/15

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Mrs S Van Jaarsveld
Department of Water and Sanitation
Pvt Bag X313
Pretoria
0001

10 February 2015

Dear Sanet

Feasibility Study for Foxwood Dam

Review of the Draft Foxwood Feasibility Study Design Report

Referring to the comments in the review document referenced above and following the clarification meeting held at DWS offices on 28 January 2015 we are confirming the following matters will be addressed either in the RID or revised Design Report:

1. **Stilling Basin**
   It is noted that the stepped spill basin with the current large bolster return may not be sustainable and that the stilling basin should be at the riverbed level; and that the return should be soft excavated in a fan shape to allow for normal return to the natural river line.

   This will be noted in the the RID report to be addressed during detailed design.

2. **Spillway**
   We have included in the RID a modified table for spillway optimisation using the Kovaks + Δ method which indicates a reduced size of spillway length. This will be addressed at detailed design with the modelling that will occur.

   The L shaped wall and the proposed rap around retaining wall alternative will be included in the RID.

   Quantities will be adjusted in the BoQ to reflect reduce length of spillway and increased embankment length.

3. **Outlet Works**
   Following our meeting we are modifying the outlets to allow for twin stacks. The design report and drawings will be editted accordingly.

The diversion will be incorporated in the construction programme with a large box culvert incorporated in the main wall or adjacent to the outlet on the right bank. This will be noted in the RID to be addressed during detailed design.

4. **Dam Design**
The estimation of silt was undertaken by Bill Pitman and is based on the Koonap catchment which falls within Region 9 of the sediment yield potential map of Southern Africa (WR90, Vol. 5, Map 8.2, 1994). The estimated average rate of sedimentation in the Upper Koonap catchments is 185 tons\km²\annum based on the Rooseboom methodology (Rooseboom, et al, 1992). This reported a sediment yield of 6 million m3. If a more detailed analysis is required it is suggested that this be incorporated within the detailed design and will be covered in the RID. This will be noted in the RID
.
Regarding the concrete strengths we analysed a concrete gravity structure and this incorporated a 40 MPa concrete, however, in the BoQ we have used current RCC costs.

5. **Geotechnical**
There are comments regarding the need for detailed investigation of the selected borrow areas in detailed design stage. This is noted in the Geotechnical Investigation report and also in the Dam Design Report and will be recorded in the RID as well.

6. **Hydrological**
We have incorporated a table in the RID and will carry this through to the Design Report. I will this discuss with Archie Thobejane with proposed write up.

General Comments

- We discussed the issue of a bridge over the spillway and acknowledge the reservations strongly expressed. We will incorporate these reservations and alternatives in both the Dam design report and in the RID.
-
- Regarding contingencies we will adjust once we have incorporated the spillway modifications.

I trust this is covers the items we discussed and that the measures to incorporate them are satisfactory to you.

Yours sincerely

James Hampton

APPENDIX J: FLOOD HYDROLOGY REPORT

DEPARTMENT OF WATER AFFAIRS

FEASIBILITY STUDY OF FOXWOOD DAM

Report on Flood Hydrology. Bill Pitman

Draft 3: 5th March 2014

INTRODUCTION

The proposed Foxwood Dam is to be situated on the Koonap River, near the town of Adelaide in the Eastern Cape. The Koonap River, which has a total catchment area of 3334 km^2, is an important tributary of the Great Fish River. Foxwood Dam will command a catchment area of 1091 km^2, which is approximately 33 % of the total area. The catchment of Foxwood lies in the summer rainfall zone, but is close to the year-round zone. Accordingly, flood producing rain can occur at any time of the year.

Flood discharges are required for a range of return periods, including the PMF (Probable Maximum Flood), as they are required for diversion during construction and for sizing the spillway. There is a DWA flow gauge, station number Q9H002, situated on the Koonap River downstream of the Foxwood site. It has a catchment area of 1250 km^2, i.e. some 15% greater than at the dam. Station Q9H002 was opened in 1926 and has a continuous record since 1933, thus providing an 80-year time series of annual peaks – more than adequate for the purpose of a flood-frequency analysis.

FLOOD FREQUENCY ANALYSIS OF RECORD AT Q9H002

As is the case with most streamflow gauges spanning a long period, the DT (Discharge Table) has been revised several times. Only the latest DT (no. 10, used since July 1983) covers the full range of flows experienced at the gauge. The DTs used before 1983 had a more limited range, hence some of the flood peaks exceeded the DT limits. It was necessary, therefore, to extrapolate these DTs to obtain estimates of the peak flows.

In addition to DT10 there were two other DTs that were in operation during times of flood, namely DT6 and DT8. The stage-discharge curves from these DTs are plotted on **Figure 1**. It can be seen that DT8 and DT10 are quite close up to the limit of DT8 at a stage of 3m. Extrapolation of DT8, on the basis of a power curve, is reasonably close to DT10 and thus appears to be acceptable. On the other hand, a straightforward extrapolation of DT6 looked unrealistically high with discharges about double that of DT10 for the same stage. Accordingly, a different type of extrapolation was adopted. Up to the limit of DT6 (1.8m) it was noted that the stage for DT10 was approximately 0.5m higher than that of DT6 for the same discharge. Extrapolation of DT6 was therefore accomplished by subtracting 0.5m from DT10. The extrapolated stage-discharge curves for DT6 & DT8 are shown in **Figure 1**. A tabulation of the maximum peaks in each hydrological year is given in **Annexure A**, where it is shown which peaks have been derived by extrapolation of the DT.

