REPORT NUMBER: 20/2/K602-02/G/4/22/1

DAM SAFETY EVALUATION REPORT:

ROODEFONTEIN

FOURTH REPORT

CARRIED OUT IN TERMS OF
GOVERNMENT NOTICE R 139 OF 24 FEBRUARY 2012

MARCH 2020

IBOM: Dam Safety Surveillance	**DEPARTMENT OF WATER & SANITATION** Private Bag X313, Pretoria, 0001

REPORT NUMBER. 20/2/ K602-02/D/1/22/4

DEPARTMENT OF WATER AND SANITATION

NATIONAL WATER RESOURCES INFRASTRUCTURE

DIRECTORATE: STRATEGIC ASSET MANAGEMENT

ROODEFONTEIN DAM

REPORT ON FOURTH DAM SAFETY EVALUATION

(Carried out in terms of Government Notice R139 of 24 Feb 2012)

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3. The Dam Safety Office for providing support and guidance during this evaluation.
4. Members of sub-directorate: Dam Safety Surveillance for their input in this report.

EXECUTIVE SUMMARY

Roodefontein Dam is situated on Piesang River, about 2 km east of Plettenberg Bay Town in Western Cape Province, Republic of South Africa. The dam can be located in block 3423AB in the geographic grid system.

Roodefontein Dam was design by Ninham Shand Consulting Engineers and constructed by Herbst Broers Civil Contractors and was completed in 1989. In 1995, the grassed cascade downstream of the concrete spillway was replaced with concrete spillway chute. The full supply level was raised by 2 m in 2004 and the NOC was raised by 1.3 m. Furthermore, the outlet works were modified by adding a multi-level draw-off pipe stack in 2000.

The dam comprises of a 315 m long zoned earthfill embankment with a 3.91 m width and a 19.5 m height. The upstream slope has a 1:3 (V: H) gradient and is protected by a 500 mm layer of rip-rap. The downstream slope has 1:2 (V:H) gradient and is protected by a grass layer.

The dam has a 40 m long reinforced concrete ogee spillway situated on the left flank. The spillway is 3.72 m high and on the downstream side, there is a concrete lined stilling basin with under-drains. The side slopes of the reinforced concrete stilling basin are 1:2 and the total with is 35.44 m with a total length of 15.01 m. The spillway has 3.7 m freeboard and a total capacity of 620.6 m^3/s. The spillway is able to pass the recommended design flood and the safety evaluation floods without overtopping the non-overspill crest.

The outlet works consist of a single 700 mm diameter pipe, situated on the right flank of the dam. The 700 mm pipe splits into a 500 Ø mm and 300 Ø mm pipes. The upstream side of the 700 mm dia. pipe is controlled by a plate flap valve while the downstream is controlled by a 500 Ø mm and 300 Ø mm gate valves. The maximum discharge rate of the outlet works is 2.1 m^3/s. It will take 16 days to lower the reservoir level from full supply level to the lowest invert level.

The Dam is founded on a succession of alluvial material, comprising clayey to sandy silt material with gravel layers underlined by stiff clay and silt derived from weathering of the underlying Kirkwood Formation mudrock strata. The spillway is founded on a very dense variably of grey, yellow, and orange silty-sand residual material derived from the in-situ weathered Kirkwood sandstone strata inter-beded with residual silty-clay. On the downstream slope of the embankment, sub-parallel horizontal cracks near the crest were observed. North of the spillway, there is a longitudinal crack at the break in slope. Either than this, the geology of the site is satisfactory.

The presence of the dam puts between 700 to 1 500 people at risk should the dam failures. It is estimated that between 15 and 30 people will lose their lives in the unfortunate event of dam failure. Furthermore, it is estimated that failure will results in direct economic cost ranging from R 458 million to R 533 million.

Mechanical and electrical components were inspected on 11 February 2020. The inspection revealed that conditions of outlet works are reasonable and the outlet works are functioning satisfactorily. However, the required maintenance work was not performed due to a lack of maintenance contractor.

A physical dam safety inspection took place on 25 September 2019. In general, the physical conditions of Roodefontein Dam are satisfactorily. However, the evaluation revealed that certain components of the dam do not meet minimum requirements of Dam Safety Regulations, Regulation 139 of 24 February 2012 and therefore the following is recommended to remedy the situation:

a) All recommendations from mechanical report should be implement as indicated in the report.
b) Complete outstanding recommendations from the previous evaluations
c) Formalise a contractual arrangement between the Owner (Department of Water and Sanitation) and the Operator of the Dam (Bitou Municipality) to allow for efficient operation of the dam.
d) Increase Freeboard of the dam to ensure compliance with 2011 Guidelines on Freeboard for Dams.
e) Repair the damaged roof of the outlet house.
f) Install a proper toe drain system to manage seepage on the downstream of the dam.
g) Install a lined - channel on the toe of the dam to collect run-off water from the toe of the dam and discharge in the river.
h) Install guardrails on the stairway leading to the gauge plate to improve safety of personnel working at the dam.
i) Request owners of the property immediate downstream of the dam to re-route the newly installed surface drain away from the dam to avoid saturating the toe of the dam which might trigger slip failure of the earthfill embankment.
j) Department should determine an appropriate action concerning the newly built horse stable downstream of the earthfill embankment.
k) Rehabilitate the earthfill embankment in order to address the horizontal crack on the embankment and the slope instability.
l) Monitor survey beacon F13 on the downstream left side of the spillway channel for a possible movement of the support material.
m) Install a safety boom upstream of the spillway
n) Provide a Civil Logbook
o) Provide training to the Operator of the dam

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LIST OF ABBREVIATIONS

ABBREVIATION	DEFINITION
APP	Approved Professional Person
DO	Dam Operator
BM	Bitou Municipality
D/S	Down Stream
DSS	Dam Safety Surveillance
DSI	Dam Safety Inspection
DSO	Dam Safety Office
DSE	Dam Safety Evaluation
DSER	Dam Safety Evaluation Report
DWS	Department of Water and Sanitation
EQ	Earth Quake
EPP	Emergency Preparedness Plan
FSL	Full Supply Level
FSC	Full Supply Capacity
LDDL	Lowest Draw Down Level
LFL	Lowest Foundation Level
MAP	Mean Annual Precipitation
MAR	Mean Annual Rainfall
NOC	Non-Overspill Crest
PE	Probability of Exceedance
$Q_{1:200}$	Flood Peak with probability of exceedance of 1 in 200
RBL	River Bed Level
RMF	Regional Rainfall Level
SEF	Safety Evaluation Flood
U/S	Upstream
WCO	Water Control Officer
Yrs.	Years
LF	Left flank
RF	Right flank

1. INTRODUCTION

The fourth dam safety evaluation of Roodefontein Dam conducted in order to comply with the prescripts of the Dam Safety Regulations, Notice R 139 of 24 February 2012.

The national government of the Republic of South Africa formulated Dam Safety Regulations to standardise the construction, monitoring and operation of dams in South Africa. These regulations aim at ensuring that the risk posed to the population and the environment is mitigated and managed. The formation of these Dam Safety Regulations is empowered by Section 123 (1) of the National Water Act: Act No. 36 of 1998.

The regulations compel all owners/operators of dams with a safety risk to conduct routine and major dam safety evaluations every 5 years and submit a report thereof, to the minister of Water and Sanitation.

This report presents the results of the fourth dam safety evaluation of Roodefontein Dam in partial fulfilment of the government notice R 139 of 24 February 2012.

The main objectives of this report are as follows:

- Discuss the state of conditions at Roodefontein Dam.
- Present the results of the performed evaluation of the dam.
- Present and discus the findings of the conducted physical inspection of the dam.
- Provide recommendations to address the identified shortcomings at the dam.

This report covers the fourth dam safety evaluation of Roodefontein Dam only and the evaluation report should be submitted to Dam Safety Office (DSO) no later than 31 March 2020.

The first part of this report focuses on the background and description of Roodefontein Dam. The second part discusses the hydrology, hydraulics, structural analysis, and risk analysis of the dam. The third part discusses the findings emanating from a physical inspection of the dam conducted on 25 September 2019. Thereafter, conclusion is drawn and recommendations presented.

2. LOCALITY OF THE DAM

Roodefontein Dam is situated on Piesang River, about 2 km east of Plettenberg Bay Town in Western Cape Province, Republic of South Africa. The dam can be located in block 3423AB in the geographic grid system. Figure 1 shows the locality map of the dam and Table 1 gives a summary of the locality data of the dam. Enlarged locality map is attached in Addendum A.

Table 1: Locality data of Roodefontein Dam.

LOCALITY INFORMATION OF ROODEFONTEIN DAM	
Name of Dam	Roodefontein
Locality Number	K602-02
River	Piesang River
Nearest Town	Plettenberg Bay
Distance to nearest Town	2 km
Province	Western Cape
Latitude	34°4'0.141" S
Longitude	23°20'6.004" E

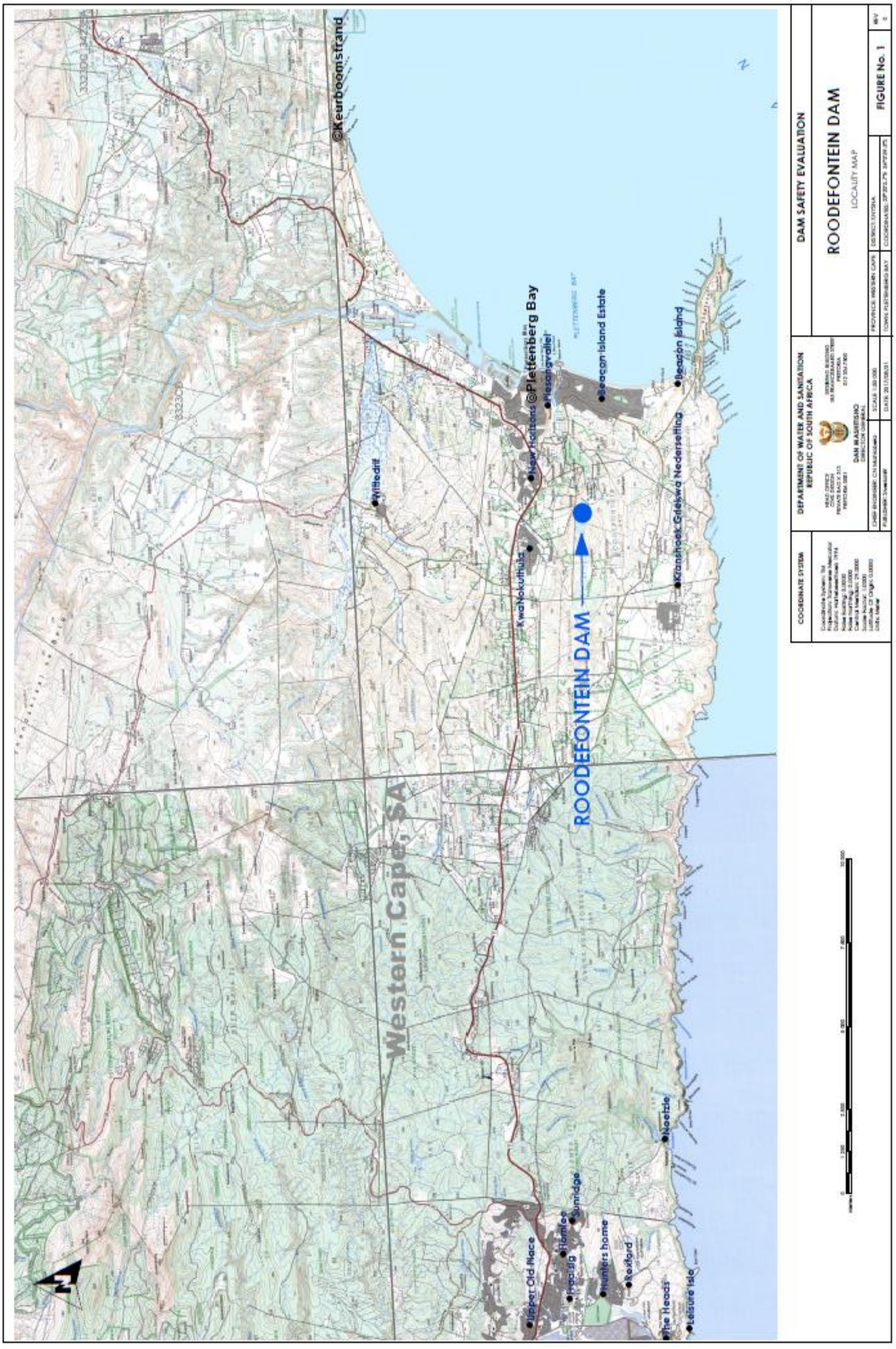

Figure 1: Geographic Map showing the locality of Roodefontein Dam

3. DESCRIPTION OF THE DAM

3.1. Historic Background of the Dam

Roodefontein Dam was originally design by Ninham Shand Consulting Engineers and constructed by Herbst Broers Civil Contractors, construction completed in 1989. In 1995, the grassed cascade downstream of the concrete spillway was replaced with concrete spillway chute. Hattingh reported that the full supply level was raised by 2 m in 2004 and the NOC was raised by 1.3 m. Furthermore, the outlet works were modified by adding a muilt level draw-off pipe stack in 2000 (Hattingh, 2011:1). The historic data of this dam is summarised in Table 2.

Table 2: Historic data of Roodefontein Dam

HISTORIC DAM OF THE DAM			
Item	**Description**	**Designer**	**Year**
Original Dam structure	Contractor: Herbst Broers (Pty)	Ninham Shand (Cape) Inc.	1989
Betterment No.1	Provision of concrete chute.	Ninham Shand (Cape) Inc.	1995
Betterment No.2	Installation of Muilti-level outlet works	Stewart Scott inc.	2000
Betterment No.3	Raising of NOC and FSL	Stewart Scott inc.	2003/4

3.2. Ownership and Classification of the dam

Roodefontein Dam is owned by the Department of Water and Sanitation and operated by Bitou Municipality. Water from the dam is used primarily for irrigation, industrial and domestic consumption (Hattingh, 2009:1). The dam has been classified as a category III. Table 3 below gives a summary of the ownership and classification information of the dam.

Table 3: Ownership and classification of Roodefontein Dam

CLASSIFICATION INFORMATION	
Owner	Dept. of Water and Sanitation
Operator	Bitou and Plettenberg Bay Municipality
Classification Category	III
Hazard Rating	High
Size	Medium
Registration date	29 January 1990
Classification date	2 September 1987
Date of completion	1995

3.3. Description of the Embankment

The dam comprises of a zoned earthfill embankment which is 315 m long, 3.91 m wide at the crest level and 19.5 m high when measured from the lowest river bed level (RL 26.5 m) to the NOC (RL 46.0 m). The upstream slope of the dam has a 1:3 (V: H) gradient and is protected by a layer of rip-rap against waves action and erosion. The downstream side of the embankment has 1:2 (V:H) gradient and it is protected by vegetation cover, see Figure 2. The statistics of the embankment is provided in Table 4.

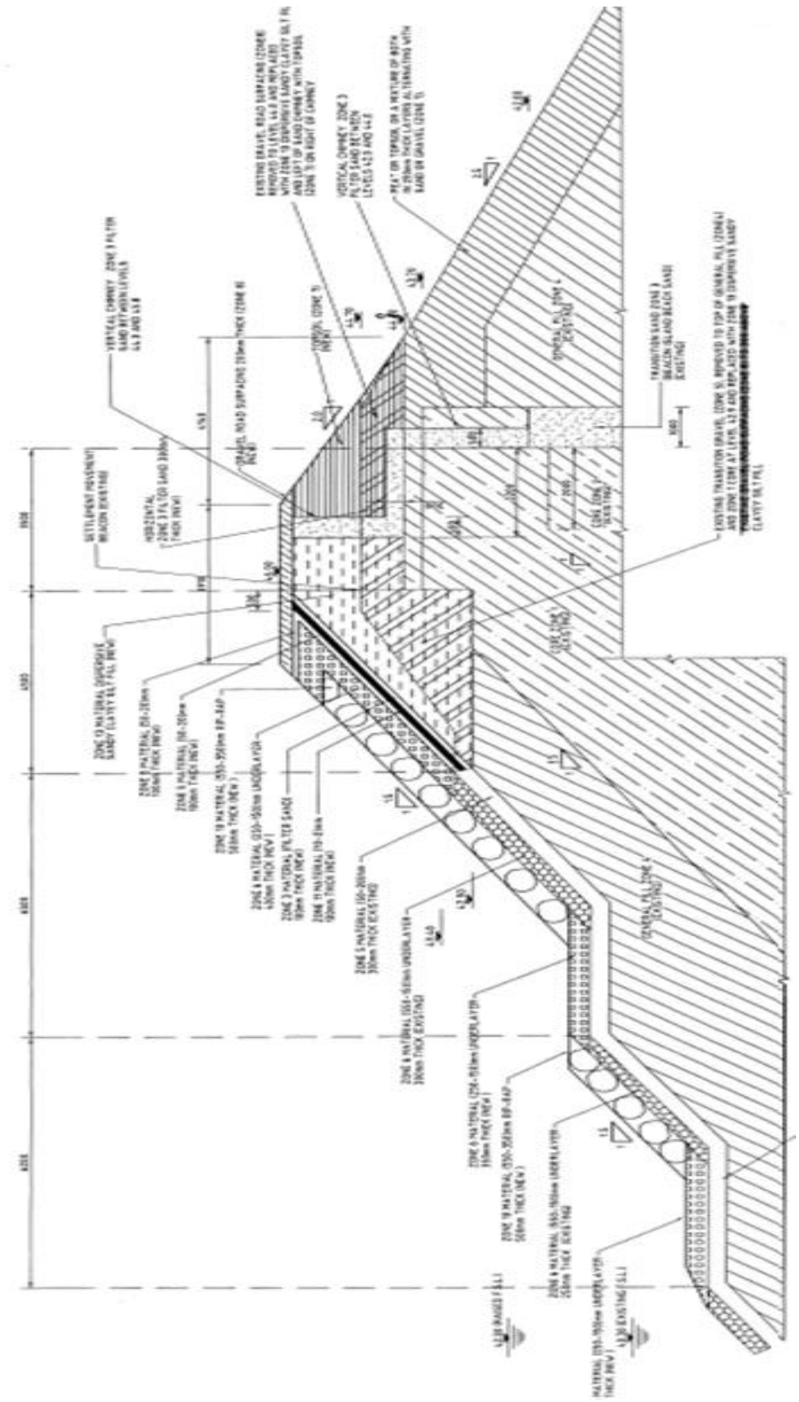

Figure 2: Typical cross-section of Roodefontein Dam – Embankment (Drw No. 150195/06)

Table 4: Statistics of the embankment

STATISTICS OF THE EMBANKMENT	
Wall type	Zoned Earthfill
Crest length	315 m
Crest width	3.91 m
Erosion protection for the crest	Gravel
Downstream slope (V:H)	1:2 and 1:2.5
Downstream slope protection	Grass
Upstream slope (V:H)	1:3
Upstream slope protection	Rip-Rap
Wall height (LFL to NOC)	19.5 m
Wall height (RBL to NOC)	18 m
Full Supply Level (FSL)	RL 42.3 m
Non-Overspill Crest (NOC) level	RL 46.0 m
Gauge Plate Reading at FSL	27.01 m
Freeboard	3.7 m

3.4. Details of the Spillway

The dam has a 40 m long reinforced concrete ogee side channel spillway situated on the left flank. The spillway is 3.72 m high when measured from the river channel, see Figure 3.