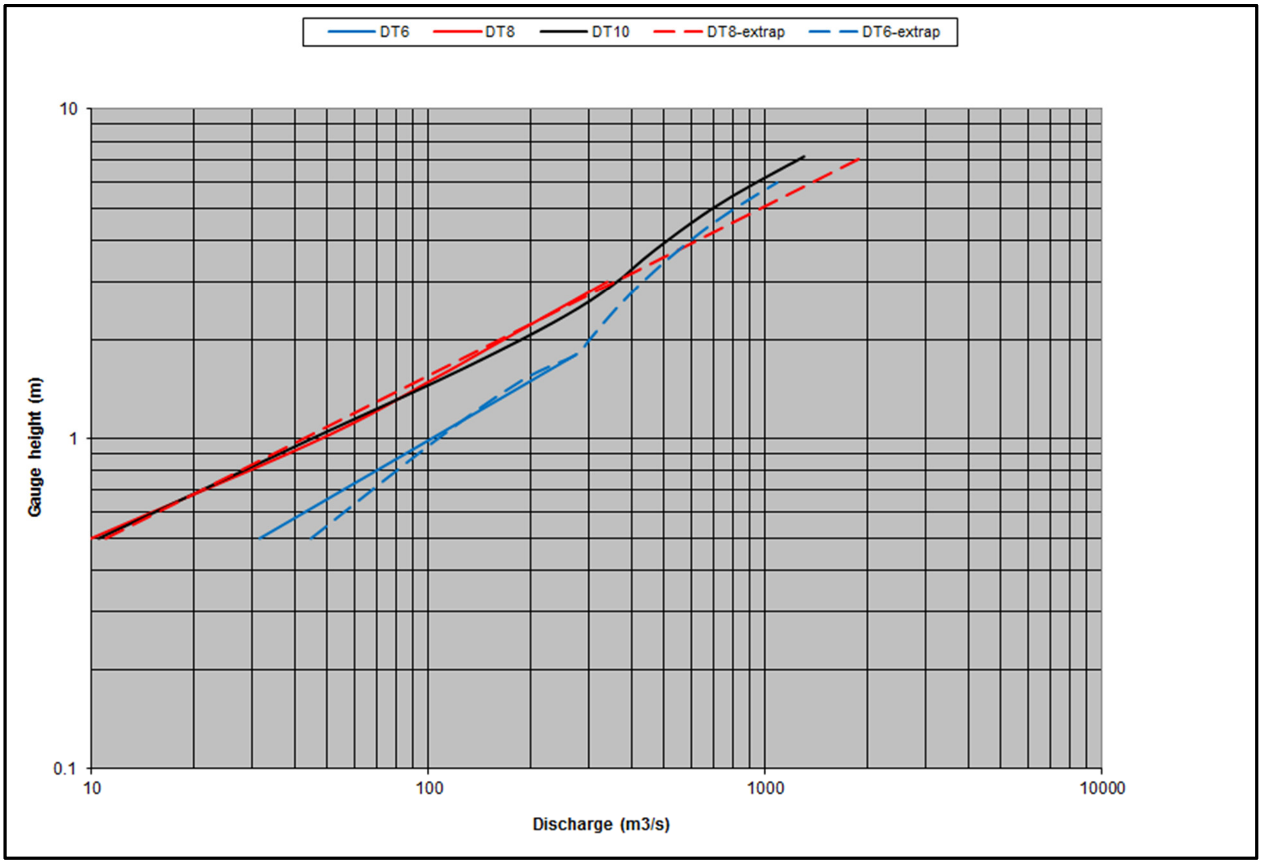


Figure 1 – Q9H002 Stage:Discharge Curves

The data in **Annexure A** were subject to a frequency analysis. Several probability distributions were tried, including Normal, Log Normal, Exponential, Log Pearson Type III, Extreme Value Type 1 (Gumbel) and Log Extreme Value Type 1 (Log Gumbel). The Log Pearson Type III (LP3) yielded the best fit to the data and the resulting frequency curve is shown in **Figure 2.**

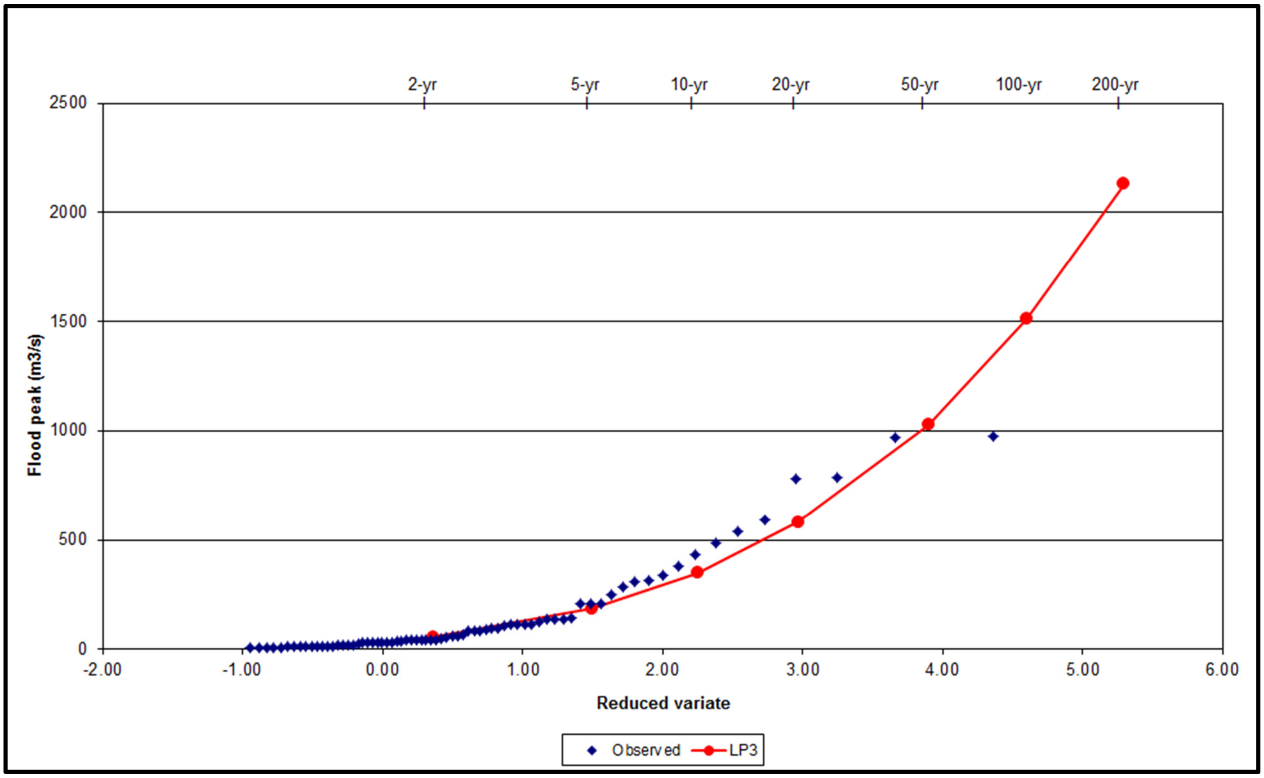


Figure 2 – Q9H002 Flood Frequency Curve

As Foxwood Dam is about 6.5 km upstream of gauge Q9H002, it is necessary to adjust the flood peaks on the basis of catchment area. The accepted adjustment for flood peaks is to scale them up or down according to the ratio of the square root of catchment area. The ratio in this case is:

$$(1091/1250)^{0.5} = 0.934$$

The flood peaks at Foxwood Dam are derived from the frequency analysis at Q9H002 and are scaled down to apply to the dam site as summarised in **Table 1**.

Table 1 – Flood-frequency relationships for Q9H002 and Foxwood Dam

Return period	Peak flood discharge (m^3/s)	
	Q9H002	Foxwood Dam
5-year	188	176
10-year	355	332
20-year	594	555
50-year	1054	985
100-year	1560	1457
200-year	2208	2063

COMPARISON WITH OTHER METHODS

In cases where there is no flood data it is advisable to adopt a number of different methods of design flood determination and to select the final results based on experience. Furthermore, the spread of results gives an idea of the confidence in the final answers. Even where there is adequate data, such as here, it is often a good idea to compare the results with different methods. Each of the methods is discussed in turn, followed by an overall assessment and selection of the final results.

Characteristics of the Foxwood catchment that are used to determine flood peaks via the various methods are summarized below.

Catchment area	1091 km^2
HRU Flood Region	3
Mean annual precipitation	598 mm
Length of longest watercourse	75 km
Length to centre of area	33 km
Average slope	0.014
HRU veld zone	5
Francou-Rodier “K” for RMF	5.2
Region number for SDF	21

1. Unit hydrograph analysis

This method is applied using the data in Hydrological Research Unit (HRU) Report No. 1/72. Synthetic 1-hour unit hydrographs are derived from catchment characteristics and veld zone, which are then used to construct hydrographs of any desired duration. Design storm rainfall and loss data is provided to dimensionalize the hydrographs for a range of return periods and the PMF. Results of this method are compared with that of the (area-adjusted Q9H002) Foxwood Dam frequency analysis in **Table 2**, where it can be seen that the two peaks are close for the 10-year flood but then diverge, with the unit hydrograph yielding the lower peaks.

Table 2 – Comparison with Unit Hydrograph Analysis

Return period	Peak flood discharge (m^3/s)		Percentage difference
	Q9H002 - adjusted	Unit hydrograph	
10-year	332	334	+1
20-year	555	476	-14
50-year	985	739	-25
100-year	1459	1043	-29

The equations given in HRU Report No. 1/71 were designed to replicate – as nearly as possible – the flood peaks derived by the unit hydrograph method as used above. It was therefore considered unnecessary to use both methods.

2. HRU Regional method

Appendix B of HRU Report No. 1/72 contains a regional map and co-axial diagram to provide quick estimates of flood peaks for return periods ranging from 5 to 200 years. The method cannot be considered accurate since it is based only on catchment area and location. Results of this method are compared with that of the (area-adjusted) Q9H002 frequency analysis in **Table 3**, where it can be seen that the HRU 5-year peak is about double but that the two frequency curves converge at high return periods.

Table 3 – Comparison with HRU Regional Analysis

Return period	Peak flood discharge (m^3/s)		Percentage difference
	Q9H002 - adjusted	HRU Regional	
5-year	176	349	+98
10-year	332	548	+65
20-year	555	787	+42
50-year	985	1165	+18
100-year	1457	1500	+3
200-year	2063	1877	-6

3. Rational Formula

The Rational Formula is designed for application to catchment areas of about 10 km^2 or less, nevertheless it is often used by hydrologists for much larger catchments. The Rational Formula is as follows:

Q_t = $0.278*A*C_t *I_t$

Q_t = Flood peak (m^3/s) for return period t

A = Foxwood Dam Catchment area (km^2)

C_t = Runoff coefficient for return period t

I_t = Rainfall intensity (mm/h) for return period and critical storm duration, which is determined by length and slope of main watercourse

Results of this method are compared with that of the (area-adjusted) Q9H002 frequency analysis in **Table 4**, where it can be seen that the peaks derived by the Rational Formula are generally very much lower than those obtained from the frequency analysis.

Table 4 – Comparison with Rational Method

Return period	**Peak flood discharge (m^3/s)**		**Percentage difference**
	Q9H002 - adjusted	**Rational Formula**	
5-year	176	203	+15
10-year	332	257	-23
20-year	555	317	-43
50-year	985	511	-48
100-year	1457	758	-48

4. Kovacs Method

The DWA Technical Report TR137 contains a map showing Francou-Rodier "K" zones and a set of equations for derivation of the Regional Maximum Flood (RMF).The report also contains factors which, when applied to the RMF, can be used to derive quick estimates of the 50-, 100- and 200-year flood peaks. The value of "K" for the Koonap River is 5.2. It has the same shortcoming as the HRU Regional Method in that it is based only on catchment area and location. Results of this method are compared with that of the (area-adjusted) Q9H002 frequency analysis in **Table 5**, where it can be seen that the peaks derived by the Kovacs Method are generally very much higher than those obtained from the frequency analysis. The main reason for this result is that Kovacs intended the factors to be conservative upper bound factors.

Table 5 – Comparison with Kovacs Method

Return period	Peak flood discharge (m^3/s)		Percentage difference
	Q9H002 - adjusted	Kovacs Method	
50-year	985	1865	+89
100-year	1457	2318	+59
200-year	2063	2816	+37
RMF	-	4164	-

5. **Standard Design Flood (SDF)**

This method is a relatively simple, but robust, method developed by Alexander (SAICE Journal, 44(1), 2002). Model parameters are provided for 29 drainage areas. The main drawback of this method is that many of the drainage areas include catchments with diverse characteristics. Results of this method are compared with that of the (area-adjusted) Q9H002 frequency analysis in **Table 6**, where it can be seen that the peaks derived by the SDF Method are higher than those obtained from the frequency analysis for low return periods but the converse is true for the longer return periods.

Table 6 – Comparison with SDF method

Return period	Peak flood discharge (m^3/s)		Percentage difference
	Q9H002 - adjusted	SDF Method	
10-year	332	438	+32
20-year	555	629	+13
50-year	985	913	-7
100-year	1457	1151	-21

Table 7 gives a comparison among all the different methods used to determine the flood-frequency analysis for Foxwood Dam. The Rational formula consistently yields lower peaks than the Q9H002 analysis (adjusted for catchment area), whereas the converse is true for the Kovacs method. The remaining three methods are much closer but have flatter slopes, with the result that peaks for shorter return periods are above Q9H002 (adjusted) and are below Q9H002 (adjusted) for the longer return periods.

Table 7 – Comparison among all methods (peak discharges in m^3/s)

Return period	Q9H002 adjusted	Unit hydrograph	HRU Regional	Rational Formula	Kovacs method	SDF method
5-year	176	223	349	203	710	282
10-year	332	334	548	257	1008	438
20-year	555	476	787	317	1346	629
50-year	985	739	1165	511	1865	913
100-year	1457	1043	1500	758	2318	1151
200-year	2063	1350	1877	1084	2816	1420

The frequency curves derived from the various methods are compared in **Figure 3**, where it can be seen that the curve obtained from analysis of the streamflow record at Q9H002 is somewhat steeper than the other curves. Only the Unit Hydrograph and Rational Formula yield similar peaks for the 5-year flood, with the other three being somewhat higher. The Kovacs curve is consistently the highest for all return periods, whereas the other curves – apart from the HRU Regional curve – reflect lower peaks for return periods in excess of 50 years. Nevertheless, the curve based on Q9H002 falls within the range of the curves obtained from the different methods and can be taken to be a reasonable estimate of the flood-frequency relationship at Foxwood Dam.