On the downstream side, the spillway has a concrete lined stilling basin with under-drains. The side slopes of the reinforced concrete stilling basin are 1:2 (Vertical: Horizontal) and the total with is 35.44 m with a total length of 15.01 m. The end part of the stilling basin has a 1.6 m wide sloped concrete upstand (Watermeyer, 2004). Details of the spillway are summarised in Table 5.

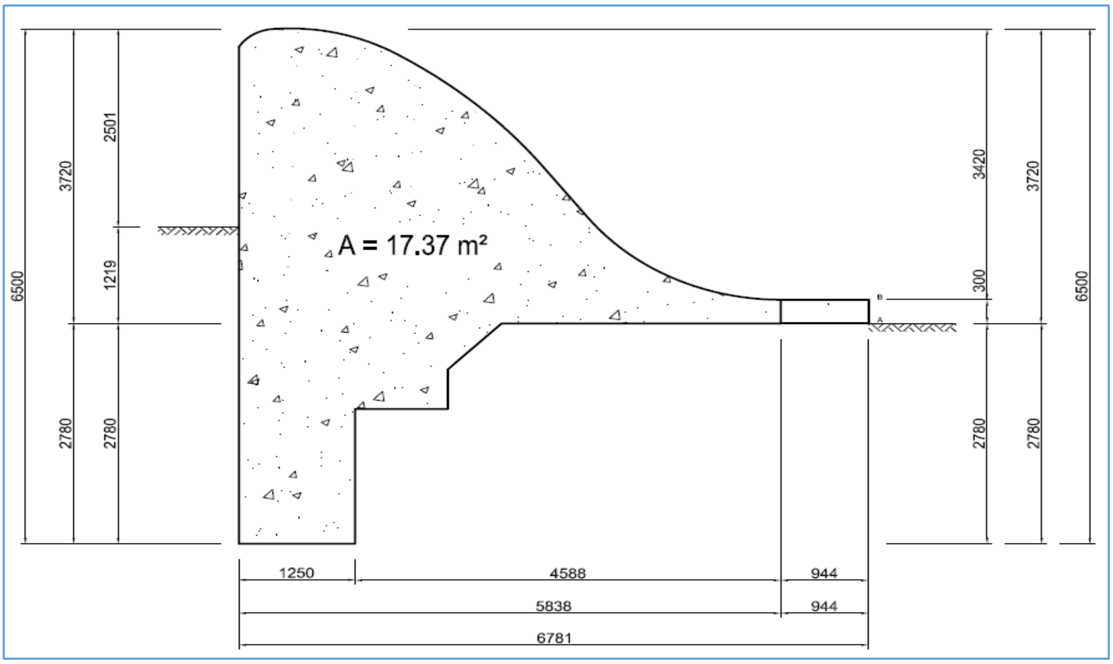

Figure 3: Cross-section of the Ogee spillway

Table 5: Details of the Spillway

STATISTICS OF THE SPILLWAY	
Spillway Type	Concrete Ogee Spillway
Crest level (FSL)	42.3 m
Total Spillway width	40 m
Height of NOC above Spillway	3.7 m
Level of Gauge Plate Zero	33.84 m
Energy Dissipation Structures	Concrete chute

3.5. Details of the Outlet Works

The outlet works consist of a single 700 mm diameter pipe, situated on the right flank of the dam. The 700 mm pipe splits into a 500 Ø mm and 300 Ø mm pipes. The upstream side of the 700 mm dia. pipe is controlled by a plate flap valve while the downstream is controlled by a 500 Ø mm and 300 Ø mm gate valves. Details of the outlet works are summarised in Table 6. Detail layout of the outlet works is attached in Addendum B.

Table 6: details of the Outlet Works (Hattingh, 2011)

STATISTICS OF THE OUTLET WORKS	
Invert level	RL 32 m
Number of outlet pipes	1
Diameter	700 Ø mm
Type of Outlet valve - Upstream	Plate flap Valve
Type of Outlet valve - Downstream	Butterfly and Sleeve Valves
Size of Outlet valve	500 mm and 300 mm
Reported Discharge Capacity at FSL	1.2 m^3/s
Reported Time to lower reservoir from FSL to Lowest Draw Down Level	24 days

3.6. Storage Capacity

The storage capacity data of Roodefontein Dam is summarised in Table 7 below. The gross volume of the dam is 2.063 million m^3 and if the inactive storage below the invert level of the outlet works in neglected, the net storage of the dam is 2.003 million m^3.

Table 7: Storage capacity of Roodefontein Dam

STORAGE CAPACITY	
Gross storage capacity at FSL - V_{gross}	2.063 x 10^6 m^3
Inactive storage below Invert level of Outlet works - $V_{inactive}$	60 000 m^3
Net Storage Capacity at FSL - V_{net}	2.003 x 10^6 m^3
Storage Capacity at NOC (RL_{noc} = 46 m) - V_{noc}	3 624 x 10^6 m^3
Water Surface Area at FSL - A_{FSL}	37.1 ha
Water Surface Area at RL 40 m – A_{40m}	28.8 ha
Water Surface Area at NOC - A_{NOC}	50.5 ha
Length of Resevoir at FSL - L_{FSL}	1.25 km

4. AVAILABLE INFORMATION

The authors evaluated the following reports as part of the fourth dam safety evaluation.

4.1. Design Reports

A list of design reports evaluated by the authors during this evaluation is provided in Table 8.

Table 8: List of available design reports

DESIGN REPORTS		
TITLE	**AUTHOR**	**YEAR**
1. Raising the FSL by 2 m by means of solid raising	Watermeyer CF	2002
2. Evaluation of dam raising option: Addendum	Pellsn HNF	2002
3. Evaluation of dam raising option, Revision 1	Pellsn HNF	2001
4. Rehabilitation of Spillway. Report No. 2214/4601	Pellsn HNF	1994
5. Rehabilitation of Spillway: Supplementary Report No. 2214A/4601	Pellsn HNF	1994

4.2. Construction Reports

A list of construction reports evaluated by the authors during this evaluation is provided in Table 9.

Table 9: List of available construction reports

CONSTRUCTION REPORTS		
TITLE	**AUTHOR**	**YEAR**
1. Construction Completion Report	Erwee H	2006
2. Raising the Spillway & Embankment of Roodefontein Dam	Watermeyer CF	2004
3. Construction Completion Report. Report No. 2465/4601	Pellsn HNF	1996
4. Construction Completion Report. Report No. 1628/4601	Powrie WE	1990

4.3. Geological Reports

A list of Geological reports evaluated by the authors during this evaluation is provided in Table 10.

Table 10: List of available Geological Reports

GEOLOGICAL/GEOTECHNICAL REPORTS		
TITLE	**AUTHOR**	**YEAR**
1. Engineering Geological Report for Dam Safety Purposes	Davis GN	2006
2. Rehabilitation of Spillway, Final Geology Report.	Van Der Merwer WJ	1996
3. The effect of an Earthquake on the dam	Ninham Shand	1996
4. Dam Safety Inspection Geological Report	Knight Hall Hendry	1993
5. Preliminary Report on Geological Investigations	Van Der Merwer WJ	1987

4.4. Hydrological Reports

A list of hydrological studies reports evaluated by the authors during this evaluation is provided in Table 11.

Table 11: List of available hydrological studies reports

HYDROLOGICAL REPORTS		
TITLE	**AUTHOR**	**YEAR**
1. Flood Frequency Analysis: Roodefontein Dam	Rademeyer P	2018
2. Flood Frequency Analysis: Roodefontein Dam	Roux M	2010
3. Flood Frequency Analysis: Roodefontein Dam	Tsehla MS	2001

4.5. Operation And Maintenance (O & M) Manual

A list of operations and maintenance (O&M) manuals evaluated by the authors during this evolution is provided in Table ***12***.

Table 12: List of available operations and maintenance manuals

O & M MANUALS		
TITLE	**AUTHOR**	**YEAR**
1. Operation, Monitoring & Maintenance Manual: Vol. 2	Watermeyer	2004
2. Operation & Maintenance Manual.	Ninham Shand	1990
3. Operation & Maintenance Manual.	Ninham Shand	1998

4.6. Dam safety evaluation(DSE) reports

A list of Dam Safety Evaluation reports evaluated by the author is provided in Table 13 below. The reports were initially called Dam Safety Inspection (DSI) Reports.

Table 13: List of available Dam Safety Evaluation Reports

DAM SAFETY EVALUATION REPORTS		
TITLE	**AUTHOR**	**YEAR**
1. Third Dam Safety Inspection Report	Hattingh LC	2011
2. Second Dam Safety Inspection Report	Jan Brink	2006
3. First Dam Safety Inspection Report	Watermeyer CF	2002

4.7. Emergency Preparedness Plan (EPP)

The EPP evaluated by the author are provided in Table 14 below.

Table 14: List of available Emergency Preparedness Plans

EMERGENCY PREPAREDNESS PLAN		
TITLE	**AUTHOR**	**YEAR**
1. Operations and Maintenance Manual & Emergency Preparedness Plan	Watermeyer CF	2005
2. Roodefontein Dam: Emergency Preparedness Plan	Weidemann EW & Horn H	2015

4.8. As-Built Drawings/Plans

A list of select few As-built drawings of the dam is provided in Table 15 below and copies of these drawings have been attached in Addendum B.

Table 15: List of selected and available As-built drawings

AS-BUILT DRAWINGS/PLANS	
DESCRIPTION	**Number**
1. Spillway Raising - Sections & Details	150193/06
2. Alternative Embankment Raising	150195/06
3. Embankment Section & Downstream Valve Chamber Details	150197/06
4. Embankment Survey	150207/06
5. Details of stilling basin	150199/06
6. Outlet pipe details	150201/06

5. GEOLOGY OF THE DAM SITE

The following geological data was extracted from the engineering geological report compiled by Davis in 20116 for dam safety purposes. The second source of the geological information used is the final geotechnical report conducted during the rehabilitation of the spillway in 1996. This report summarises the geology of the dam and its region, detailed information is contained in the above mentioned sources, refer to addendum C.

5.1. Regional Geology

The regional geology consists of sandstone and conglomerate of the Enon formation in the Uitenhage group as can be seen in Figure 4. It is reported that sedimentary strata of the Cape Super-group form part of the geology of this area.

5.2. Site Geology and Foundation Conditions

Davis, 2006 reports that the embankment of Roodefontein Dam is founded on a succession of alluvial material, comprising clayey to sandy silt material with gravel layers underlined by stiff clay and silt derived from weathering of the underlying Kirkwood Formation mudrock strata.

The spillway is founded on a very dense variably of grey, yellow, and orange silty-sand residual material derived from the in-situ weathered Kirkwood sandstone strata interbeded with residual silty-clay.

During construction major slope instability were witnessed on the northern slope flanking the spillway excavation. It is reported that this slope instability is expected to continue since a detail stability analysis revealed that the slope have a safety factor of unity (1).

5.3. Seismicity

Roodefontein Dam is located in a region with low natural earthquake hazard. The dam is in an area with a predicted horizontal acceleration of 0.05 g with a 10% probability of being exceeded in a 1:50 year return period.

5.4. Evaluation of the Geology

On the downstream slope of the embankment, Davis noted with concern sub-parallel horizontal cracks near the crest. North of the spillway, there is a longitudinal crack at the break in slope. Either than this, the geology of the site is satisfactory. These findings have been adopted in this report since no new findings were made during this fifth dam safety evaluation.

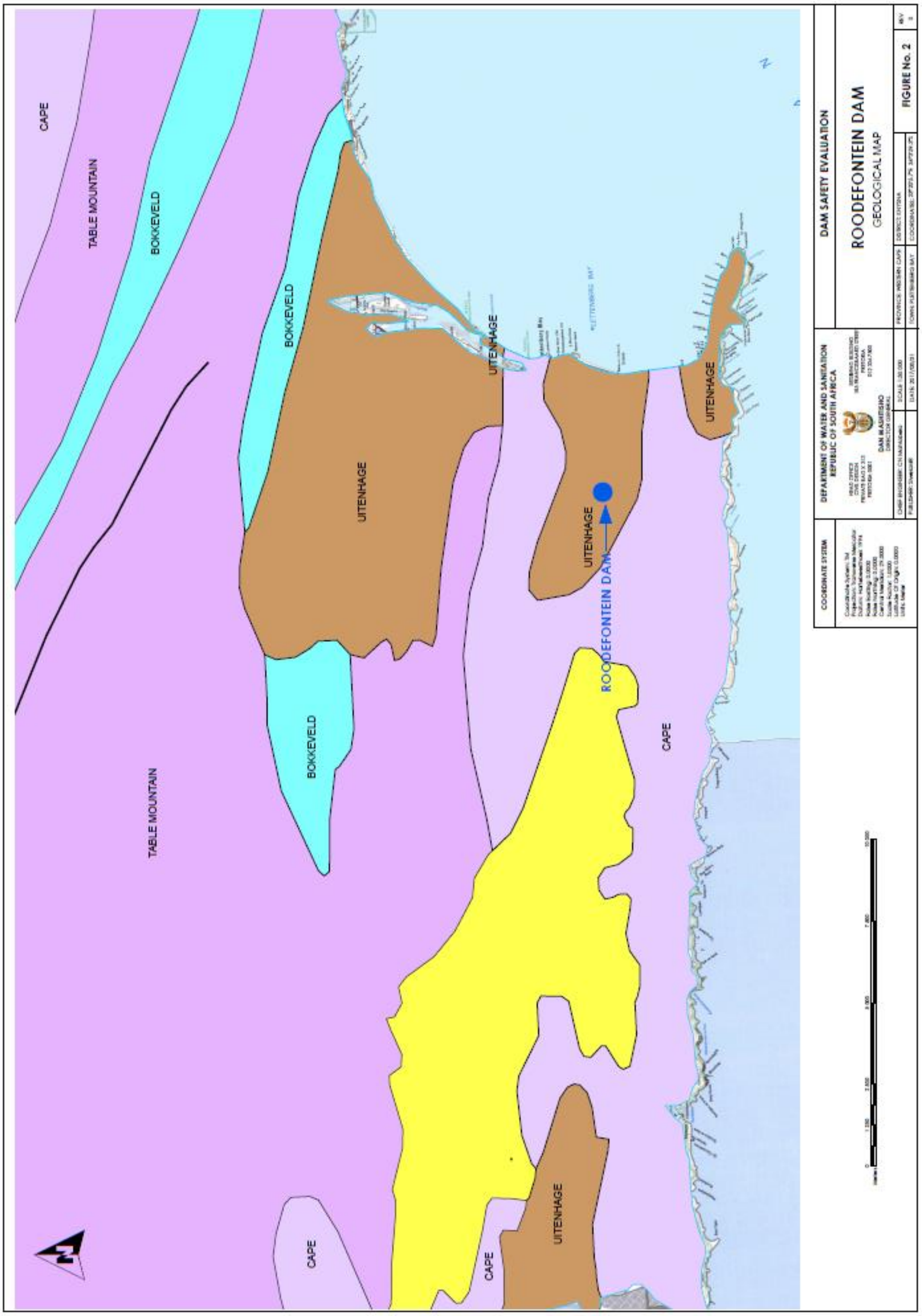

Figure 4: Regional geology of areas around Roodefontein Dam.

6. HYDROLOGY

The hydrology information was extracted from the flood frequency analysis report of February 2018 compiled by Rademeyer under the supervision of Van der Spuy (refer to addendum D). The following sub-sections cover different aspects of the hydrology of the dam.

6.1. Catchment Characteristics

The dam is located on Piesang River in Francou-Rodier region 5.2 and with 3.5 hours as a total time of concentration. The catchment for Roodefontein Dam is 28 km^2 in size and has a mean steepness of 16%. The longest water course is 14.8 km in length with a mean slope of 0.011683. The Mean Annual Precipitation (MAP) is 930 mm. Key characteristics of the catchment are listed in Table 16 below.

Table 16: Catchment Characteristics of Roodefontein Dam.

CATCHMENT CHARACTERISTICS						
A (km^2)	L (km)	L_c (km)	T_c (h)	S_L (m/m)	S_A (%)	MAP (mm)
28	14.8	8.25	3.5	0.011683	16	930

6.2. Flood Peaks

The recommended design flood (RDF: Q_{RDF}) which is $Q_{1:200}$ for this category III dam has a peak flow of 95 m^3/s and the Safety Evaluation Flood (SEF) is 530 m^3/s. The flood frequency analysis report recommends that the Regional Maximum Flood (RMF) should be treated as the SEF and this recommendation is in-line with the 2011 SANCOLD guidelines. A list of recommended flood peaks is provided in Table 17.

Table 17: Recommended flood peaks from the flood frequency report (Rademeyer, 2018: i)

Probability of expediency (%)						
20	10	5	2	1	0.5	0.01
Return Period (Years)						
1:5	1:10	1:20	1:50	1:100	1:200	1:10 000
Flood Peak (m^3/s)						
20	30	45	65	80	95	530

6.3. Selection of RDF and SEF

The 2011 SANCOLD guidelines on Freeboard state that for a medium size dam (12 m ≤ Dam height ≤ 30 m) the Recommended Design Discharge (RDD) should be based on flood peak with 1:200 return period. Therefore, the recommended design flood for Roodefontein Dam is $Q_{1:200}$ (95 m^3/s).

The same guidelines further recommend that the Regional Maximum Flood (RMF) should be considered as a Safety Evaluation Flood (SEF). In the case of Roodefontein Dam, the SEF is 530 m^3/s.

6.4. Hydrographs

The 2018 flood frequency report recommends that the observed hydrograph should be used for flood routing purposes. The author of this report selected the August 2006 flood which peaked at 34.4 m^3/s. This hydrograph is shown in Figure 5 below.