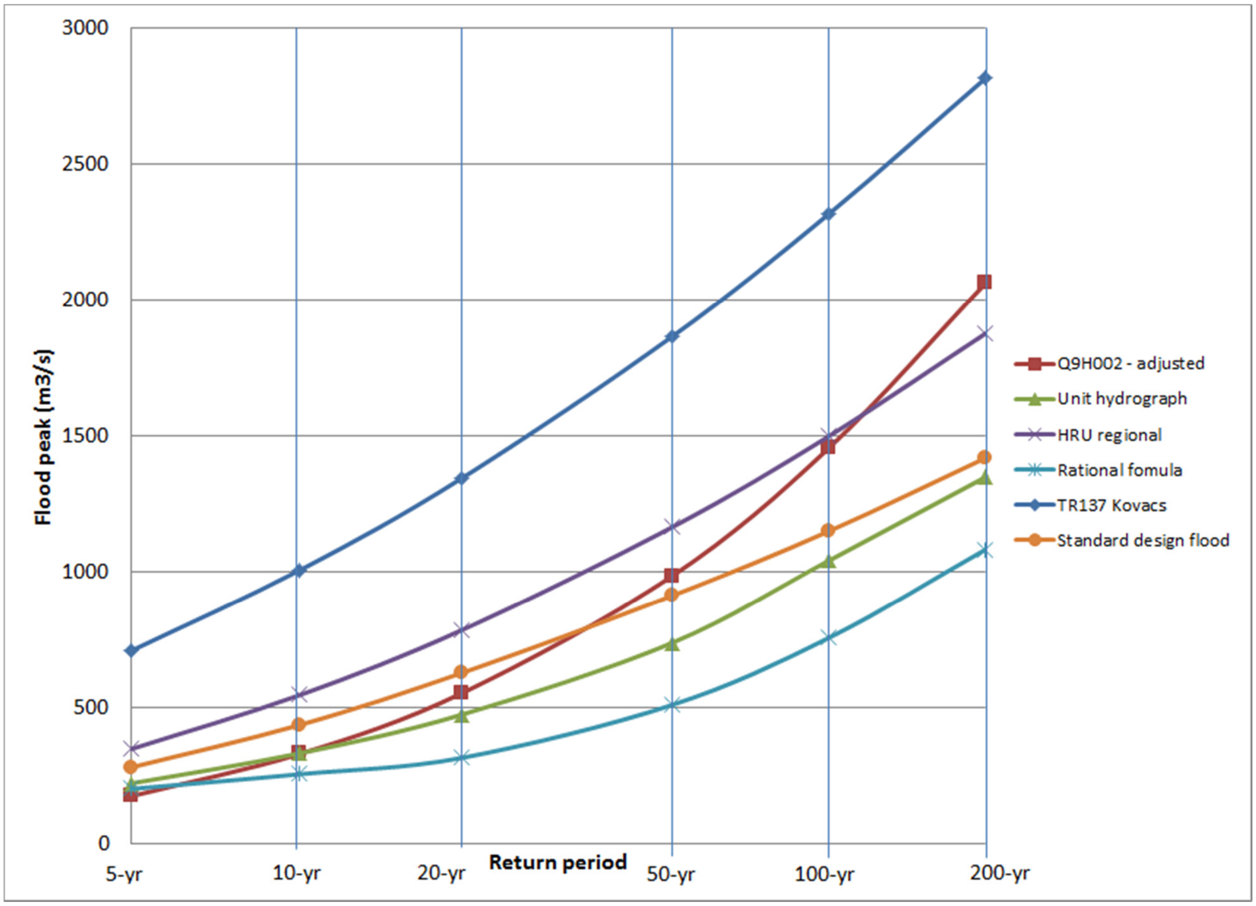


Figure 3 – Comparison of Flood Frequency Curves at Foxwood Dam

PROBABLE MAXIMUM FLOOD (PMF)

In the absence of adequate data on extreme storms, the 1 in 10,000 flood is sometimes adopted for the PMF. Extrapolation of the frequency curve in **Figure 2** yields a 1 in 10,000 flood peak of 9900 m^3/s (before adjustment for catchment area). However, such a large extrapolation is not reliable and depends to a great degree on the frequency distribution adopted. Another quick method is to calculate the RMF at the site using a "K" value for the next highest region – in this case 5.4. The resulting flood peak is 5218 m^3/s. (When using this value, no reduction is allowed for flood attenuation.). As PMF hydrographs are required for routing purposes the Unit hydrograph method has been adopted. This method yielded the following PMF hydrographs covering a range of storm durations and the results are shown in **Table 8**. Note that the 3-hour storm yielded the highest peak of 7250 m^3/s, which is approximately 1.75 times the RMF of 4164 m^3/s (see Table 5). As this ratio is less than 2 the PMF peak can be considered a reasonable, if conservative, estimate.

Table 8 – Hydrographs for the PMF

Time hours	Discharge (m^3/s) for a range of storm durations				
	3-hour	4-hour	6-hour	8-hour	12-hour
0	0	0	0	0	0
1	191	153	114	94	69
2	868	696	518	427	316
3	2944	2360	1759	1448	1072
4	5666	4695	3498	2880	2132
5	**7250**	6356	4850	3993	2956
6	6672	**7014**	5744	4729	3501
7	4896	6260	6309	5288	3915
8	3537	4650	**6444**	5733	4244
9	2781	3431	5647	**6004**	4514
10	2255	2718	4721	5970	4736
11	1845	2204	3215	5192	4916
12	1509	1804	2563	3959	5063
13	1237	1480	2086	3013	**5117**
14	1025	1218	1714	2415	4973
15	856	1011	1412	1972	4303
16	714	844	1166	1624	3314
17	596	784	968	1340	2550
18	493	584	806	1106	2053
19	403	480	667	915	1678
20	319	386	546	755	1378
21	244	302	440	617	1129
22	175	226	345	497	921
23	118	159	261	392	749
24	65	99	185	298	604

ROUTING OF PMF HYDROGRAPHS THROUGH RESERVOIR

A survey of the reservoir basin of the proposed Foxwood Dam yielded the relationship between wall height and storage volume for capacities up to and beyond 2 nMAR. This information, in conjunction with depth-discharge characteristics of the proposed spillway, enable the flood routing exercise to be undertaken to determine the maximum depth and discharge over the spillway during passage of the PMF.

A 250m wide spillway was assumed with a coefficient of 2.0, yielding the following relationship between depth and discharge:

$$\text{Discharge } (m^3/s) = 2.0 \times 250 \times (\text{Depth } (m))^{1.5}$$

Four reservoir capacities were analysed as shown in **Table 9**. The data is based on a (natural) MAR of 47.61 million m^3 and a dead storage (for sediment accumulation) of 6.11 million m^3.