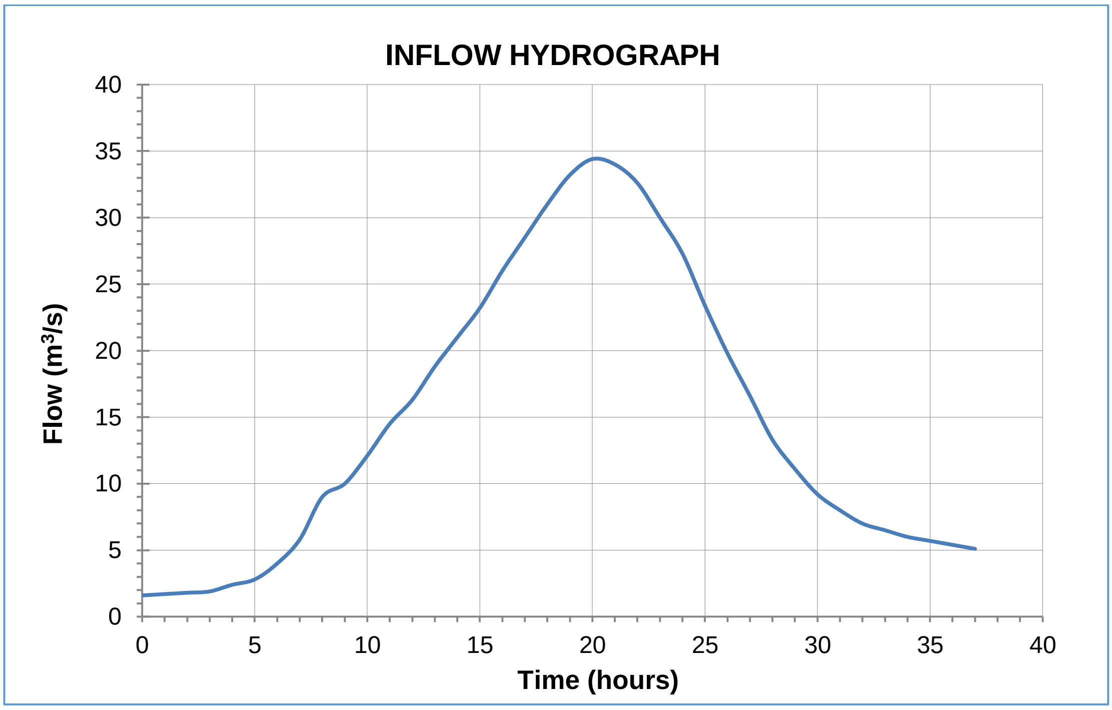

Figure 5: The August 2006 observed inflow hydrograph.

6.4.1. Routing of Recommended Design Flood (RDF)

The observed inflow hydrograph was adjusted using the RDF peaks (Q = 95 m^3/s) in Table 17. The resultant hydrograph was then routed through the spillway using basic principles of flood routing following Newton-Ralpson's Method. The results of the RDF routing are graphical shown in Figure 6.

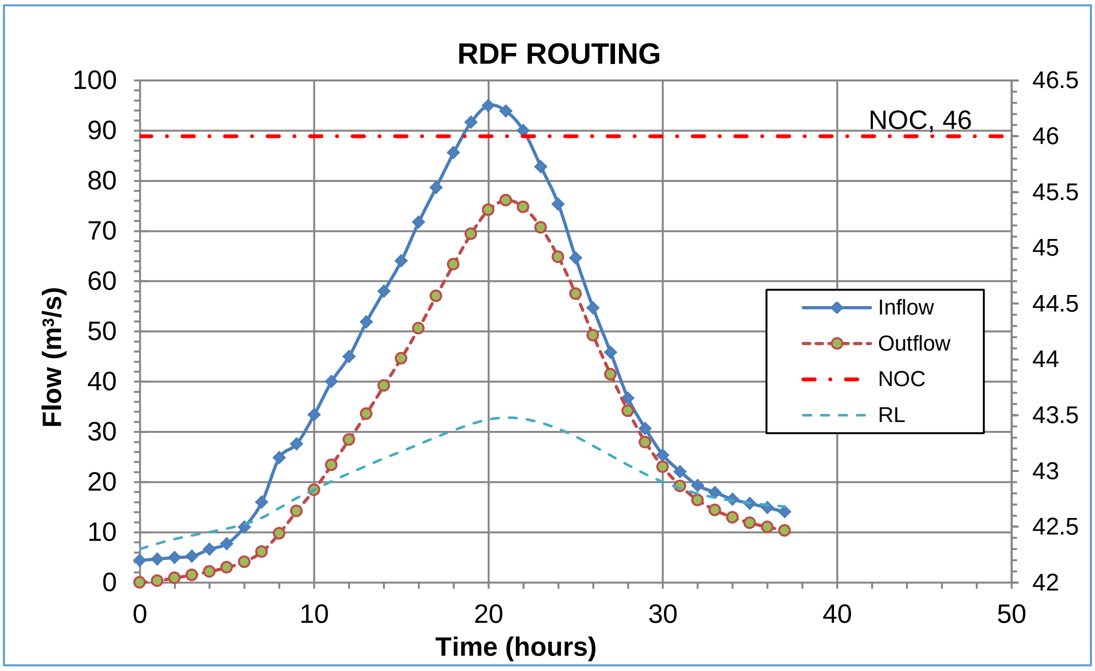

Figure 6: RDF hydrograph with reduced levels

6.4.2. Routing of Safety Evaluation Flood (SEF) routing

The observed inflow hydrograph was adjusted using the SEF value provided in Table 17 and routed through the spillway of the dam. The resulting hydrograph is shown in Figure 7 below. Refer to addendum E for detailed calculations.

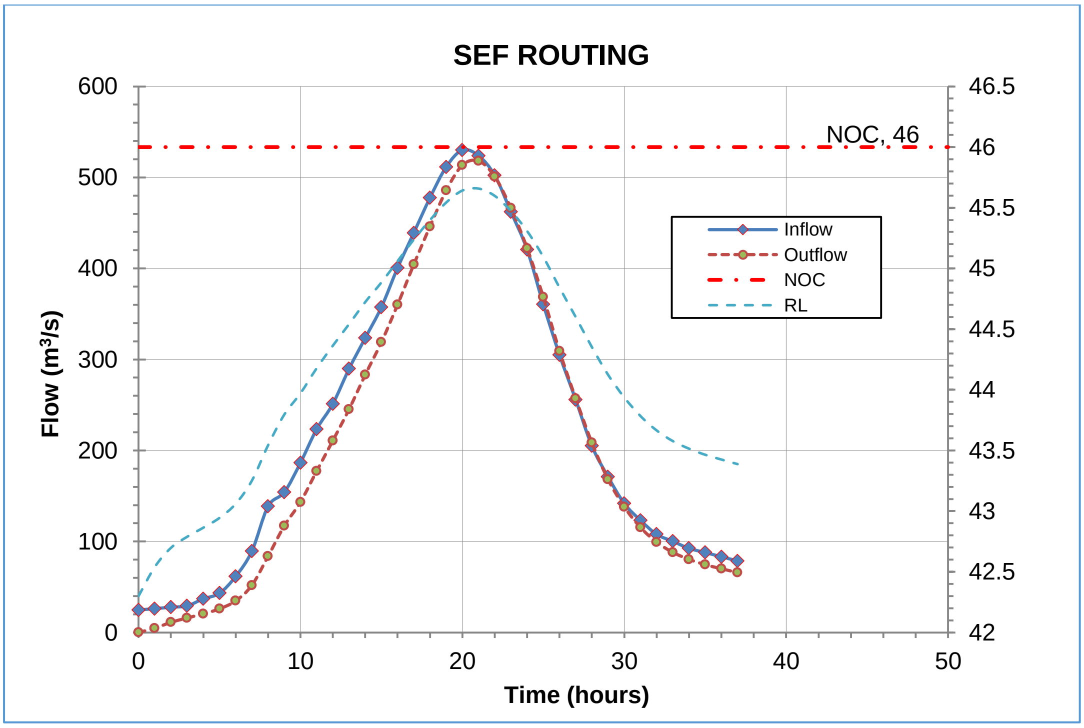

Figure 7: SEF routing hydrograph with corresponding reduced levels.

6.4.3. Results of Flood Routing

In the case of RDF routing, the NOC of the dam is not overtopped. The maximum discharge obtained is 76.1 m^3/s and this corresponds to a 1.18 m stage which has a reduced level of 43.48 m. This implies that the available freeboard under RDF conditions is 2.52 m. The attenuation in the case of RDF is 19.9% and the total translation for this case is 1 hour which is the same as that of the SEF routing.

In the case of SEF routing, the NOC is also not overtopped and the maximum discharge is 518.06 m^3/s. The corresponding stage is 3.36 m stage with a reduced level of 45.66 m above mean seal level. The available freeboard under this condition is 0.34 m, the attenuation is 2.3% and the translation is 1 hour. The routing results are summarised in Table 18 below.

Table 18: Flood routing results for RDF and SEF cases.

FLOOD ROUTING RESULTS		
PARAMETER	**RDF ROUTING**	**SEF ROUTING**
Maximum Discharge (m^3/s)	76.1	518.06
Height above Spillway Crest (m)	1.18	3.36
Available Freeboard (m)	2.52	0.34
Attenuation (%)	19.9	2.3
Translation (hours)	1	1

7. HYDRAULICS ANALYSIS

7.1. Spillway Capacity

The uncontrolled ogee spillway has a maximum discharge capacity of 620.6 m^3/s before overtopping the NOC. The quoted discharge capacity is based on using a dynamic discharge coefficient which was derived using the DWS equation as shown in Table 19. The use of a constant value of a discharge coefficient (C = 2.2) results in a total Spillway discharge capacity of 626.3 m^3/s. The discharge capacity curve based on a dynamic C is shown in Figure 8 below and it is recommended for use.

Table 19: Discharge equations used to determine the spillway discharge capacity

Discharge Capacity (Q) = $CLH^{1.5}$ ……………………………….Constant C = 2.2

Discharge Capacity (Q) = $CLH^{1.5}$ ……………………………….Dynamic $C = 0.59 + 1.59\sqrt{\frac{H_1}{H_0}}$

Where H_1 is the stage under consideration and H_0 is the total freeboard (3.7 m)

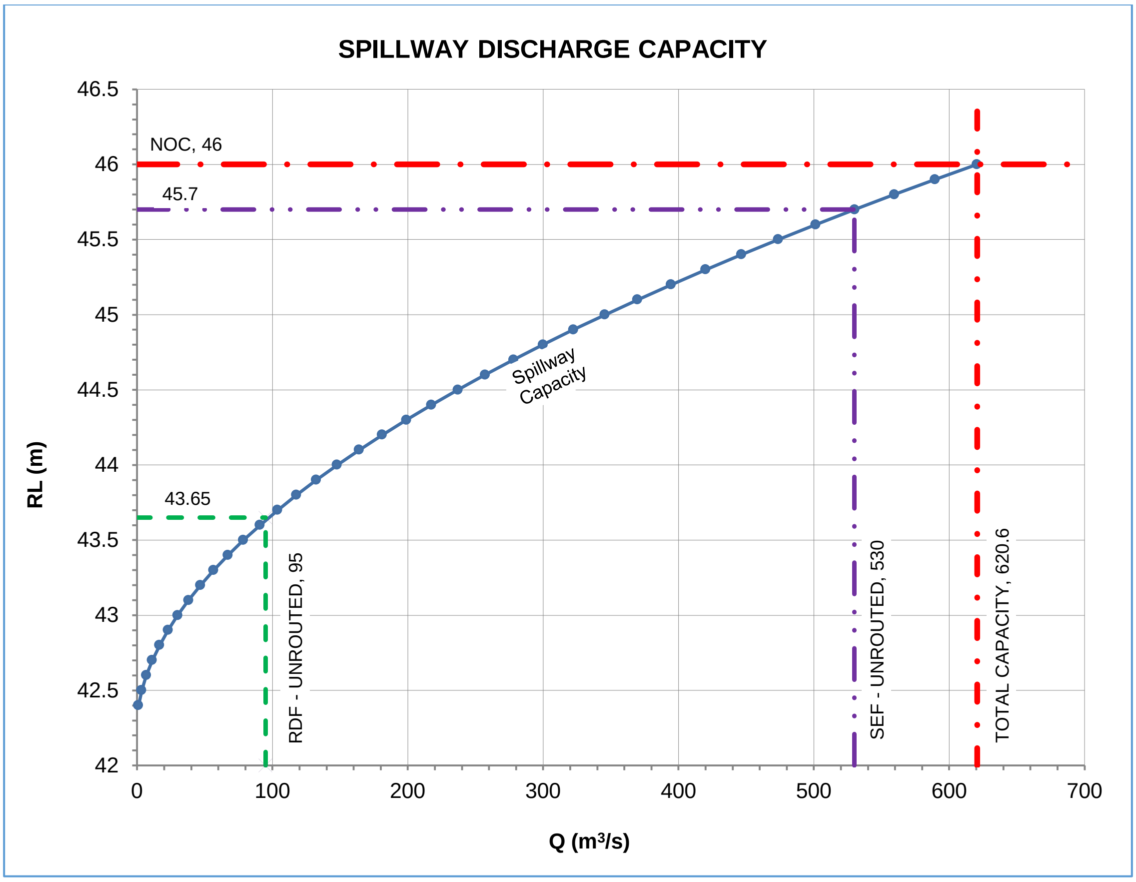

Figure 8: Discharge Capacity of the Spillway determined using a dynamic discharge coefficient

7.2. Freeboard Requirements

It was revealed in the preceding chapters that Roodefontein Dam is a category III dam with significant hazard potential. The 2011 SANCOLD guidelines on Freeboards dictate that the total freeboard of the dam should comprise of the influence of the following components; RDD surcharge, Wind wave run-up; Wind set-up; Surge and Seiches; Earthquake wave and Flood outlets.

The selected Freeboard input parameters are shown in Table 20 below. Detailed calculation of the freeboard requirements are presented in Addendum F.

Table 20: Freeboard input parameters

FREEBOARD INPUT PARAMETERS	
PARAMETER	**VALUE**
Wind speed (m/s) (Fig 2.3-4 SANCOLD)	24
Fetch (m)	1110

7.2.1. Flood surcharge

The results of RDF routing reveal that the maximum discharge has a stage of 1.18 m. The reduced level is 43.48 m above mean see level as discussed in the preceding chapters.

7.2.2. Wind Wave run-up

The wind wave run-up (R) = $H_s \times A \times \xi_p \times \gamma_r \times \gamma_b \times \gamma_h \times \gamma_\beta$ = **0.690** m. Values of the parameters A, γ_r, γ_b, γ_h and γ_β are 1.6, 1, 1, 1 and 0.85 respectively.

7.2.3. Wind set-up

The wind setup for the dam (n_w) = $0.5 \times (\rho_{\alpha\iota\rho} / \rho_\omega) \times C_D \times (U_{10})^2 / (g \times h_{ave}) \times F$ = **3.508** m.

7.2.4. Surge and Seiches

The component of surge and seiches is taken as 0 m.

7.2.5. Earthquake Wave

The earthquake component in case of Roodefontein is taken as 0 m.

7.2.6. Land slide

The surrounding slopes of the reservoir are relatively flat and therefore the probability of a land slide is negligible and therefore the freeboard component of this condition is 0 m.

7.2.7. Available Freeboard

In Table 5 it was established that the available freeboard is 3.7 m.

7.2.8. Freeboard Combinations

Freeboard combinations for category III dams are given in table 3.1 of the SANCOLD guidelines on freeboard and it has been reproduced in Table 21 of this report for the convenience of the reader. Combination number 2 and 5 are the severe cases with freeboard requirement of 5.068 m. Based on the available freeboard and the calculated value (5.1 m) it can be concluded that the dam does not have enough freeboard. An additional 1.37 m is required to ensure that the dam satisfy minimum freeboard requirements.

Table 21: Freeboard combinations

Combination number	RDD Surchage	Wind wave and run-up 100-year event	Wind set-up	Surges and seiches	Earth-quake wave	Land-slide wave	Flood outlets	TOTAL (m)
1	1.18	0.38						1.6
2	1.18	0.38	3.508	0				5.1
3					0			0.0
4						0		0.0
5	1.18	0.38	3.508	0			0	**5.1**

7.3. Drawdown Capacity

7.3.1. Available Drawdown Rate

Roodefontein Dam has 700Ø mm steel pipe which branches into a 500Ø mm and a 650Ø mm outlets. The invert level of the 700Ø mm pipe is at RL 32.0 m. The upstream side of the outlet pipe is controlled by a plate flap valve while the downstream is controlled by a butterfly valve and a sleeve valve. The outlet works statistics is summarised in Table 22, below.

Table 22: Outlet works statistics for Roodefontein Dam.

Lowest invert level	RL 32.0 m
Size of outlet pipe	700Ø mm NB encased steel pipe
Downstream control	500 NB isolating valve and 300 NB sleeve valve
Upstream control	Plate flap valve

Volume - Elevation data contained in the operation, monitoring and maintenance manual was used to estimate the outlet capacity. The orifice condition has given an average discharge capacity of 3.26 m^3/s at FSL meanwhile the pipe condition has an average discharge capacity of 2.11 m^3/s. Refer to the rating curve as shown in Figure 9, below. The pipe condition should be used in assessing the outlet capacity, since it is more conservative than the orifice condition. Refer to Addendum G for detailed calculations of drawdown.

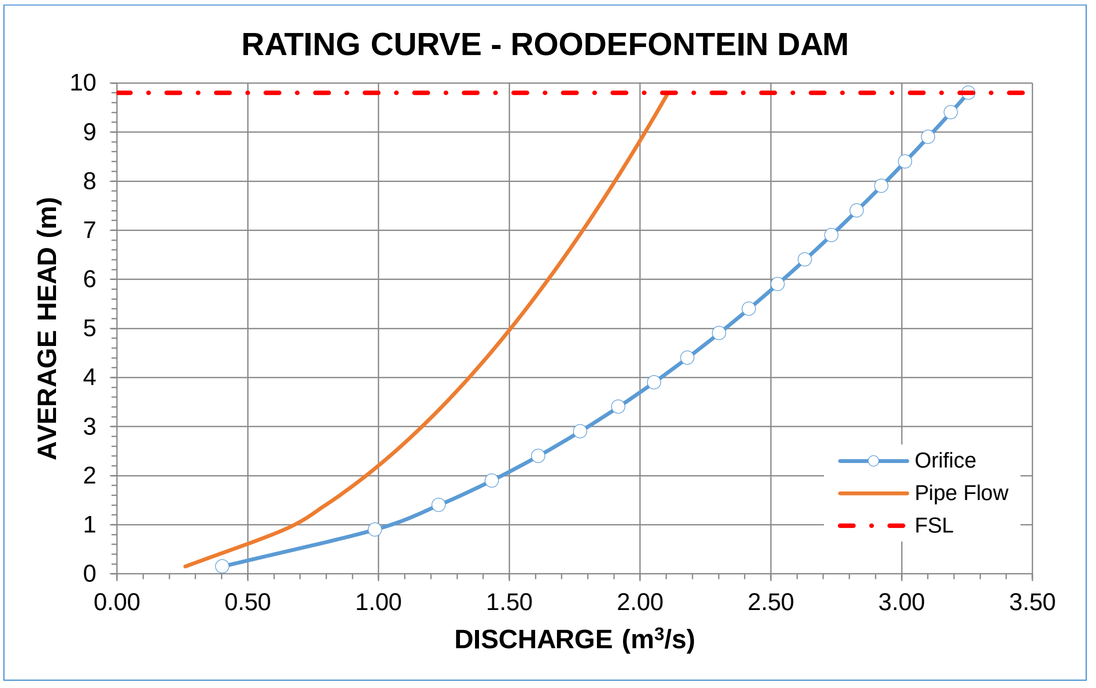

Figure 9: Outlet works Rating curve

To empty the active storage of the dam (that is lowering the reservoir level from full supply level to the lowest invert level) it will take 16 days. Eight (8) days are required to evacuate the top third of the reservoir as shown in Figure 10 and Table 23. The top third of the reservoir normally contain 50% or more of the total active volume in a v - shaped reservoir. As a rule of thumb, lowering the top third of the reservoir should halve the hydrostatic load on the dam wall and results in slowing down of a progressing failure mode, (Courtnadge, 2017).

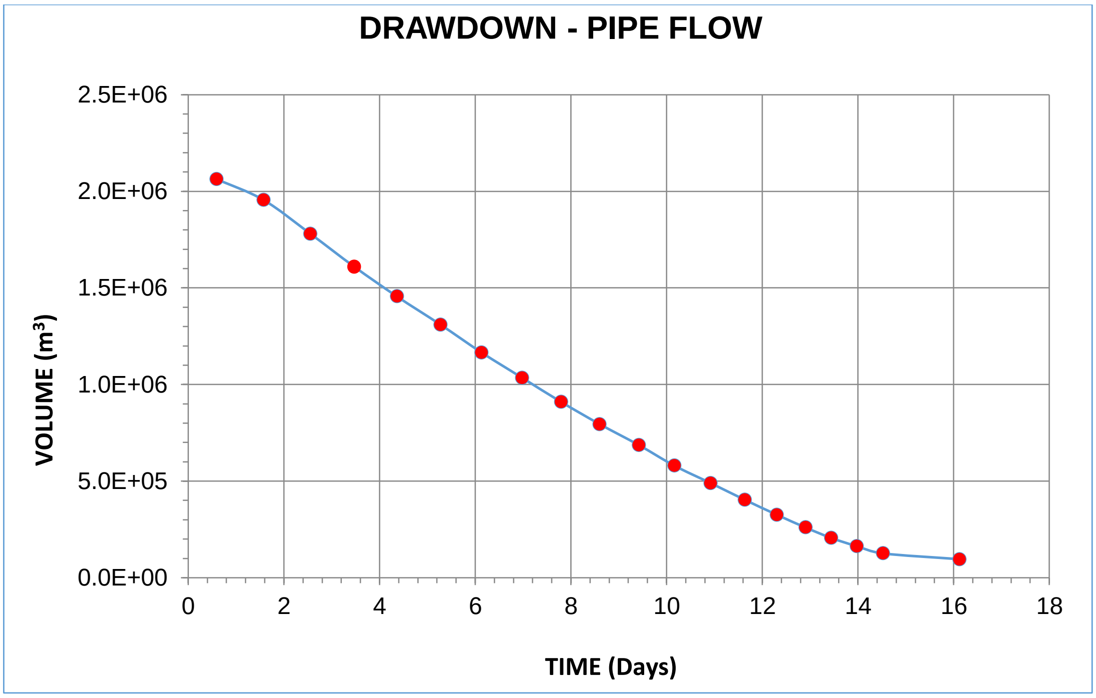

Figure 10: drawdown curve

Table 23: Capacity of installed low level outlet works

DRAWDOWN CAPACITY	
Installed drawdown rate (m^3/s)	Q_D = 2.1 m^3/s
Installed drawdown rate (%H/day)	4.8 %
Installed drawdown rate (mm/day)	491 mm/day
$T_{33\%}$ of the reservoir (m)	3.4 m
RL of $T_{33\%}$	38.9 m
Time to clear $T_{33\%}$	7.79 days
Time to evacuate the active volume	16 days

7.3.2. The Theoretical Drawdown Capacity

Literature on drawdown capacity indicates that the main function of the facility is to lower the reservoir in order to arrest a detected failure mode. The relevant failure mode for drawdown facility is internal erosion in the case of Roodefontein Dam.

7.3.2.1. Hydraulic Gradient

The height of water level at FSL is 14.3 m above river bed and the width of the embankment's base is calculated to total 94 m. These two parameters lead to a hydraulic gradient equal to 0.15, see Table 24.

Table 24: Hydraulic gradient of the dam

HYDRAULIC GRADIENT	
Height of reservoir - H (m)	14.3 m
Width of the embankment's base - L (m)	94.0 m
Hydraulic gradient – I (H/L)	0.15

7.3.2.2. Hole Erosion Index (I_{HET})

Internal erosion is driven by the erodebility of construction material which is assessed using the erosion rate index (I_{HET}). The detection of the erosion is a function of the frequency of surveillance.

The core of the dam is made of Sandy Clay material with 15% plasticity and classified as a low plasticity clay material (Mayer, 200:20) as shown in Table 25. This material used for the construction of the clay core has an erosion rate index of 3.5 as read from the Erosion Rate Index figure, see addendum F.

Table 25: Soil properties for the embankment

SOIL PROPERTIES	
Soil properties for clay core	Sandy Clay; PI = 15%; LL = 27% Classified as CL
Erosion rate index - I_{HET}	3.5

From the theoretical drawdown rate curves, the erosion rate index of 3.5 and hydraulic gradient of 0.15 do not intercept. This might suggest that the used material does not have the ability to form a roof support, meaning the failure mode might not progress to breach. This in essence suggests that the construction material has a self-healing ability.

7.3.3. Drawdown Activation Time

The dam uses a manual activation system for the rapid drawdown. The emergency preparedness plan is silent on the time it takes the operator to travel from his/her base to the dam parking area and then walk into the intake tower to activate drawdown procedure. It is assumed that the activation process will take 3 hours and this assumption will be validated at a later stage. Refer to Table 26 for the used activation parameters for Roodefontein Dam.

Table 26: Drawdown activation parameters

ACTIVATION TIME	
Type of drawdown activation method	Manual
Distance from Dam Operator's base to the dam	Not know
Time to activate the drawdown process	3 hours

7.3.3.1. Telecommunication at the dam

The ability of the operator to communicate with the supervisor/authorised person to activate drawdown procedure within reasonable time plays a vital role in the evaluation of drawdown capacity. Dams located in remote areas with limited network coverage prove to be problematic in the event of emergencies.

At the time of writing this report, major telecommunication companies in South Africa were Telkom SA, Vodacom, MTN and CellC.

The CellC website as accessed on 13 February 2019 showed that there is 3G coverage on the right side of the dam and most parts of the left side of the dam, see Figure 11.

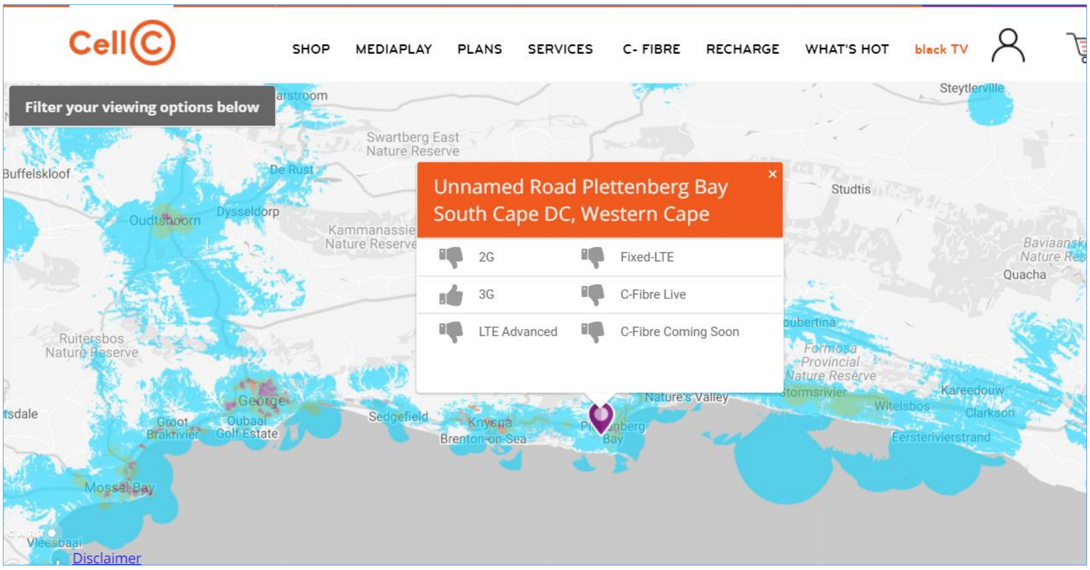

Figure 11: CellC coverage map at Roodefontein Dam ((https://www.cellc.co.za/cellc/coverage-map :13/02/2019))

Vodacom website showed that the dam and the surrounding areas have at least 3G coverage as shown in Figure 12.

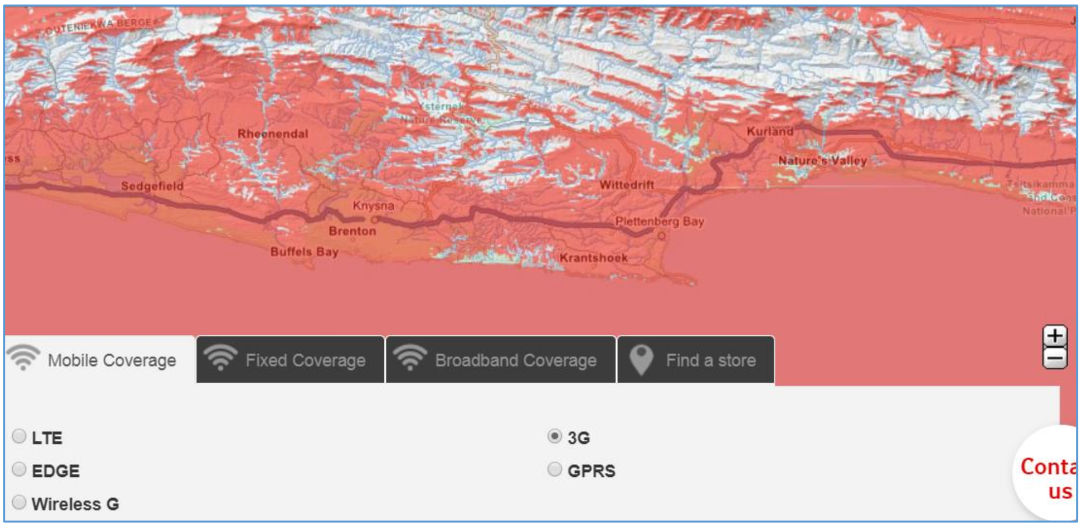

Figure 12: Vodacom coverage map around Plattenberg Bay.
(https://www.vodacom.co.za/vodacom/coverage-map: 13/02/2019)

MTN on the other hand, has very limited network coverage at the dam and the surrounding area as shown in Figure 13. At the time of writing this report, the Department of Water and Sanitation had a contract with Vodacom SA to provide telecommunication devices to qualifying employees. It was therefore assumed that the operator of the dam has a Vodacom device at the time of writing this report.

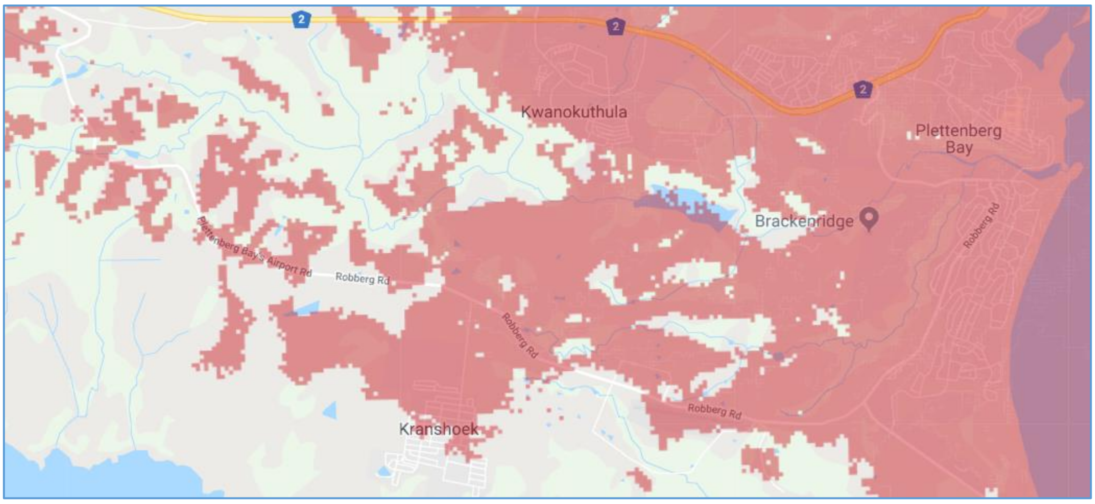

Figure 13: MTN network coverage around Roodefontein Dam
(https://www.mtn.co.za/Pages/Coverage_Map.aspx : 13/02/2019)

7.3.3.2. Access Roads

The dam can be access from the right side of the spillway through a gravel road. This gravel road has been constructed from good material and drainage seems to be working fine. The Operator should not have difficulties accessing the dam to activate the drawdown procedure.

7.3.4. Frequency of Surveillance

The operating and maintenance manual indicates that the dam is inspected every five (5) years, three (3) months, and randomly in-between. In section VI (4) of the manual, it is stated that in the event of strong overflow on the spillway, the dam should be inspected daily. In section VI (5) of the same manual, the operator is encouraged to inspect the dam on weekly basis. However, in section V (2), the operator is directed to record reservoir water level on daily basis while the reservoir is higher or equal to RL 43.0 m. it was noticed in the manual that section IV emphasises the 3 month intermediate inspections and too silent on the weekly/daily routine inspections. The inspection frequency is summarised in Table 27.

Table 27: Surveillance frequency at Roodefontein Dam

FREQUENCY OF SURVEILLANCE	
Major inspection	5 Yearly
Intermediate inspection	3 monthly
Routine inspection	Weekly
Special inspection - A	Daily – When there is a strong overspill
Special inspection - B	Daily – When Water Level is above RL 43.0 m

7.3.5. Evaluation criteria for drawdown capacity

The criteria used to evaluate the adequacy of the installed drawdown capacity are based on USBR guide and the Environmental Agency. The criteria is summarised in Table 28 for the convenience of the reader.

Table 28: Drawdown capacity evaluation criteria

Evaluation Criteria		
Reference	Criteria	
EA, Table 6.2[1]	Minimum rate of 5%H/day and a maximum of 1m/day	515 mm ≤ H ≤ 1000 mm
USBR	Evacuate 75% of Height in 10 to 20 days	
[1]Guide to drawdown capacity for resevior safety and emergency planning, Volume 1 . A Courtnadge etl (date not known).		

7.3.6. Final evaluation of the installed drawdown capacity

The installed drawdown facility has a rate of 491 mm/day which is smaller than the required 515 mm/day. However, the facility is able to lower the top third of the dam in 8 days (7.8 days) which is quicker than the USBR recommended minimum of 10 days.

The used construction material for the core seem to not have the ability to form a roof support and therefore should internal erosion progresses, it will not reach the breach stage, that is; the material will collapse and heal itself. However, the collapsed pipe might lead to the localised settlement of the crest/NOC which intern will reduce the registered height of the freeboard.

The network coverage is fairly ok; the operator should be able to communicate with authorities to seek permission to activate drawdown procedure in the event of an emergency after getting to the site.

The access road to the site as discussed previously is in fairly good condition and it should not hinder the operator under emergency or during the performance of surveillance.

The documented weekly inspection could be problematic should a failure mode start a day after inspection. This means that the failure mode has 6 days to continue undetected.

The drawdown capacity at Roodefontein seems to be adequate.

8. MONITORING AND INSTRUMENTATION

In section – V of the *Operation, Monitoring and Maintenance Manual* it is directed that leakage flow, reservoir water level, Spillway discharge, deformation (Levelling), Rainfall, Draw-off from the reservoir and stability of the left flank above the spillway should be monitored. The *Operation, Monitoring and Maintenance Manual* will be referred to in this report as *The Manual.* All monitoring records are attached in addendum H.

8.1. Seepage Monitoring

It is stated in section V (1) of The Manual that leakage flow should be estimated and recorded. Records of seepage measurements were not available, however there is extensive seepage emanating from the right flank.

8.2. Water Level Monitoring

Section V (2) of *The manual* states that the reservoir water level should be recorded on weekly basis under normal operation. In the event where the reservoir water level exceeds reduced level 43.0 m, the record should be taken on daily basis. The operator of the dam did not have this information.

8.3. Spillway Discharge Monitoring

In section V (3) of The Manual, it is prescribed that periodic flows over the spillway and high reservoir water level reached should be recorded. The operator of the dam did not have this information.

8.4. Deformation Monitoring

A topographic survey of the dam showed that the crest experienced a maximum settlement of 420 mm between Chainage 76 and 93. Along the non-overspill crest, there are uneven settlements. Refer to Addendum H for survey data.

8.5. Rainfall Monitoring

Section V (5) of The Manual prescribes that rainfall recorded should be taken on daily basis. The operator of the dam did not have this information.

8.6. Draw-off from the Reservoir Monitoring

In section V (6) of *The Manual*, it is stated that draw-off volumes from the reservoir should be recorded on weekly basis. Records were not available.

8.7. Stability of Left Flank

Colour photographs of the left flank at and above the spillway should be taken every August of each year as stated in section V (7) of *The Manual*. These photographs should be taken standing 150 m from the spillway. The owner and the operator of the dam were unable to provide the evaluators with evidence supporting execution of this requirement.

9. STRUCTURAL ANALYSIS

9.1. Stability of the Embankment

9.1.1. Properties of the Embankment

The details of the embankment were discussed in details in the previous chapters. The relevant properties of the embankment for stability analysis are the upstream slope of the dam, which has a gradient of 1 v: 3 h on average and the downstream has 1v:2 h. The upstream slopes are protected by Rip-rap against erosion and the downstream slopes are protected by normal Rockfill.

9.1.2. Soil Properties of Embankment

The used soil properties for determination of stability analysis were obtained from the design report: *Raising the full supply level of the dam by 2.0 m* and are listed in Table 29. The author could not locate a document which has in-situ soil properties of the used material during the raising and therefore simply used the worst case soil properties as defined by Watermeyer.

Table 29: Soil properties used for stability analysis (Watermeyer, 2002:12)

SOIL PROPERTIES USED FOR STABILITY ANALYSS				
	Unit Weight		**Cohesion**	**Angle of Friction**
	$\gamma_{Saturated}$	$\gamma_{unSaturated}$	C	ϕ
Core Material 1	22	20	5	24
Core Material 2	22	20	5	24
General Fill	22	20	2	24
Rockfill/Gravel Toe	22	20	0	40
Sand Filter	22	20	0	35
Foundation Material	18	16	0	30

9.1.3. Embankment Slope - Safety Factors

The calculated safety factors of embankment stability are provided in Table 30. There is a 6% difference between the new values and the old values determined by Watermeyer on load case 5. No comparison was done for load cases 2, 3 and 4 since Watermeyer did not present his outcomes for these load cases. The 2006 and the 2011 (Second and Third) dam safety evaluations did not produce new calculations; rather they cited values from Watermeyer.

The calculated values are lower than those of the stability criteria and Watermeyer recommended that after the raising of the dam, in-situ material properties should be determined to evaluate the final stability of the dam. The results of such a study were not available at the time of compilation of this report. Detailed calculations of safety factors are attached in addendum I.