Table 9 – Full supply capacity (FSC) and spillway crest elevations for selected dam sizes

Live capacity (%MAR)	FSC (million m^3)	Crest elevation (m)
50	29.92	608.5
100	53.72	614.6
150	77.52	619.5
200	101.33	623.1

The PMF flood hydrographs in Table 8 were routed through the reservoir, starting with the 3-hour (i.e. shortest storm duration) hydrograph. It was not necessary to route any hydrograph longer than a 6 hour duration as it became obvious that the longest duration yielding the highest peak was only 4 hours. The results are summarized in **Table 10** below, where the greatest peaks for each dam size are highlighted

Table 10 – Maximum PMF discharge and depth for a range of dam sizes

Storm Duration (h)	50% MAR dam		100% MAR dam		150% MAR dam		200% MAR dam	
	Disch. (m^3/s)	Depth (m)	Disch. (m^3/s)	Depth (m)	Disch. (m^3/s)	Depth (m)	Disch. (m^3/s)	Depth (m)
3	**6694**	**5.5**	6126		5816		5750	
4	6555		**6200**	**5.4**	**6035**	**5.2**	**5954**	**4.9**
6			5990		5937		5897	

ANNEXURE A

Maximum annual peaks at gauge Q9H002

Hydro year	Date	Peak height (m)	DT number	Peak flow (m3/s)	Extrapolated peak (m3/s)	Notes
1932	19330403	1.067	6	113.035		
1933	19340224	1.829	6	273.16	273.16	just over limit
1934	19350513	1.189	6	135.687		
1935	19351110	0.366	6	18.583		
1936	19361115	0.975	6	97.082		
1937	19371218	5.639	6	273.16	976	
1938	19390217	1.524	6	206.275		
1939	19400320	1.219	6	141.515		
1940	19410406	0.305	6	13.662		
1941	19411029	1.067	6	113.035		
1942	19421112	0.914	6	87.054		
1943	19440528	1.524	6	206.275		
1944	19441024	0.168	6	5.05		
1945	19460210	0.488	6	30.195		
1946	19461126	0.914	6	87.054		
1947	19480420	3.658	6	273.16	536	
1948	19490228	1.524	6	206.275		
1949	19500312	3.048	6	273.16	439	
1950	19501208	1.189	6	135.687		
1951	19520303	0.579	6	40.294		
1952	19530914	1.067	6	113.035		
1953	19531022	4.877	6	273.16	777	
1954	19541018	0.329	6	15.524		
1955	19560923	0.229	6	8.424		
1956	19561102	3.962	6	273.16	590	
1957	19580118	0.238	6	8.99		
1958	19581215	0.61	6	44.001		
1959	19600131	0.512	6	32.743		
1960	19601229	0.762	6	64.047		
1961	19620321	1.143	6	126.95		
1962	19630407	0.762	7	59.144		
1963	19640326	1.067	7	108.328		
1964	19641019	0.299	7	9.581		
1965	19651104	1.219	7	134.898		
1966	19670128	0.674	7	46.762		
1967	19680917					no data - omitted from analysis
1968	19690718					no data - omitted from analysis
1969	19700829	0.98	8	45.497		
1970	19710822	3.491	8	360.56	430	
1971	19720330	0.805	8	29.146		

1972	19730329	0.437	8	7.334		
1973	19740304	5.4	8	360.56	782	
1974	19750318	0.554	8	12.524		
1975	19760126	6.12	8	360.56	970	highest on record
1976	19761005	0.982	8	45.708		
1977	19780101	0.795	8	28.333		
1978	19790721	0.665	4	11.72	19	minor event - extrapolation unimportant
1979	19800112	0.835	4	11.72	31	minor event - extrapolation unimportant
1980	19810325	0.727	4	11.72	24	minor event - extrapolation unimportant
1981	19811209	0.42	4	6.914		
1982	19830728	0.518	4	11.351		
1983	19840321	0.547	10	12.844		
1984	19850210	0.641	10	18.165		
1985	19851103	2.638	10	305.662		
1986	19861109	0.996	10	44.319		
1987	19880307	0.599	10	15.7		
1988	19890422	0.601	10	15.814		
1989	19891116	2.671	10	312.085		
1990	19901130	0.412	10	6.659		
1991	19911030	0.821	10	30.291		
1992	19930228	0.161	10	0.633		
1993	19931205	0.904	10	36.663		
1994	19950114	0.955	10	40.803		
1995	19960127	1.082	10	52.386		
1996	19970613	1.56	10	115.34		
1997	19980309	0.664	10	19.573		
1998	19981215	0.389	10	5.85		
1999	20000326	1.36	10	85.238		
2000	20001118	2.805	10	336.555		
2001	20020910	3.15	10	381.805		
2002	20030512	0.336	10	4.184		
2003	20040926	1.429	10	95.388		
2004	20050307	0.809	10	29.408		
2005	20060803	1.384	10	88.738		
2006	20061010	0.957	10	40.969		
2007	20071227	0.943	10	39.814		
2008	20090314	0.602	10	15.871		
2009	20100111	0.596	10	15.529		
2010	20110610	3.842	10	484.25		
2011	20120331	1.176	10	62.342		

ANNEXURE B: FREEBOARD CALCULATION

Summary run-up earthfill 1MAR

Dam Parameters Ref: values pulled out of CAD

Dam size (MAR)	Fetch length (m)	Average depth along fetch length (m)	Basin area(m2)	Full supply Capacity(million $m^{3)}$	Average basin depth (m)
1	3279	18.919	4630000	53.72	11.603

Mean hourly wind speed U [m/s] (ref SANCOLD 2011. Guidelines on freeboard for dams. Volume II. Fig 2.3-2, Fig 2.3-3, Fig 2.3-4

return period	wind speed (m/s)
1:25	19
1:50	20
1:100	21

NB: the following sheets for run-up calculation where obtained from the CD accompanying the document SANCOLD 2011. Guidelines on freeboard for dams. Volume II

Calculation of Hs and Tp according to (a) Saville, (b) Doneland and (c) Young & Verhagen

	1:25 yrs	1:50 yrs	1:100 yrs
Hs (m)	0.86	0.91	0.96
Tp (s)	3.65	3.75	3.84

Calculation of wave run-upon a defined sloped face of a dam wall due to the wind generated wave calculated

Run-up for rough slopes - reduction factor applied to smooth slope formulae;Ahrens (1981), Allsop (2985), TAW (2002)

	1:25 yrs	1:50 yrs	1:100 yrs
Ahrens (1981)	0.93	0.98	1.04
Allsop et al (1985)	Out of range		
TWA (2002a)	0.92	0.98	1.03
Rough slopes - explicit formula	0.80	0.85	0.89