Table 30: Factors of safety for the embankment slopes

EMBANKMENT SAFETY FACTORS					
Side	Load Condition		Safety Factors		Limit
			Watermeyer	Siwelani	
Downstream	FSL	Load case 1	1.3	1.30	1.5
Downstream	FSL, EQ	Load case 2	-	0.99	1.2
Upstream	FSL	Load case 3	-	1.76	1.5
Upstream	FSL, EQ	Load case 4	-	1.05	1.2
Upstream	RDD	Load case 5	1.0	1.07	1.2

9.2. Stability of the Spillway

The Ogee concrete spillway at the dam is 3.72 m height measured from the downstream side of the dam. The total bottom width that has an influence in stability determination is 5.84 m. Detailed dimensions of the raised spillway are contained in drawing number 150193/06: Spillway Sections and Details. Figure 14 is a simplified sketch of the spillway cross-section.

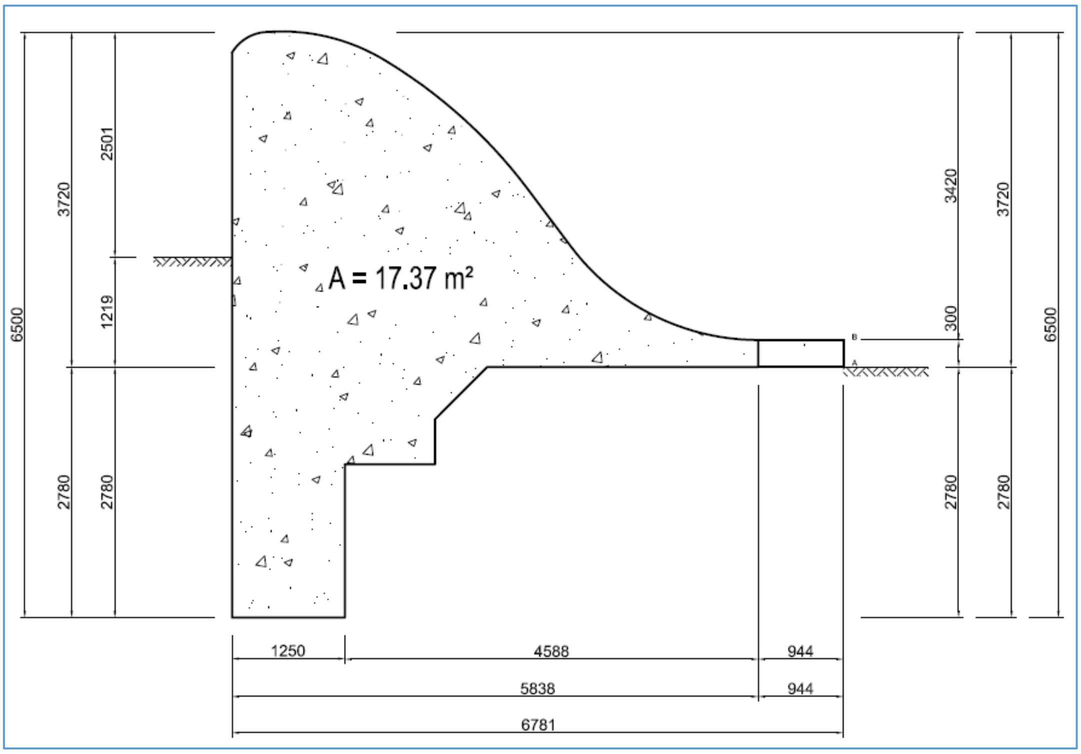

Figure 14: Cross-Section of the concrete spillway

9.2.1. Assumptions

The assumptions made in determining concrete stability of the spillway are shown in Table 31 below. It was further assumed that the tail water level is at zero height in order to neglect the positive influence of this component. The iffluence of ice has not been considered since the area where the dam is located does not experience ice with 25 mm thickness. Detailed calculations have been attached in addendum I.

Table 31: Assumed parameters for concrete stability analysis

ASSUMED PARAMETERS FOR CONCRETE STABILITY ANALYSIS	
Parameter	**Value**
Unit weight of water	9.81 KN/m^3
Unit weight of concrete	24 KN/m^3
Area reduction factor	1
Submerged unit weight of sediments	18KN/m^3
Angle of friction for sediments	30°
Cohesion of foundation	1500 KN/m^2
Angle of friction for foundation	59.53°

9.2.2. Stability Evaluation

Watermeyer performed a comprehensive stability analysis in 2002 during the raising of the full supply level for Roodefontein Dam. The results were evaluated for the purposes of this evaluation and remain valid. Details of his estimates are covered in Annexure D of a report titled *Raising the full Supply Level (FSL) of the Dam by 2.0 m by means of solid raising: Design Report*.

10. CHECKING OF REGISTRATION INFORMATION

10.1. Dam Safety Office Record

The registration information as captured by Dam Safety Office was evaluated and found to be valid and accurate. There are no revisions required. A copy of the information is attached in Addendum J.

11. EVALUATION OF HAZARD POTENTIAL

11.1. Dam Classification

Roodefontein Dam is classified as a category three (3) dam with a higher hazard potential. The summary of the categorising information is provided in Table 32.

Table 32: Classification data of Roodefontein Dam

CATEGORIZATION OF THE DAM	
Size class:	Medium
Hazard potential rating as classified:	High
Category	III
Do you agree with the rating?	Yes

11.2. Failure Modes Analysis

The following sub-sections of the report will briefly discuss identified potential failure modes relevant to Roodefontein Dam.

11.2.1. Internal Erosion

Roodefontein Dam like any other embankment dam is susceptible to internal erosion. One of the threats or initiators of internal erosion could be high floods increasing the loading on the earthfill embankment. Alternatively, deterioration of construction material could lead to internal erosion under normal operation conditions. The dam has a clay core, vertical chimney, horizontal blanket and a Toe drain. This indicates that modern dam engineering philosophy was followed during the design and construction of the dam.

In the event where the core initiates internal erosion, the vertical chimney constructed from Beacon Beach Sand should stop the clay material from washing out and prevent internal erosion from progressing. The newly raised section of the core is constructed from sandy clay with low plasticity and it does not have the ability to form a roof support, therefore internal erosion should not progress to breach stage.

The contact area between newly constructed sandy clay core and the old clay core is concerning in broad terms. This area can lead to cracking consequently contact internal erosion. However, in the case of Roodefontein Dam, the vertical chimney drain should prevent the clay material from escaping as mentioned above.

Internal erosion failure mode for Roodefontein Sam is credible and significant, therefore should be analysed in details under dam break analysis. The Environmental Agency, stipulated that probability of dam failure due to internal erosion ranges between 1×10^{-10}.

11.2.2. Slope instability

Slope instability can be initiated by reservoir loading, rapid drawdown or saturation as results of either overtopping or heavy rain.

11.2.2.1. Slope instability due to Overtopping

The calculated freeboard of the dam seems to be insufficient as discussed in the previous sections. This increases the probability of dam failure due to downstream slope saturation as a result of overtopping. The estimated probability of dam failure due to slope instability as a consequence of saturation is 1.03×10^{-06}.

11.2.2.2. Slope instability due to rainfall

Plattenberg Town has an average annual rainfall of 930 mm. The probability of dam failure due to saturation of downstream slopes by rainfall is relatively low. This failure mode is credible but not significant.

11.2.2.3. Embankment Slope instability due to rapid drawdown

The calculated factor of safety for rapid drawdown condition is 1.1 which is lower than the modern limit of 1.2. The dam under this condition satisfies the laws of equilibrium and therefore instability of upstream slope due to rapid drawdown is not expected. It should be noted that this failure mode is credible but not significant.

11.2.2.4. Embankment Slope instability due to loading

The calculated safety factor for normal operation loading is 1.3 and 1.76 for downstream and upstream slopes, respectively. Modern dams have a minimum safety factor of 1.5 (Jansen 1988) for both the downstream and upstream slopes under normal loading. The downstream slope of this dam is below the required 1.5. Since the loading conditions satisfy equilibrium condition in favour of stability, the failure mode is credible but not significant under normal loading.

Under extreme conditions with earthquake loading, the upstream slope has a 1.05 safety factor. Under these conditions, the dam satisfies the laws of equilibrium and therefore, the failure mode is credible yet insignificant.

11.2.2.5. Spillway Overturning

The hydrostatic force, uplift, silt load, ice and wave loading could result in the spillway overturning should they exceed the counter weight and tail water loading of the spillway. The calculated factor of safety against overturning for the spillway is found to be 3.05 and the probability of the structure overturning is 2.8×10^{-05}.

11.2.2.6. Spillway Sliding

The same loading as discussed above could result in sliding of the spillway. The calculated factor of safety against sliding is 1.87 and its probability is 1.03×10^{-06}.

11.2.3. Erosion

Embankment dams are susceptible to erosion; hence a form of protection is required. Erosion can be caused by heavy rain, reservoir wave and flood waves as a result of overtopping of the NOC.

11.2.3.1. Earthfill erosion due to rain and reservoir waves

It was discussed in the previous chapters that the upstream slope is protected by means of 550 mm thick layer of riprap comprising of boulders ranging from 350 mm to 550 mm. The riprap is constructed on a 400 mm thick bedding layer comprising of 250 mm to 150 mm material. A 300 mm secondary bedding layer was constructed to prevent the earthfill from eroding through the 400 mm layer.

The downstream slope is protected by means of 250 mm thick layer of rockfill. The probability of earthfill erosion due to rainfall is too low, therefore, making the failure mode credible but insignificant.

The upstream slope protection seems to be thick enough and well graded. The probability of earthfill erosion due to reservoir waves is 1×10^{-4}.

11.2.3.2. Earthfill Erosion due to Overtopping of NOC

The flood waves are powerful enough to erode the downstream rockfill slope protection and the earthfill should overtopping occurs. In the event of overtopping of the NOC, the crest will experience backward erosion and create a passage for the reservoir water to evacuate uncontrolled.

These flood waves and the escaped reservoir water will transport earthfill material, this process should it continue, the embankment dam will breach at some point. Roodefontein Dam does not have enough freeboard as calculated in this report and therefore the probability of failure due to this failure mode is high, credible and significant. The probability of the dam failing due to overtopping is 1.4×10^{-06}.

11.3. Dam Break Analysis

During this fourth dam safety evaluation, dam break modelling was not conducted. The risk analysis of the inundated area is based on the 2002 dam break model that was performed by Watermeyer and presented in his design report for the raising of the fully supply level by 2.0 m by means of solid raising.

11.3.1. Inundation

The flood wave converges into the 1:100 flood plains 4.2 km downstream of the dam. The estimated time for the flood wave to reach 4.2 km is 38 minutes, meaning that this wave has an average speed of 1.8 m/s to inundate the section.

The area after this convergence point should not be affected in terms of socio-economic disadvantages or suffers any fatalities since municipal by-laws prohibits construction of buildings within 1:100 flood plains. Watermeyer's results of a dam break flood analysis are shown in Table 33.

Table 33: Dam break results (Watermeyer, 2004:N.9)

DAM BREAK RESULTS		
Distance Downstream of the Dam (km)	Peak Flood Flow Surface Level above Mean sea Level (m)	Time to reach the Location (minutes)
0.7	30.7	22
1.0	25.3	23
1.2	22.3	24
1.5	18.0	25
1.75	14.6	26
2.1	10.1	27
3.0	9.0	32
3.4	7.2	34
3.6	8.6	35
4.2	8.5	38

11.4. Consequences of Dam Failure

In the third dam safety evaluation, Hattingh suggested that the population at risk in 2011 ranged between 376 and 750 people. Direct monetary losses were estimated between R300 and R350 million. Socio-economic impact was selected as high and ecological status was class C. The current impact analysis of a dam failure on economy, environment, health of people and ecology is covered in the following subsections.

11.4.1. Economic Consequences

To estimate the economic consequences of dam failure the author analysed the landscape within the flood plains. One structure (Horse stable) was added to the inundated area since 2004. Therefore, the only varying parameter becomes inflation and the cost of this structure. It is estimated that the horse stable will cost about R 2 million to replace.

Since 2011, annual national inflation fluctuated between 5.0 and 4.5 and the average inflation value is 5.3 as shown in Table 34. To quantify the current economic costs associated with failure of Roodefontein Dam the average inflation value and the economic costs reported by Hattingh in his 2011 report were used. This resulted in revised economic cost ranging between R 458 million and R 533 million.

Table 34: South African Annual Inflation from 2011 to 2018 (Statistics South Africa)

South African Inflation (2011 - 2018)			
Year	Annual Inflation (%)	Economic Cost (million)	
2011	5.0	300	350
2012	5.6	317	370
2013	5.7	335	391
2014	6.1	355	414
2015	4.6	372	434
2016	6.4	395	461
2017	5.3	416	486
2018	4.7	436	509
2019 (March)	4.5	456	531

11.4.2. Environmental Consequences

In general, dam failures have high environmental consequences and Roodefontein Dam is not exceptional to this concept. The expected level of erosion and destruction of the vegetation downstream of the dam and well a loss of aquatic life is expected to be of higher order.

11.4.3. Loss of Life

Due to the tourism status of this area, there number of lives that can be lost in the event of a dame failure is estimated be between 15 and 30.

11.4.4. Social Consequences

The social impact caused by failure of Roodefontein Dam remains high since Plattenberg and the surrounding areas are tourist's locations.

11.4.5. Ecological Consequences

The ecological state has not been review, therefore remains as class C as stated in the third dam safety evaluation report.

11.5. Evaluation of Emergency Preparedness Plan

The first Emergency Preparedness Plan (EPP) compiled by Watermeyer in 2004 has a hybrid format, that is; it combines the internal and external protocol to be followed in the unfortunate case of an emergency. In the EPP, a map showing access roads is attached so is a contour map showing potential areas which will be affected by flooding. Contact details of different authorised personnel and external organisations that handle emergency situations have been provided in page N.6 to N.8.

The recent Emergency Preparedness Plan was compile by Weidemann and Horn in 2015. This copy has updated contact details of downstream dwellers that will be affected by a dam failure and contact details of emergency services have also been included.

Precise action to be taken by the operator has been stated clearly and in a simplified format. The internal protocol has been well covered in this copy of the EPP.

11.6. Summary of Hazard Potential

Table 35 gives a summary of impact associated with failure of Roodefontein Dam. Most of the information in Table 35 was extracted from the previous dam safety evaluation report . The previously estimated 24 number of people who might lose their lives due to dam failure has been retained in this report because of the newly built horse stable immediately downstream of the dam wall.

Table 35: Risk analysis results of Roodefontein Dam

RISK ANALYSIS RESULTS		
Risk analysis level:		Level 0
Trigger event of failure:		Sunny Day Failure
Probability of failure:		5×10^{-4} and 5×10^{-5}
Analysis method:		Dam Break Analysis
Population at risk:		750
Estimated loss of life:		Between 15 and 30
Financial Loss	Directly:	R 458 million to R 533 million
	Indirectly:	R 4.6 million to R 5.3 million
Economic Losses		High
Environmental Losses		High
Social Impact		High
Ecological Loses		Class C

12. INSPECTION OF THE DAM

12.1. Dates and Details of Evaluators

The fourth dam safety inspection of Roodefontein Dam took place on 25 September 2019. The author, the Approved Professional Person, and other officials as listed in Table 36 conducted the inspection.

Table 36: Team members who conducted visual inspection of the dam

DETAILS OF PERSONNEL			
NAME	**ROLE & RANK**	**COMPONENT / ORGANIZATION**	
Mr Siwelani, RW	Author, Candidate Engineer	DSS*	DWS
Mr Mahlabela, CN	APP, Chief Engineer	DSS	
Mr Kgopiso, JM	Evaluator, Pr.Technician	DSS	
Mr Mabale, TI	Evaluator, Pr Technologist	DSS	
Mr Mothlagomang, LW	Evaluator, Pr Technician	DSS	
Mr Desai, T	Evaluator, Candidate Engineer	DWSRO**	
Mr Mouton, D	Evaluator, Candidate Engineer	DWSRO	
Mr Janse Van Rensburg, L	Evaluator, Pr Technician	DWSRO	
Mr Weidemann	Evaluator, Area Manager	DWSRO	
Ms Samuel, FL	Evaluator, Pr Eng	BM***	
Mr Tarentaal, R	Evaluator, Pr Eng	BM	
*DSS - Dam Safety Surveillance. **DWSRO - Dept. of Water & Sanitation Regional Office. *** BM - Bitou Municipality			

12.2. Weather and Water Level

On 25 September 2019, the skies were clear (Figure 16) around the dam with temperatures below 30 °C. On the inspection date, the gauge plate was at 9.8 m (Figure 15). Weather records revealed that on 24 September 2019 there was 10 - 50 mm rain in Plattenberg Bay. A summary of the weather data and Inspection dates is provided in Table 37 below.

Table 37: Inspection data of Roodefontein Dam

VISUAL INSPECTION DATA	
Date:	25 September 2019
Gauge Plate reading:	About 9.8 m (Figure 15)
Had it recently rained?	Yes, on 24 September 2019. Rain Intensity: 10 - 50 mm
Describe weather:	The skies were clear

Figure 15: Gauge Plate reading

Figure 16: Clear Weather at the dam on 25 September 2019

12.3. Dam Site

12.3.1. Surrounding Terrain

The dam site is located in a mini-valley, with both the right and left sides having densely vegetated steep slopes as shown in Figure 17 and Figure 18 , below.

Figure 17: A densely vegetated hilly left side of the reservoir's surrounding

Figure 18*:* A densely vegetated hilly right side of the reservoir's surrounding

The valley opens up on the downstream side of the dam. This side of the dam has recreational facilities comprising of a golf course, and a newly built horse stables, refer to Figure 19 below.

Figure 19: A valley downstream side of the dam with a newly built horse stables

12.3.2. Access to dam site and Lighting of dam wall

The dam site is accessible through Piesang Valley Road which connects to the National Route - N2. On Piesang Valley Road, 1.9 km off N2, there is a local gravel road on the right, leading to the dam site. Distance from Piesang Valley Road to the Dam site is estimated at 2.4 km. Refer to Figure 20 below.