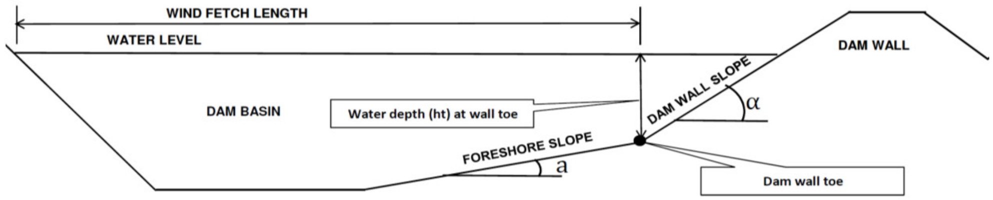
Reduction factors of wave run-up due to: (a) oblique wave attack, (b) wave breaking on shallow foreshore and (c) bermed slope

Reduction factor due to shallow foreshore, γh = (H2%/Hs)/1.4

1:25 yrs	1:50 yrs	1:100 yrs
0.95	0.94	0.92

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]	18.9		
	Chosen wind fetch, F [m]	3 279		
	Dam wall slope [verical/horizintal e.g. 1/3=0.333 = tanα]	0.25		
	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]	19.0	20.0	21.0

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]		0.86	0.91	0.96
	TRM Chapter4 ; Page 369	**Tp** [s] - Based on Ts ≈ 0.93Tp		3.65	3.75	3.84
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]		0.74	0.79	0.84
		Tp [s]		2.95	3.03	3.11
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=		0.299	0.277	0.258
		B1=		0.040	0.038	0.036
		A2=		0.169	0.152	0.138
		B2=		0.014	0.013	0.012
		Hs [m]		0.54	0.57	0.60
		Tp [s]		2.98	3.06	3.13

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))*	**0.86**	**0.91**	**0.96**
Tp [s] *(corresponding Tp of Hs selected)*	**3.65**	**3.75**	**3.84**

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 ≈ 1.4			
	H2%=[m]	**1.20**	**1.27**	**1.35**
	$T_{m-1,0}$ =Tp/1.1	3.32	3.40	3.49
	Tm = 0.79Tp	**2.88**	**2.96**	**3.03**
WRCF § 2.6	ξ=tanα/(Hs/(1.56T²))^0.5			
TRM Page488	$ξ_p$=tanα/(Hs/(1.56(Tp)²))^0.5	1.23	1.23	1.22
	$ξ_{m-1,0}$=tanα/(Hs/(1.56($T_{m-1,0}$)²))^0.5	1.12	1.12	1.11
	$ξ_m$=tanα/(Hs/(1.56(Tm)²))^0.5	0.97	0.97	0.97

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]	1.7	1.8	1.9
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	1.6	1.7	1.8
		R2% Mean + standard deviation	1.7	1.8	1.9
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	1.50	1.59	1.67
		Armour stone - single layer on impermeable base	1.18	1.25	1.32
		Armour stone - double layer on impermeable base	0.93	0.98	1.04
2(b) ii	**Allsop et al (1985)**		**1:25 yrs**	**1:50 yrs**	**1:100 yrs**
	TRM Page 492	$R_{2\%}$ = γf Hs(Aξ_p + B) [m]	Eq 5.7		
		$R_{2\%}$ = γf Hs(Aξ_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	$R_{2\%}$ = γf 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	1.41	1.49	1.57
		: Mean + standard deviation	1.49	1.58	1.67
		Armour stone - single layer on impermeable base : Mean	1.11	1.17	1.23
		: Mean + standard deviation	1.17	1.24	1.31
		Armour stone - double layer on impermeable base : Mean	0.87	0.92	0.97
		: Mean + standard deviation	0.92	0.98	1.03
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(0.96ξ_m) for ξ_m < 1.5 and R2% = Hs(1.17ξ_m^0.46) for ξ_m > 1.5	0.80	0.85	0.89

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 :	γβ = 1 - 0.0022|β| for 0º ≤ |β| ≤ 80º wave direction measured from direction of normal to slope (i.e. normal direction=0º)			
		Input wave approach direction :	0.0	Degrees [º]	
		Reduction factor due to obique wave attack, γβ =	1.00		
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) **:**	0.020	(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+5.8tan(a))h_t]/[(0.6725+0.2025(Hs/h_t))Hs]	1.434	1.342	1.259
	Page 360 ; Eq 4.60:	$H_{2\%}$ on shallow foreshore in m = ($H_{2\%}$/H_{rms})Table 4.10H_{rms}	1.14	1.19	1.25
	TRM Page 496 : Eq. 5.14:	Reduction factor due to shallow foreshore, γh = ($H_{2\%}$/Hs)/1.4	0.95	0.94	0.92
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)

APPENDIX K: DAM OPTION SELECTION REPORT

Foxwood Dam Options Selection Memorandum - Summary

Following a meeting at DWA on 20 May 2014 it was requested that Arup clarify the criteria adopted to determine the proposed size of dam and the most appropriate dam type for that size at the Foxwood Dam site. This memorandum is a response to that request.

A more comprehensive memorandum, including details of estimated construction costs, has been provided to DWA.

1.1 Regional Water Resource Context

Arup have carried out a review of the potential of the Koonap River to contribute to regional water resource management strategies within the Great Fish River catchment. A memorandum entitled ***'Foxwood Dam – Water Resource Context'*** which was issued to DWA on 25 June 2014. conclude that:

> *From an operational point of view therefore, the opportunity for the Koonap River to add useful water to the Great Fish River is limited. It is therefore concluded that maximum benefit can be made of the water resource of the Koonap River for stimulating socio-economic development in that catchment.*

1.2 Summary of Water Resources

The Water Resources report for the study notes that:

'The EWR operating rule recommended for the Foxwood Dam system is that high flow EWRs should be met by spills from Foxwood Dam and that the low flow EWRs can be met by inflows from the incremental catchments downstream of Foxwood Dam. This operating rule impacts the storage size of Foxwood Dam as it is important that regular spills can occur.'

The abridged Scenario 3 table reproduced below indicates the yields that are available (for various degrees of assurance) where high EWR flows are supplied by natural spills from the dam and not by releases from the dam. This criterion (ie high flow EWR's supplied by spillages) is satisfied only for dam capacities ≤ 1 MAR. In these circumstances the critical period is relatively short and natural spills from the dam would satisfy the high flow EWR's. The maximum yield available when this criterion is satisfied is $19.1 x 10^6 m^3/a$ at 95% assurance and for a 1MAR dam.

Scenario 3 – Foxwood Dam system with low flow EWR supplied by releases, high flows from spills

Reservoir capacity as a ratio of nMAR	**Live storage**	**Dead Storage**	**FSC**	**Long term yield ($10^6 m^3/a$) at Recurrence Interval**		
	($10^6 m^3$)	**($10^6 m^3$)**	**($10^6 m^3$)**	**1:20**	**1:50**	**1:100**
0.5 nMAR	23.81	6.11	29.92	12.8	11.0	9.5
0.75 nMAR	35.71	6.11	41.82	17.2	13.8	12.4
1.0 nMAR	47.61	6.11	53.72	19.1	16.4	14.6
1.5 nMAR	71.42	6.11	77.52	22.9	20.3	18.0
2.00 nMAR	95.22	6.11	101.33	26.2	22.8	20.6

For dam capacities ≥ 1.5 MAR the critical period becomes much longer, up to approximately 16 years, and the high flow EWR's would have to be supplied from the dam by releases down river, ie Scenario 2. The abridged Scenario 2 table below indicates the yield available from the dam for various dam sizes and this operating rule to satisfy the EWR's.