The inspection team did not locate any form of lighting installations on the dam wall

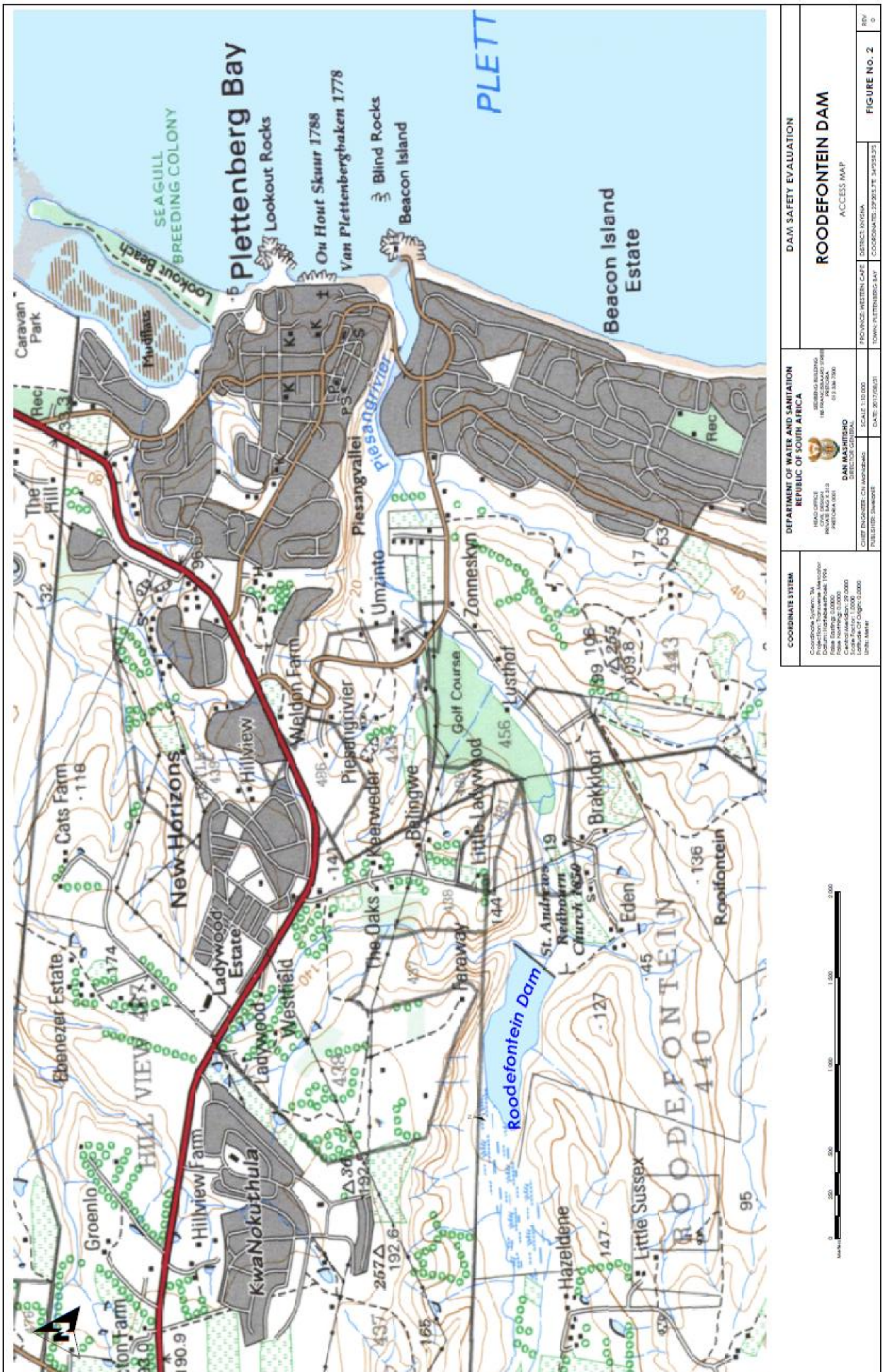

Figure 20: Access Route to Roodefontein Dam.

12.3.3. Access to the dam

The dam is within a cluster of private properties and a security-controlled gate is utilised through one of the private properties to access the dam wall. However, members of these private properties have uncontrolled access to the dam site and wall.

12.4. Operational Health and Safety

12.4.1. The Safety Boom

There is no safety boom installed at the dam, see Figure 21.

Figure 21: Side view of the spillway showing a lack of safety Boom

12.4.2. Safety

The stairway on the upstream slope leading to the gauge plate is relatively steep and it does not have railing, see Figure 22. This setup might expose officials working at the dam to a danger of falling. The evaluation team did not observe any unsafe practice on the day of inspection nor find evidence proving that there is unsafe practice at the dam.

Figure 22: Stairway leading to the gauge plate.

12.5. Evaluation of Dam Owner's Operation and Maintenance Programme

12.5.1. Dam Safety Logbook

Officials at the dam did not have a copy of a Civil Logbook on the day of the inspection. However, a generic copy of the logbook is available in Pretoria and it will be made available.

It was stated that the operator of the dam, Mr R Tarentaal has not attended the *Water Control Officer Course* which would assist him in executing his operational duties more efficiently.

12.5.2. Operation and Maintenance manual

The owner of the dam has a copy of the operations and Maintenance (O&M) manual in place. The first O&M Manual was produced in 1998, an update was approved in 1990 and the latest copy was approved in 2005, see Figure 24 & Figure 23. On the day of the inspection, copies of these documents were in Pretoria offices of the Owner and on - site. Officials from Bitou Municipality did not have copies of these documents, however a disk with electronic copies of the documents was provided to them during the site inspection.

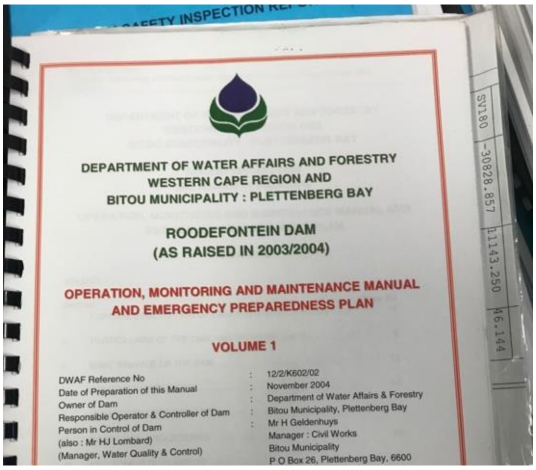

Figure 24: A copy of 2004 O&M document

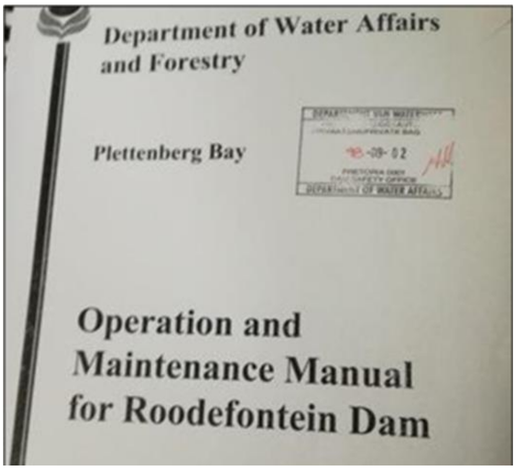

Figure 23: A copy of the 1990 O&M document

12.5.3. Emergency Preparedness Plan (EPP)

The dam owner has an Emergency Preparedness Plan (EPP) in place. The lasted copy was approved in 2015 and the older copy was approved in 2005, see Figure 25 & Figure 26 . On the day of inspection, officials at the dam had a copy of the 2015 EPP. The 2015 EPP is valid and the contact details are valid.

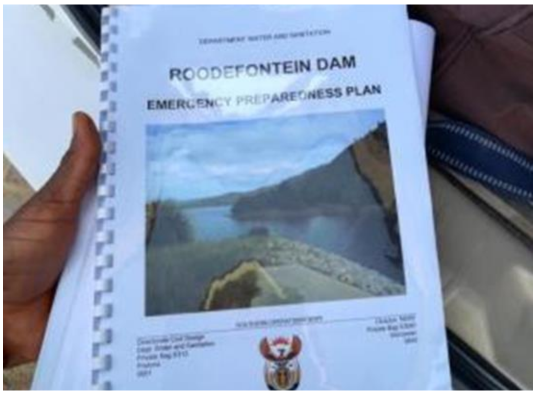

Figure 26: 2015 copy of EPP

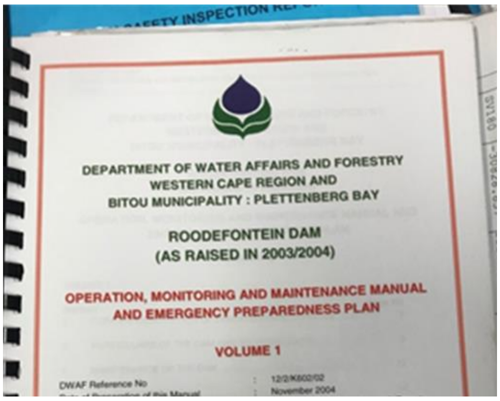

Figure 25: A copy of 2005 EPP

12.6. Non Overspill Crest (NOC)

12.6.1. NOC Properties

As-Built drawing No. 150195/06 (WA166401/020) indicates that a 20 mm layer of gravel protects the non-overspill crest. During the inspection it was realised that grass has been incorporated and it performed satisfactory in protecting the NOC, see Figure 27.

Figure 27: Non - Overspill crest of Roodefontein Dam.

12.6.2. Defects

The visual inspection revealed that there is no settlement, cracks or ponding on the crest and the cross fall was not visible.

12.6.3. Undesirable Elements and Activities

There is evidence of ratting caused by vehicle wheels on the crest, see Figure 27 above. The only found evidence concerning animals is the presence of ants at their early stage as shown in Figure 28; however, at this stage these activities are not critical to an extent of causing a dam failure.

Figure 28: Ants activities on the crest

12.7. Upstream Slope of Dam Wall

12.7.1. Vegetation Growth and Slope Protection

On the upstream slope, a 500 mm layer of Rip-Rap provides protection to the slopes. This layer is built on a 400 mm bedding layer consisting of 150 - 250 mm crusher. Both layers are in good condition and the material quality is excellent.

On the day of inspection, the slopes had some isolated vegetation as can be seen in Figure 29 and Figure 30 below. The quantity and height of this vegetation should not cause any failure of the dam. However, the Owner of the dam should clear this vegetation regularly.

Figure 29: Isolated vegetation on the upstream slope.

Figure 30: Upstream slopes of the dam wall.

12.7.2. Defects

On the upstream slope, the evaluators did not find any evidence consistent with bulging, sliding, cracking, or erosion.

12.7.3. Animal or Termite Activities and Footpath

The evaluators did not find any evidence concerning animal or termite activities nor vehicles and footpaths.

12.8. Downstream slope of Dam wall

12.8.1. Vegetation growth

The area downstream of the outlet works has been kept well with grass trimmed correctly. The area close to the spillway has a few bushes, see Figure 31 and Figure 32. Despite the bushes, the overall condition of the downstream slope as far as vegetation growth is concerned is satisfactory.

Figure 31: Some vegetation on the downstream slope of the wall.

Figure 32: View of the dam wall showing some bushes on the downstream slope.

12.8.2. Slope protection

A layer of crimping grass protects the downstream slopes, see Figure 33 and Figure 34 below. As mentioned in the previous sub-section, the grass layer was well taken care off and it was performing satisfactory.

Figure 33: View of the downstream slope from the toe.

Figure 34: Grass protection on the downstream slope of the dam.

12.8.3. Defects

The team could not satisfactorily evaluate the existing horizontal crack due to the grass cover see Figure 35.

The evaluation team did not find any evidence to prove the existence of erosion, bulging, wet patches, and seepage leaks. Furthermore, there were no signs of animal activities, vehicle pathways, or footpaths.

Figure 35: Existing horizontal crack covered by grass on the downstream slope.

12.9. Downstream Toe and Flanks

The dam does not have a conventional toe drain system. The study of existing as-built drawings, existing reports, and the physical inspection of the dam confirmed the nonexistence of the toe drain. On the toe there is surface water drain in a form of unlined channel, see Figure 36.

Figure 36: Strong vegetation growth in the Toe-Channel.

On the day of inspection the unlined channel had overgrown vegetation, see Figure 36 above. The vegetation was consistent with the presence of water.

Downstream of the right flank, a sizeable amount of seepage was observed, see Figure 37. The seepage water collects behind the outlet house and is channelled through a small channel which connects to the toe-channel, see Figure 38.

Figure 37: Seepage from the right flank day lighting next to the outlet house

Figure 38: Seepage water channel to a Toe-Channel and discharge in the river

12.10. Other observations

A lined-channel from the downstream property discharges into the toe-channel of the dam and therefore, the amount of water ponding downstream of the dam increases due to this new structure, see Figure 39.

Figure 39: Lined Surface drain from the downstream property discharging into the Toe-Channel

12.11. Spillway

12.11.1. Spillway Approach Channel

Previous reports indicate that the left flank experienced a slip failure in the past. Material from this failure was deposited in the approach channel, see Figure 40 and Figure 41 below. The influence of these deposits on the dam should be investigated.

Figure 40: Spillway approach channel.

Other than the material deposits from the slip failure, there is no additional loose material in the approach channel.

Figure 41: material deposit due to a slip failure on the right flank

12.11.2. Conditions of concrete

On the day of inspection water level was below the full supply level, allowing the team to closely inspect the spillway, see Figure 42 and Figure 43 below. The concrete used on the spillway is intact with no signs of major cracks or seepage.

Figure 42: Side view of the spillway

Figure 43: Concrete conditions of the Staling Basing

12.11.3. Defects on the Spillway

On the downstream left side of the spillway, one of the concrete panels exhibits signs of minor movement as can be seen in Figure 44.

Other than these minor movements, there are no signs of significant cracking, significant movement or settlement on the spillway.

Figure 44: Signs of minor concrete panel movement.

12.11.4. Stability of the Walls

All the walls around the dam looked stable with no signs of elements which could result in instability.

12.12. Outlet Works

Detailed inspection of the outlet works is covered in the appended Electro-Mechanical Report. The roof of the outlet house has been damaged and it requires repair work, see Figure 45.

Figure 45: Damaged roof of the outlet house

12.13. Other Observation

12.13.1. New Infrastructure

A new horse stable has been constructed on the downstream side of the dam. This new structure is located less than 100 m from the toe of the dam, see Figure 46 and Figure 47.

Figure 46: A horse stable constructed on the downstream of the dam wall

Figure 47: View of the downstream taken from the NOC.

12.13.2. Contractual Arrangement

During the inspection, it was indicated that there is no formal contractual arrangement between the Department and the Operator (Bitou Municipality). This lack thereof, negatively affects the safe operation of the dam.

12.14. General observations

In general, the physical conditions of Roodefontein Dam are satisfactory. However, the evaluation revealed that certain components of the dam do not meet the requirements of Dam Safety Regulations, Regulation 139 of 24 February 2012.

12.15. Mechanical Evaluation

A physical inspection of mechanical and electrical components was conducted on 11 February 2020. The mechanical report reveals that conditions of outlet works are reasonable and the outlet works are functioning satisfactorily. However, the required maintenance work was not performed due to a lack of maintenance contractor (Kolorovic, 2020). The electro-mechanical report makes the following:

- The shaft on the isolating gate valve is bent and it is difficult to operate, therefore, it must be repaired or replaced as soon as possible.
- The upstream valve must be repaired since it is not sealing.
- The indication on the wheel is missing and must be rectified.
- The front seal rubber on the sleeve valve is damaged and must be replaced.
- The sleeve valve must be refurbished since is in poor conditions.
- Open and close indicator must be installed since the original ones are missing.
- Repair the corrosion protection on the underwater manifold.
- Investigate alternatives for the manifold outlet works.
- Introduce a mechanical logbook to ensure good maintenance programme.
- Drawings of the new outlet works must be submitted to Strategic Asset Management as soon as possible for record keeping.

13. CONCLUSION

13.1. Contractual arrangement

The lack of a formal contractual arrangement between the Owner of the dam (Department of Water and Sanitation) and the Operator (Bitou), Municipality negatively affects the safe operation of the dam.

13.2. Freeboard

The dam does not have sufficient freeboard to safely pass the recommended design flood of 95 m^3/s.

13.3. Outlet House

The outlet house has been damaged and repair work is required.

13.4. Toe Drain

The current unlined Toe-Channel is performing satisfactorily in conveying seepage water, however, it create saturation on the toe of the dam.

13.5. Private drainage channel

The newly constructed private drainage channel discharges into the unlined Toe-Channel of the dam and therefore contributes into saturation of the toe of the dam.

13.6. The newly constructed horse stable

The newly constructed horse stable, few meters downstream of the toe of the dam affects the risk profile of the dam. The Department might be exposed to financial claim should the dam fails.

13.7. Crack on the earthfill embankment

The horizontal crack on the earthfill embankment seem to have stabilised/stopped, however, the cause root for the cracks is still unknown.

13.8. Embankment stability

The calculated downstream safety factor is 1.3 under full supply level - loading and therefore does not satisfy the required minimum of 1.5.

13.9. Concrete panel on the spillway channel

The movement of concrete panel on the left side, downstream of the spillway could be an indication of earth movement.

13.10. Safety Boom

The lack of a safety boom could result in a public safety noncompliance if boats are allowed at the dam.

13.11. The Civil Logbook

The absence of the Civil Logbook prevents the efficient operation of the dam.

13.12. Water Control Officer Course

The operator of the dam conducts his duties without the required training provided through the Water Control Officer Course. This lack of training has a potential to limit the officer from effectively discharging his duties.

14. RECOMMENDATIONS

14.1. Outstanding from Previous Inspection

There are few outstanding recommendations from the previous evaluation report and these are summaries in *Table 38* below.

Table 38: Recommendations not implemented from the previous Dam Safety evaluation.

UN-ATTENDED RECOMMENDATIONS FROM THE PREVIOUS INSPECTION REPORT (S)	RESPONSIBLE OFFICE	TIME FRAME
1. Survey of the slope failure on the left bank, upstream of the spillway.	SAM	ASAP
2. The drainage canal upstream at the toe of the dam should be cleaned and kept clean as part of the routine maintenance at the dam.	BM	Regularly
3. Investigate the rehabilitation of natural slope failure behind the outlet house.	SAM	ASAP

14.2. New recommendation from this Inspection

Table 39 gives a breakdown of dam safety related recommendations stemming from this evaluation. The times indicated in column three (3) of *Table 39* should be taken from the date at which the Dam Safety Office approves this report. Recommendations having substantial financial implications have been given a minimum of 3 year for completion. Column three (3) of *Table 39*, titled time means the maximum time recommended to conclude the activity. The authors have noted that at the time of writing this report, the owner was in a process of appointing a service provider to rehabilitate the dam.

Table 39: Dam safety related recommendations

DAM SAFETY RECOMMENDATION	RESPONSIBLE OFFICE	TIME
1. All recommendations from the mechanical report should be implement as indicated in the report.	See Report	SR
2. Complete the outstanding recommendations from the previous evaluation as repeated in Table 38 of this report.	See Table 38	12 months
3. Formalise the contractual arrangement between the Owner (Department of Water and Sanitation) and the Operator of the Dam (Bitou Municipality) to allow for legal delegation of powers and efficient operation of the dam.	DDG: IBOM	12 months
4. Increase the freeboard of the dam to ensure compliance with 2011 *Guidelines on Freeboard for Dams.*	SAM	3 Years
5. Repair the damaged roof of the outlet house.	BM & SO	12 months
6. Install a proper toe drain system to manage seepage on the downstream of the dam.	SAM	3 Years

7. Install a lined - channel on the toe of the dam to collect run-off water from the toe of the dam and discharge in the river.	SAM	3 Years
8. Install guardrails on the stair-way leading to the gauge plate to improve safety of personnel working at the dam.	SO	3 Year
9. Request owners of the property immediate downstream of the dam to re-rout the newly installed surface drain away from the dam to avoid saturating the toe of the dam which might trigger slip failure of the earthfill embankment.	SO & BM	immediately
10. Department should determine an appropriate action concerning the newly built horse stable downstream of the earthfill embankment.	DG	12 months
11. Rehabilitate the earthfill embankment in order to address the horizontal crack on the embankment and the slope instability.	SAM	3 Year
12. Monitor survey beacon F13 on the downstream left side of the spillway channel for a possible movement of the support material.	SO & BM	Quarterly
13. Install a safety boom upstream of the spillway	SO & BM	1 Year
14. Provide a Civil Logbook	SAM	3 Months
15. Provide training to the Operator of the dam	SAM	1 Year

BM - Bitou Municipality | SO - Department of Water and Sanitation: Southern Operation
SAM - Strategic Asset Management | DDG:IBOM – Deputy Director General: Infrastructure Branch

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https://www.mtn.co.za/Pages/Coverage_Map.aspx : 13/02/2019

16. ADDENDA

16.1. **Addendum A:** Access and Locality Map

16.2. **Addendum B:** As-Built Drawings

16.3. **Addendum C:** Geology

16.4. **Addendum D:** Hydrology Report

16.5. **Addendum E:** Flood routing

16.6. **Addendum F:** Freeboard Calculations

16.7. **Addendum G:** Drawdown Capacity

16.8. **Addendum H:** Monitoring Data

16.9. **Addendum I:** Stability Analysis

Addendum A: Access and Locality Map

0 250 500 1 000 1 500 2 000
Meters

COORDINATE SYSTEM	DEPARTMENT OF WATER AND SANITATION REPUBLIC OF SOUTH AFRICA		DAM SAFETY EVALUATION			
Coordinate System: TM Projection: Transverse Mercator Datum: Hartebeesthoek 1994 False Easting: 0.0000 False Northing: 0.0000 Central Meridian: 29.0000 Scale Factor: 1.0000 Latitude Of Origin: 0.0000 Units: Meter	HEAD OFFICE CIVIL DESIGN PRIVATE BAG X 313 PRETORIA 0001 DAN MASHITISHO DIRECTOR GENERAL	SEDIBENG BUILDING 185 FRANCISBAARD STREET PRETORIA 012 336 7500	**ROODEFONTEIN DAM** ACCESS MAP			
	CHIEF ENGINEER: CN Mahlabela	SCALE 1:10 000	PROVINCE: WESTERN CAPE	DISTRICT: KNYSNA	**FIGURE No. 2**	REV 0
	PUBLISHER: SiwelaniR	DATE: 2017/08/31	TOWN: PLETTENBERG BAY	COORDINATES: 23°20'5.7"E 34°3'59.3"S		

Meters 0 150 300 600 900 1 200

COORDINATE SYSTEM	DEPARTMENT OF WATER AND SANITATION REPUBLIC OF SOUTH AFRICA		DAM SAFETY EVALUATION			
Coordinate System: TM Projection: Transverse Mercator Datum: Hartebeesthoek 1994 False Easting: 0.0000 False Northing: 0.0000 Central Meridian: 29.0000 Scale Factor: 1.0000 Latitude Of Origin: 0.0000 Units: Meter	HEAD OFFICE CIVIL DESIGN PRIVATE BAG X 313 PRETORIA 0001 **DAN MASHITISHO** DIRECTOR GENERAL	SEDIBENG BUILDING 185 FRANCISBAARD STREET PRETORIA 012 336 7500	**ROODEFONTEIN DAM** ACCESS MAP			
	CHIEF ENGINEER: CN Mahlabela	SCALE 1:6 000	PROVINCE: WESTERN CAPE	DISTRICT: KNYSNA	FIGURE	REV 0
	PUBLISHER: SiwelaniR	DATE: 2020/01/29	TOWN: PLETTENBERG BAY	COORDINATES: 23°20'5.7"E 34°3'59.3"S		

Addendum B: As-Built Drawings

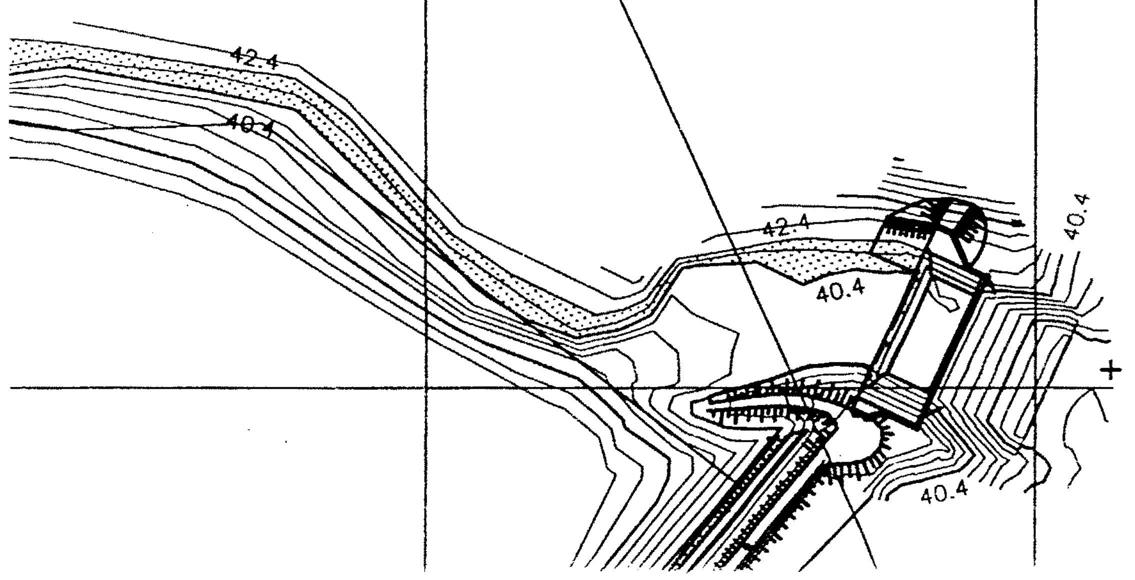
RAISED ROODEFONTEIN DAM

150207.tif
2006/03/14

SYSTEM WG.23	Y	X	Stud Level 06·01·2005
DAM1	-30786.129	11324.100	46.194
DAM2	-30798.369	11333.898	44.674
LF1	-30967.717	10946.632	46.038
LF3	-30984.223	10963.457	41.992
OD1	-30980.885	10970.344	39.434
OD2	-30975.255	10981.848	39.438
OD3	-30967.533	10998.591	39.390
RF1	-30942.766	11003.789	44.252
RF2	-30952.848	11008.488	44.226
RF3	-30961.570	11012.549	44.252
RF4	-30944.950	11009.883	46.005
STNLNN	-30993.741	10945.983	45.472
STNRNN	-30716.297	11277.904	52.307
SV4.4	-30941.475	11008.515	45.907
SV60	-30905.823	11051.168	46.104
SV120	-30867.335	11097.214	46.133
SV180	-30828.857	11143.250	46.144
SV240	-30790.378	11189.285	46.158
SV300	-30751.892	11235.321	46.169
L1	-30955.493	10974.403	42.323
L3	-30960.363	10964.046	42.335
R2	-30950.544	10985.329	42.347
R3	-30945.517	10996.024	42.341
WTL	-30739.168	11224.808	39.798

438/30

+ 10800 X



+ 11000 X

DRG. No.	REFERENCE DRAWINGS

WATER LEVEL = 39,80 m
ON 06·01·2005
(GAUGE PLATE READING = 7,51 m)

VALVE CHAMBER DATUM
LEVEL = 30,637 m (Stud in Floor)

NOTE:

1. DETAILS OF EARTH EMBANKMENT OBTAINED FROM KANTEY & TEMPLER/ NINHAM SHAND DRAWING No. 4601 CT 3Z
2. WATER LEVEL: 39.80m; GAUGE READING: 7.51m (6 JANUARY 2005)

REV	DESCRIPTION	DATE	CHKD	APPRV
Z	RECORD DRAWING	OCT. 2004		
1				
0	ISEUED FOR CONSTRUCTION	APR. 2002		

DESIGNED	CW	
DRAWN	NJ	
CHECKED	JK	
PROJ. ENGINEER	KT	
PROJ. MANAGER	KT	

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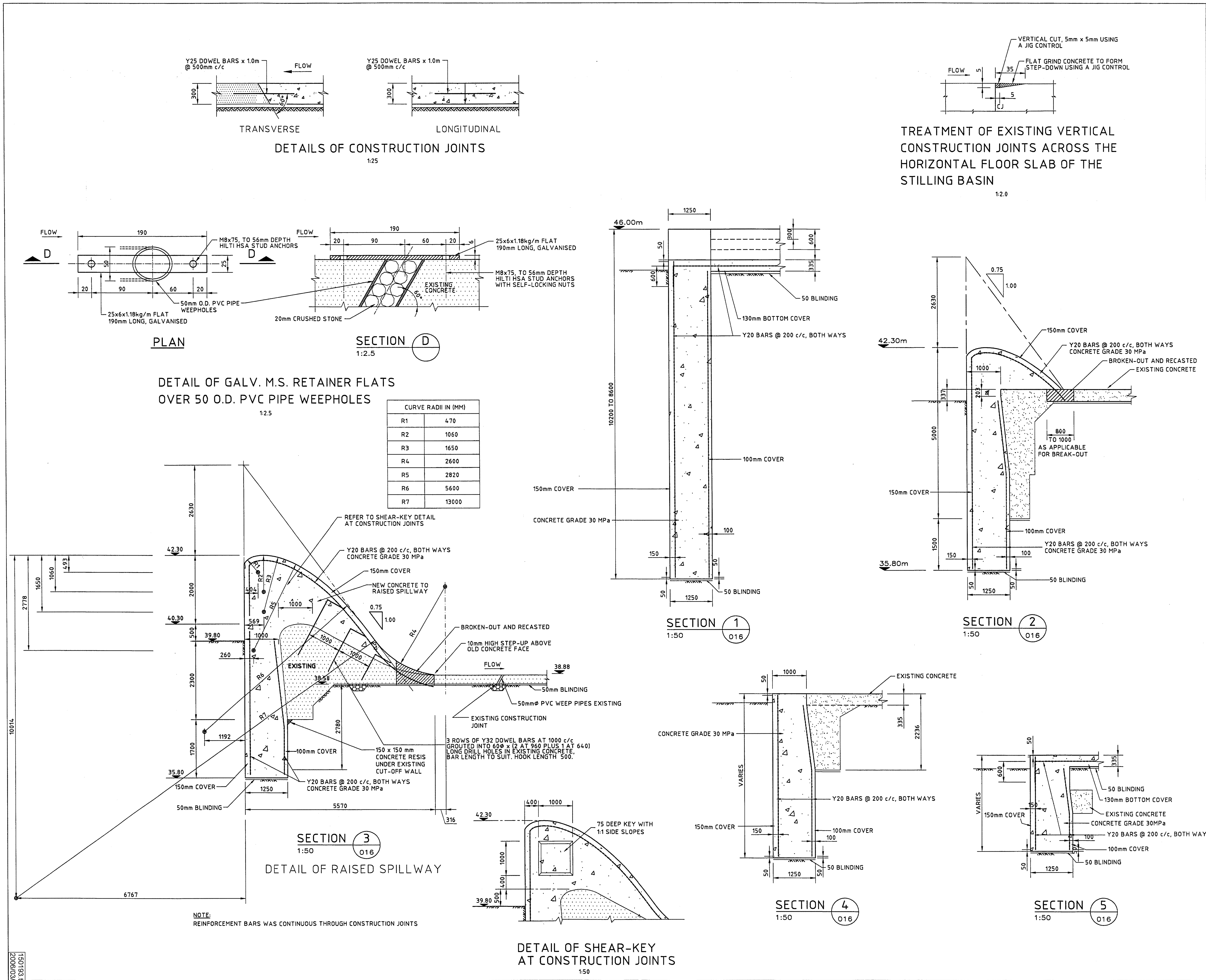
6570
Tel: (044) 382 5293
Fax: (044) 382 6577
Email: knysna@kny.ssi.co.za

APPROVED:

DATE: JUNE 2003



CURVE RADII IN (MM)	
R1	470
R2	1060
R3	1650
R4	2600
R5	2820
R6	5600
R7	13000

DRG. No.	REFERENCE DRAWINGS

REV	DESCRIPTION	DATE	CHKD	APPRV

DESIGNED	CW	01/10/2004
DRAWN	NJ	01/10/2004
CHECKED	JK	01/10/2004
PROJ. ENGINEER	KT	01/10/2004
PROJ. MANAGER	KT	01/10/2004

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6570
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Fax: (044) 382 6577
Email: ssikny@mweb.co.za

APPROVED: DATE

SCALES RELATE TO A SIZE A1 DRAWING

/APP. OCTOBER 2004

PROJECT/DRAWING TITLE

RODEFONTEN DAM
PLETTENBERG BAY

SPILLWAY RAISING,

SPILLWAY SECTIONS AND DETAILS

SCALE: AS SHOWN | SHT. No. 1 OF 1

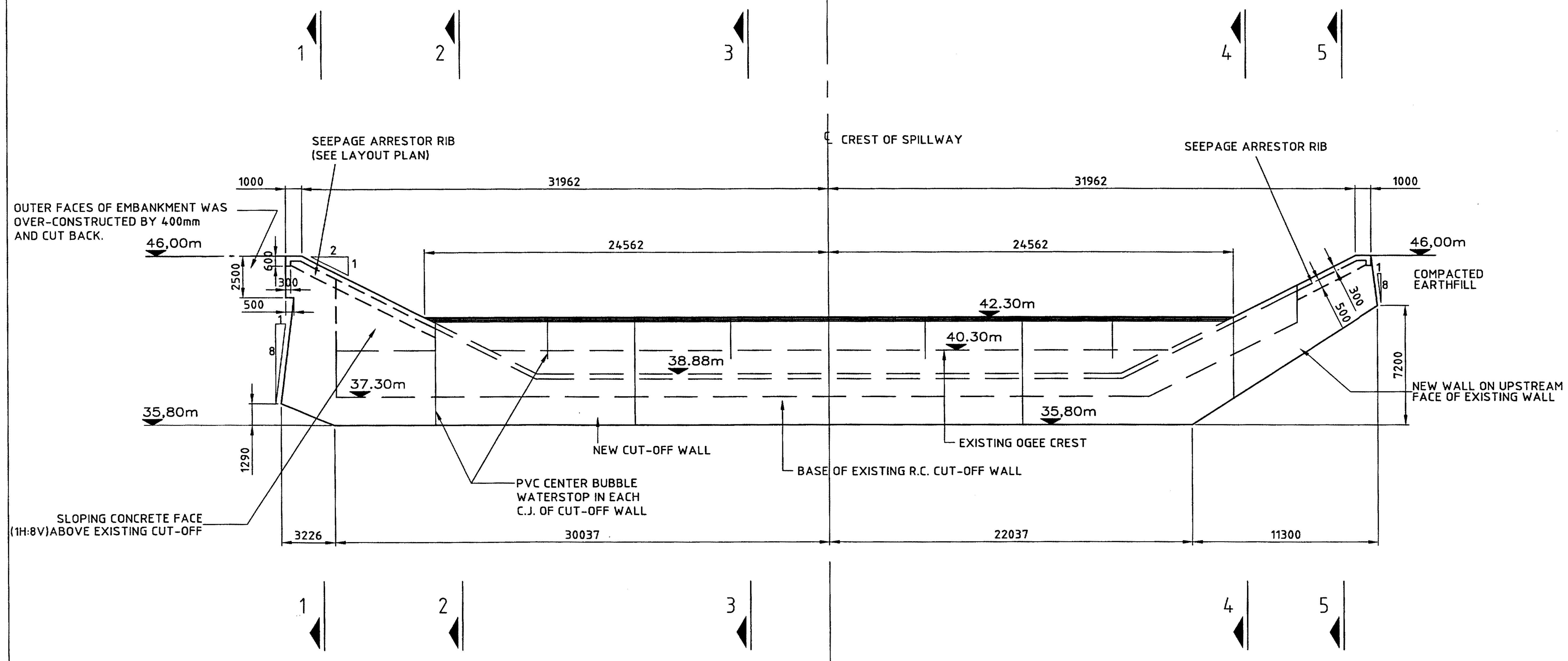
CONTRACT No. | PROJECT No. WA166401

DRAWING No. WA166401/017 | REV △

DRAWING SIZE : A1 PLOT SCALE : | DATE PLOTTED : | CAD REF :