Scenario 2 – Foxwood Dam system with total EWR (incl. high flows) supplied by releases from storage

Reservoir capacity as a ratio of nMAR	**Live storage**	**Dead Storage**	**FSC**	**Long term yield ($10^6 m^3/a$) at Recurrence Interval**		
	($10^6 m^3$)	**($10^6 m^3$)**	**($10^6 m^3$)**	**1:20**	**1:50**	**1:100**
0.5 nMAR	23.81	6.11	29.92	9.7	7.8	6.7
0.75 nMAR	35.71	6.11	41.82	13.7	11.1	9.3
1.0 nMAR	47.61	6.11	53.72	15.9	13.3	11.3
1.5 nMAR	71.42	6.11	77.52	19.8	16.9	14.9
2.00 nMAR	95.22	6.11	101.33	22.8	19.5	17.2

These analyses indicate that the consequence of creating storage larger than approximately 1 MAR is to sacrifice net yield to the need to satisfy EWR's because water must be released from storage for this purpose. Comparison of the tables for Scenario 3 and Scenario 2 indicates that the larger dam (1.5 MAR) yields about the same as the smaller dam (1 MAR), ie just more than $19x10^6 m^3/a$.

1.3 Water Requirements

The domestic water requirements of the three towns that could potentially benefit from a water supply from the Foxwood Dam are Adelaide, Bedford and Fort Beaufort. The existing water sources available to these towns are reported to be sufficient to meet projected water needs to the year 2035 provided the water services infrastructure is well maintained and is operated effectively. The creation of additional sources can significantly improve the security of supply to Adelaide which is reliant predominantly on run-of-river diversions from the Koonap River with no significant storage.

There are no records of industrial water use in the Koonap River valley.

It is envisaged that development of the water resources of the Koonap River will stimulate the implementation of new irrigation opportunities for resource-poor farmers. Irrigable land has been shown to exist along the Koonap River and the use of this resource can make a significant contribution to the objectives of the NDP, namely to create sustainable work opportunities, eradicate poverty and reduce inequalities. An agricultural development model built a partnership between existing commercial farmers and new resource-poor farmers is envisaged. This would be real socio-economic development and would make a significant impact on rural development and agrarian reform.

Such new irrigation development would make full and effective use of the water that could be made available from a new major dam in the Koonap River, but this water would be expensive. It would be necessary for not only the cost of resource development but also the cost of establishing new irrigation farmers on viable production units to be financed by government. Significant subsidies would be necessary for both the capital investment and for operating costs would be necessary for many years.

1.4 Topographical and Geotechnical Data

Photograph 1 below shows the proposed dam site. The findings of the geotechnical investigation are reported in the Geotechnical Investigation Report which is currently under review by DWA. A summary of the findings is provided below for reference:

- The site and available construction materials are suitable either an earth embankment dam (homogenous or with clay core), a rockfill embankment dam with clay core, or for a concrete gravity dam.
- Extensive quantities of soil shell material are available but are potentially dispersive requiring gypsum stabilization.
- Shallow sandstone bedrock is expected in the left flank area which will be suitable for the location of a side spillway in the case of a rockfill or earthfill structure. However, the topography of the left flank is such that significant excavation would be required to achieve the required spillway levels for dam sizes of 1MAR and below.

1.5 Preliminary Dam Data

For context, key data for a 1 MAR dam is as follows:

- Catchment Area 1091 km^2
- Gross Mean Annual Runoff (MAR) 47.61×10^6 m^3
- Probable Maximum Flood in the order of 6000 m^3/s
- 200 year Flood Estimate 2063 m^3/s
- Full Supply Level for 1MAR storage 615 MSL
- Spillway Length – to be optimized 150 m to 250 m
- Height of Embankment 43 m
- Crest length 485 m

Photograph 1 View looking upstream, from immediately downstream of the proposed Foxwood Dam site

1.6 Dam Type and Capacity Selection

In order to select the preferred dam type and size cost estimates of four types of dam were considered, namely:

- Earthfill
- Rockfill
- Concrete Gravity, and
- Composite Gravity Spillway and Earthfill

The capacities from 0.5 MAR to 2 MAR, with a sedimentation allowance, were evaluated.

1.6.1 Cost estimates

Cost estimates were based on escalated unit rates for all major construction items from recent DWA projects. These estimates were validated against resource-based costs and benchmarked against current rates for dam construction provided by a contractor. These rates were applied in the bills of quantities for each combination of size and type of dam.

Table 1 below provides a summary of the estimated dam construction costs.

DAM OPTIONS COSTS				
	0.5 MAR	**1.0 MAR**	**1.5 MAR**	**2.0 MAR**
Earthfill	R 1,100,609,905	R 1,065,266,128	R 1,032,278,740	R 997,119,004
Rockfill	R 1,182,934,223	R 1,157,190,750	R 1,128,368,899	R 1,093,577,285
Gravity Concrete	R 754,079,833	R 942,822,832	R 1,090,354,742	R 1,213,159,821
Composite	R 751,689,283	R 903,883,873	R 1,030,187,388	R 1,140,320,471

Table 1 Summary of estimated dam construction costs

Figure 1 below illustrates the cost breakdown by major BoQ item for the 1MAR dam options.

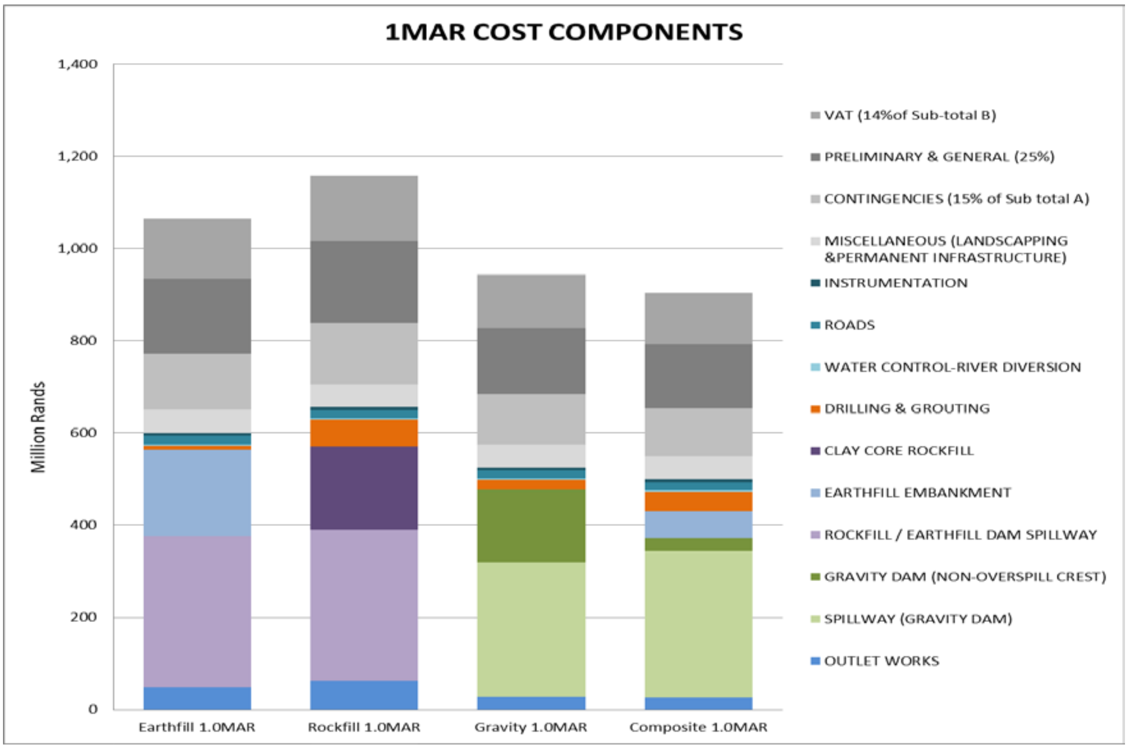


Figure 1 Cost Breakdown for 1 MAR dam options

1.6.2 Unit Reference Value

Using the DWA URV method for comparing projects over the project planning period (45 years) was followed. It is noted that varying the social discount rate does not impact on the outcome of the comparison of the different dam types for the same dam size. The calculated URVs for an 8% social discount rate are shown in the graph below.

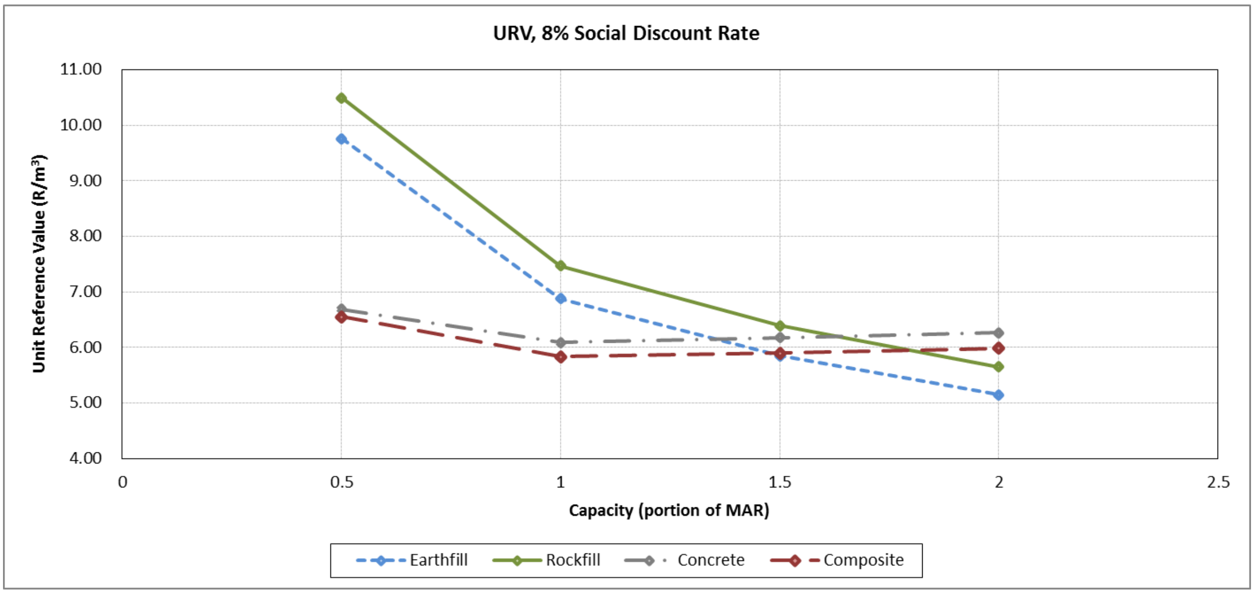


Figure 2 Unit Reference Value trends for 8% social discount rate

1.7 Conclusion and Recommendation

The URVs demonstrate that the construction costs of the earthfill and rockfill dams for sizes less than 1 MAR are very much warped by the huge cost of spillway excavations. Gravity dams are more cost effective on the basis of URVs up to 1.5MAR storage.

1.7.1 Dam size

It is recommended that a **1 MAR dam** is developed at the Foxwood Dam site:

- Impounding the Koonap River with a larger dam would impact on the natural ecological system of the river valley and would likely create complications in terms of obtaining environmental authorisation for the project.
- The analysis indicates that the available yields from a new dam are approximately equivalent for 1 MAR storage and 1.5 MAR storage due to releases from dams with larger storage capacities being needed to supply high flow EWR's (1MAR yield of 19.1 m^3/a vs 1.5MAR yield of 19.8 $x10^6 m^3/a$.).
- Providing for the Reserve from natural spillages reduces opportunity for human error.
- Storage capacities larger than 1 MAR at Foxwood would prejudice further water resource development elsewhere in the catchment.
- It is very unlikely that there will be sufficient domestic or industrial water demand in a regional context to make full use of the yield of dam larger than 1MAR.

- Providing for the development of a 1250 ha irrigation scheme on irrigable land located on various properties, now in successful production by established commercial farmers, will be a very significant development and will provide the basis for other similar schemes.
- Since rural development, irrigated agriculture and agrarian reform are competencies located in other government departments, their participation in implementation of the envisaged scheme to provide opportunities for new farmers to enter this sector is imperative. These departments have been consulted in formulating the development proposals and they have participated in deliberations of the Project Steering Committee. No institutional models, with supporting financial arrangements, have so far been put forward as a basis for implementing the irrigation scheme as a government initiative.

1.7.2 Dam type

A 1 MAR **Composite Gravity Dam with Earthfill Embankment on the right flank** is recommended for development at the Foxwood Dam site with the following motivation:

- Lowest URV among the four options for a 1 MAR dam.
- The spillway energy dissipation is more complicated for a side-channel spillway option, with significant changes of direction and the discharge of water into the river.
- No long term maintenance of a deep spillway excavation cut.
- Reduces the risks of material selection which include some elements of dispersive materials.
- The PMF and RDF design floods are best catered for with a concrete gravity dam although preliminary estimates indicate that the PMF flood will predominate for the composite option.
- Outlet works are incorporated within the gravity structure to an elevation suitable for effective discharge into the river bed. The other options require free standing towers and tunnels at founding depths similar to the cut off foundation.