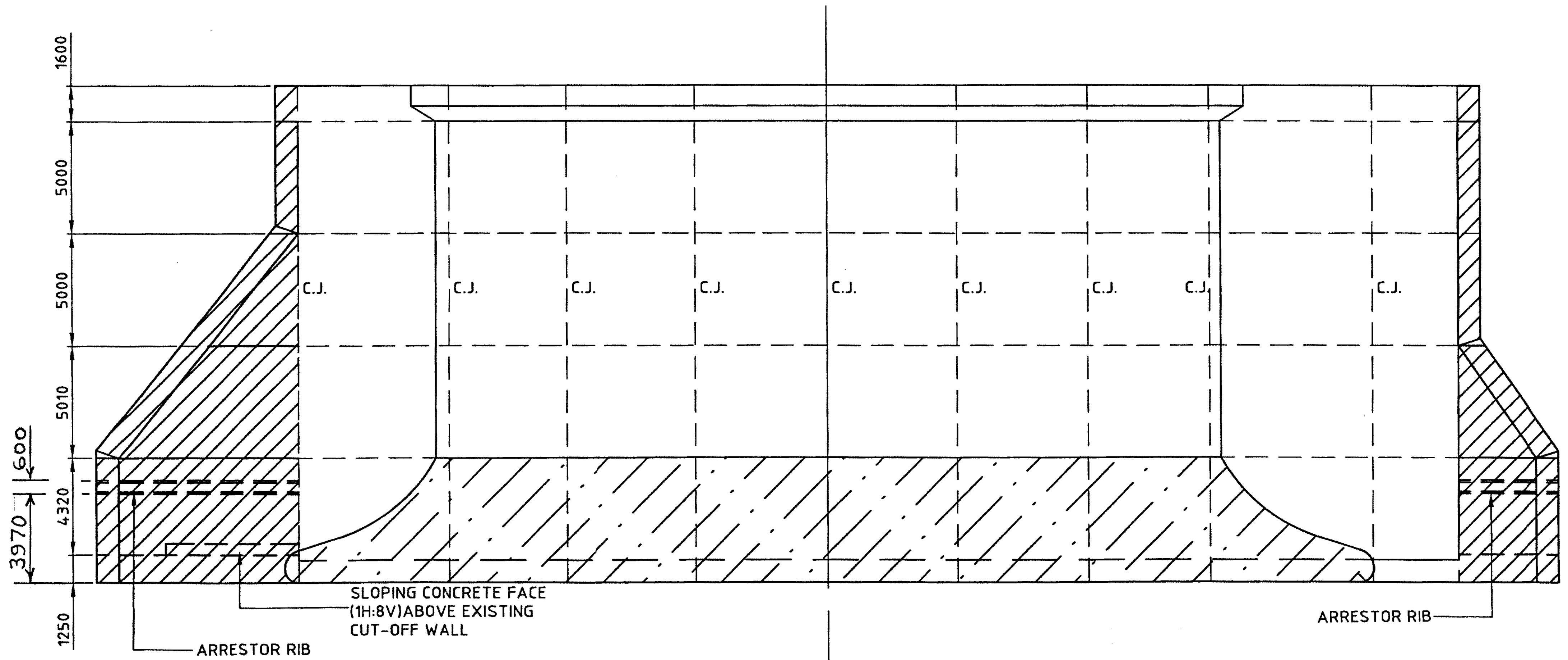
150193.tif
2006/03/14

DRG. No.	REFERENCE DRAWINGS
WA166401/017	SPILLWAY SECTIONS & DETAILS
WA166401/018	SPILLWAY SECTIONS & DETAILS



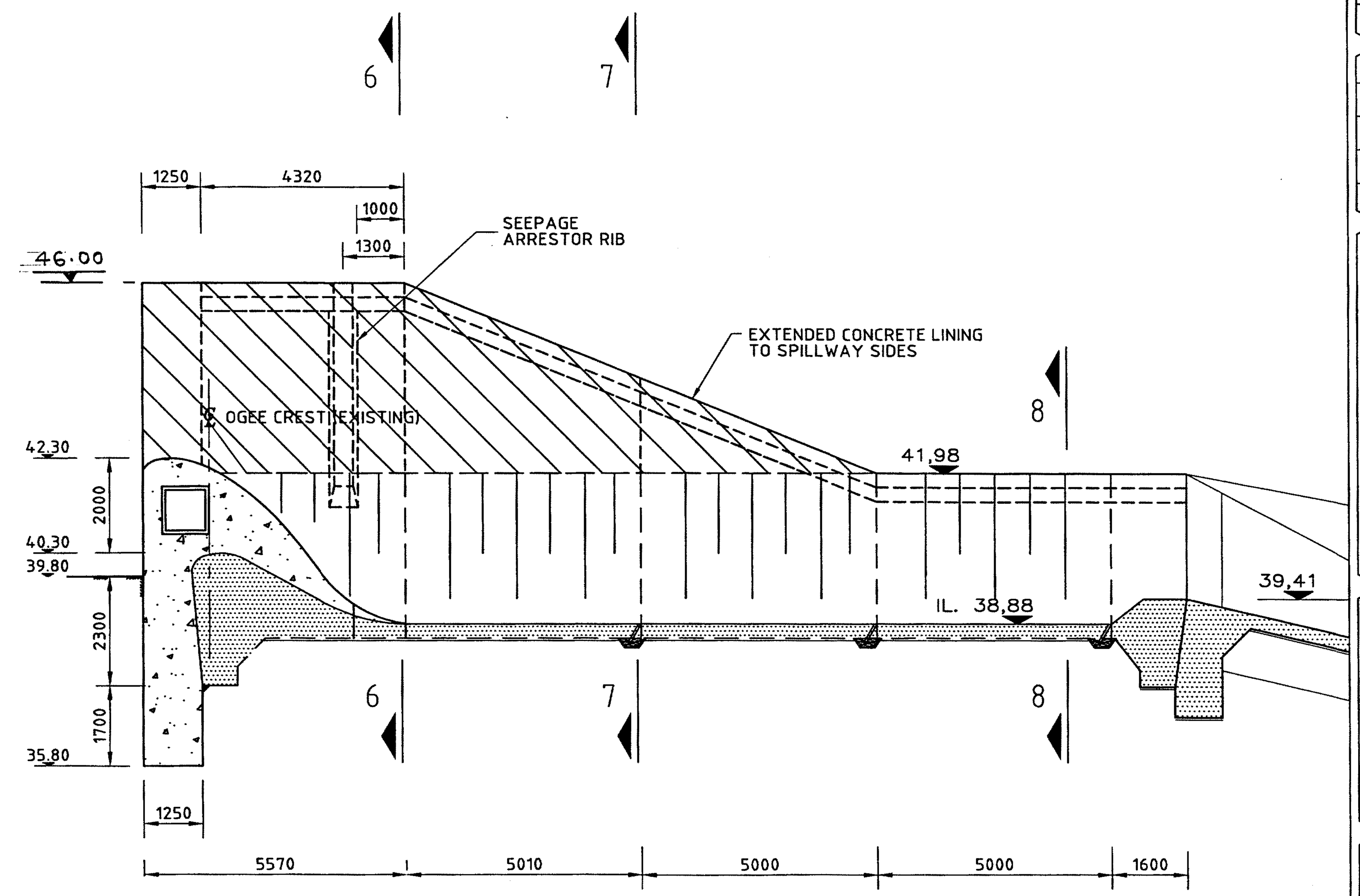
ELEVATION OF SPILLWAY LOOKING DOWNSTREAM

SCALE 1:200



PLAN VIEW OF SPILLWAY

SCALE 1:200



LONGITUDINAL SECTION ALONG THE CENTRE LINE

SCALE 1:100

REV	DESCRIPTION	DATE	CHKD	APPRV
0	ISSUED FOR CONSTRUCTION	APR. 2002		

DESIGNED	CW	01/10/2004
DRAWN	NJ	01/10/2004
CHECKED	JK	01/10/2004
PROJ. ENGINEER	KT	01/10/2004
PROJ. MANAGER	KT	01/10/2004

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Fax: (044) 382 6577
Email: ssikny@mweb.co.za

APPROVED: DATE

SCALES RELATE TO A SIZE A1 DRAWING

/APP. OCTOBER 2004

PROJECT/DRAWING TITLE

ROODEFONTEN DAM PLETTENBERG BAY

SPILLWAY RAISING,

SPILLWAY ELEVATION AND LONGITUDINAL SECTION

SCALE AS SHOWN | SHT. No. 1 OF 1

CONTRACT No. | PROJECT No. WA166401

DRAWING No. WA166401/016 REV 0

150174.tif
2006/03/14

Reg. No. 150174/06

K602/02

DRG. No.	REFERENCE DRAWINGS
REFERENCE DRAWINGS	

REV	DESCRIPTION	DATE	CHKD	APPRV
REVISIONS				

DESIGNED	KT	03/2000
DRAWN	FB	03/2000
CHECKED	HE	03/2000
PROJ. ENG.	HE	03/2000
PROJ. MANAGER	AT	03/2000
APPROVED :		

STEWART SCOTT
PO Box 925
PORT ELIZABETH
6000
Tel: (041) 373-9300
Fax: (041) 373-9302
Email: ssi@pl.ssi.co.za

MUNICIPALITY
OF
PLETTENBERG BAY

PROJECT/DRAWING TITLE

ROODEFONTEIN DRAW-OFF WORKS

DETAILS

SCALE AS SHOWN	SHT. No. 1 OF 1
CONTRACT No.	PROJECT No. WA155708
DRAWING No. WA155708/T/02	REV B

100 mm

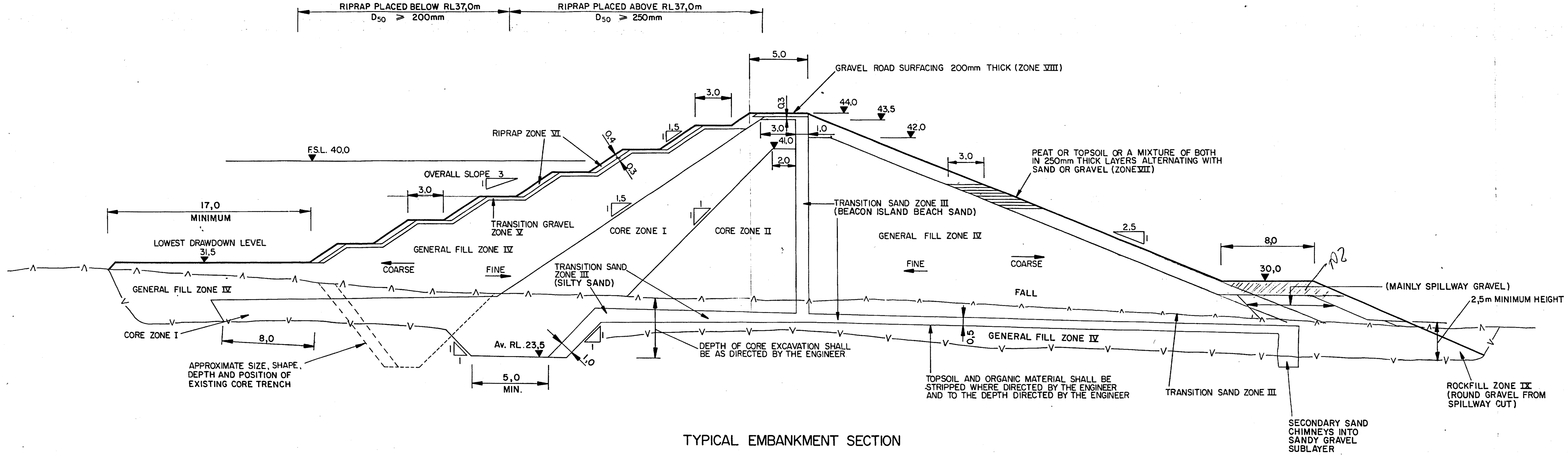
LEGEND:

—^— EXISTING GROUND PROFILE

—v— APPROXIMATE EXCAVATION PROFILE

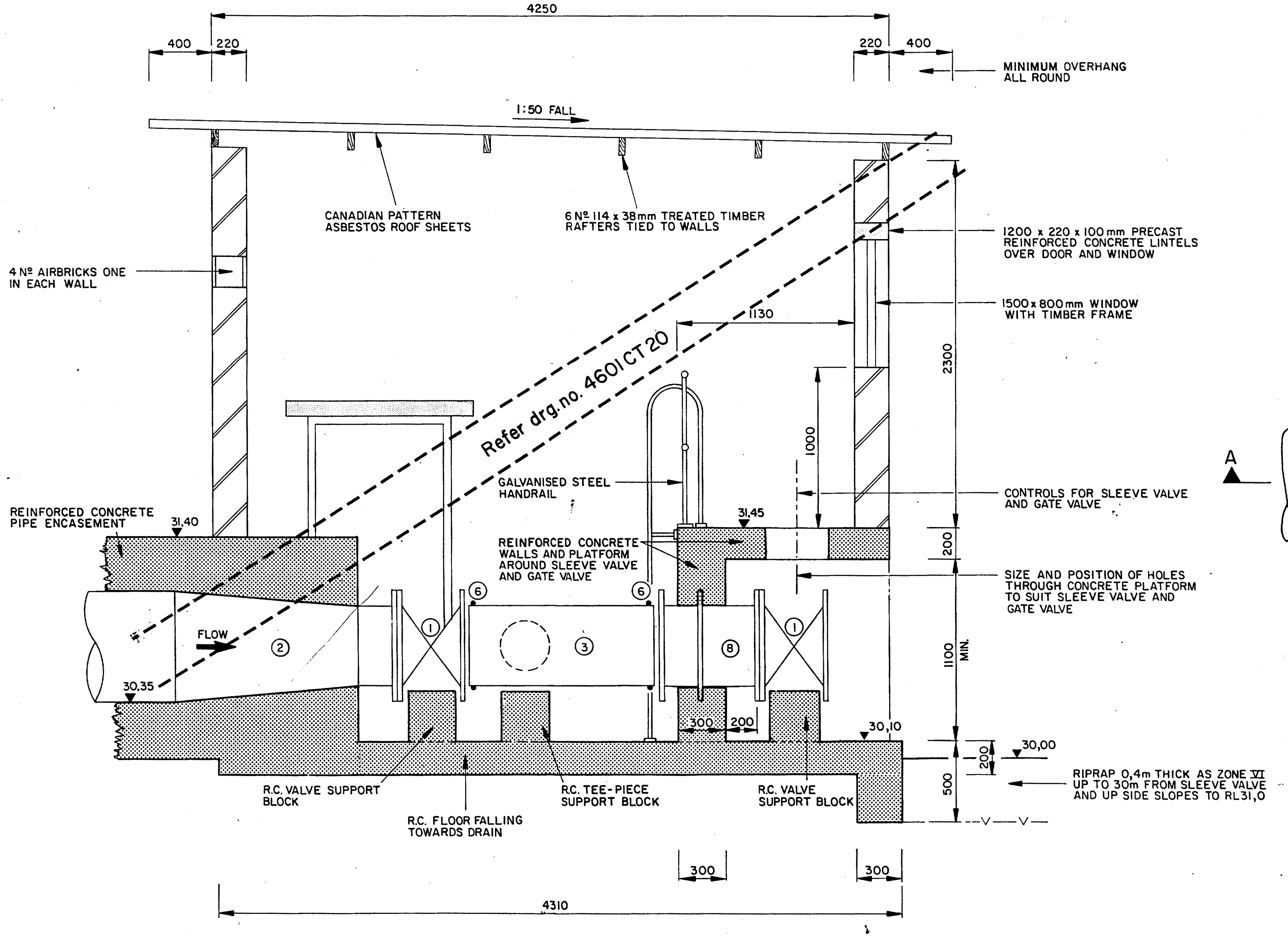
PIPE CODE

1. 500 mm DIAMETER GATE VALVE WITH ELECTRIC AND MANUAL CONTROL (2NO.)
2. 700 mm TO 500 mm DIAMETER STEEL REDUCER.
3. 500 mm DIAMETER STEEL PIPE APPROXIMATELY 1200 mm LONG WITH 500 x 300 mm TEE AND 300 x 300 mm TEE AS SHOWN.
4. 300 mm DIAMETER SLEEVE VALVE WITH ELECTRIC AND MANUAL ACTUATORS.
5. BLANK FLANGE TO FIT 300 mm DIAMETER PIPE.
6. FLEXIBLE 500 mm DIAMETER VIKING JOHNSON COUPLING.
7. FLEXIBLE 300 mm DIAMETER VIKING JOHNSON COUPLING.
8. 600 mm LONG 500 mm DIAMETER STEEL PIPE WITH PUDDLE FLANGE.
9. 600 mm LONG 300 mm DIAMETER STEEL PIPE WITH PUDDLE FLANGE.



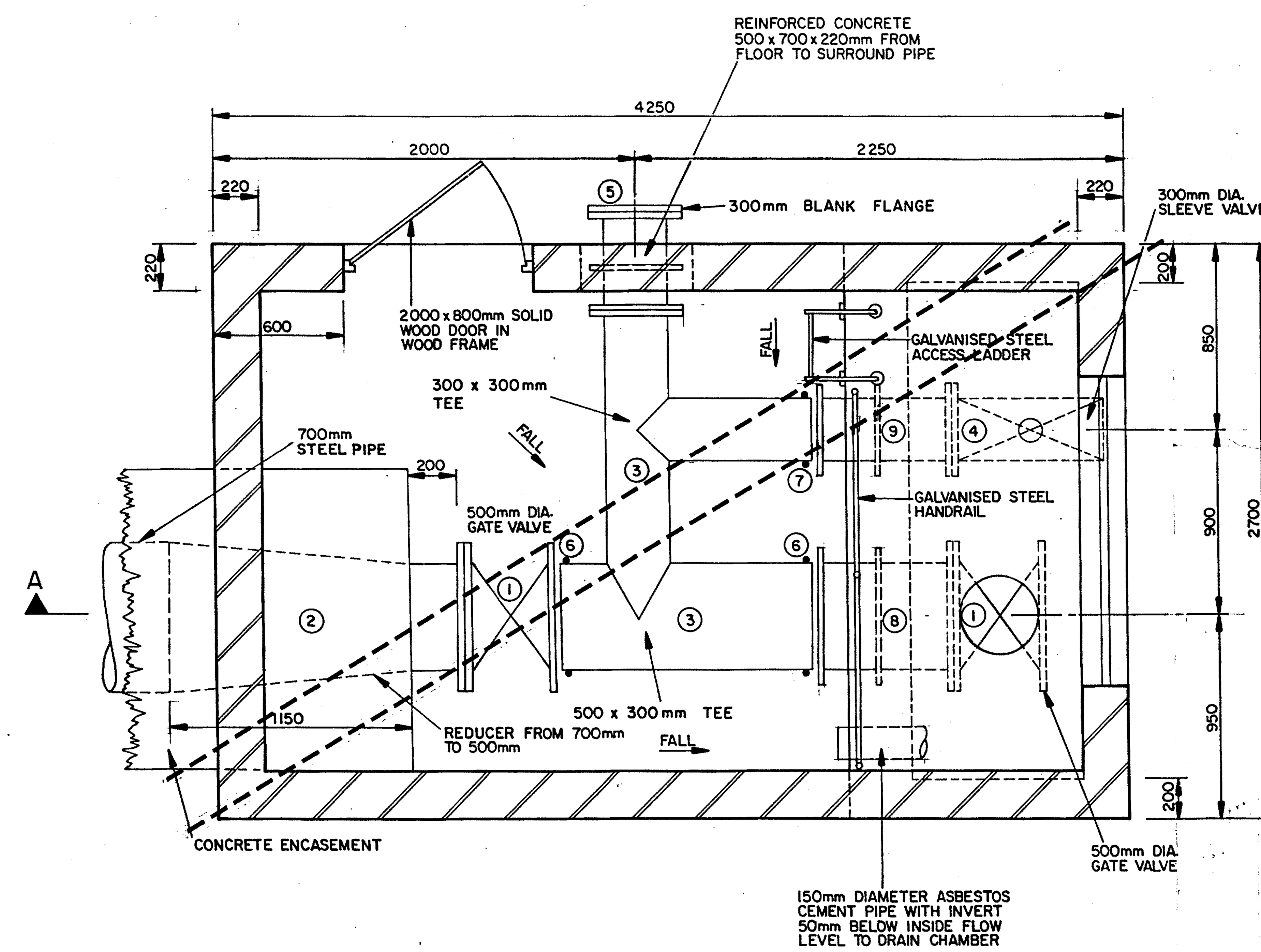
TYPICAL EMBANKMENT SECTION

SCALE 1:200



SECTION A-A

SCALE 1:20



PLAN

SCALE 1:20

No.	AMENDMENTS	APPR.	DATE
Z	RECORD DRAWING	[signature]	7/89
D	VALVE CHAMBER CANCELLED	[signature]	10/88
C	CONCRETE AROUND VALVES AMENDED	[signature]	3/88
	REV B USED FOR TENDER		12/87
B	OUTLET REVISED CORE EXTENDED UPSTREAM	R.C.W.	11/87
A	VALVE CHAMBER REDRAWN & BERM ADDED DOWNSTREAM	R.C.W.	7/87

SURVEYED K & T	BOOK No.
PLOTTED	SURVEY CHECKED K & T
DRAWN H.E.G.	DRAWING CHECKED R.C.W.
DESIGNED R.C.W.	DESIGN CHECKED W.E.P.

RCWakelin Pr Eng
PROJECT ENGINEER
DATE 87-05-08

FOR [signature] Pr Eng
NINHAM SHAND Inc.
DATE 87-05-08

K&T KANTEY & TEMPLER CONSULTING ENGINEERS
IN ASSOCIATION WITH NINHAM SHAND CONSULTING ENGINEERS

THE GARDINER AND ROSS GROUP

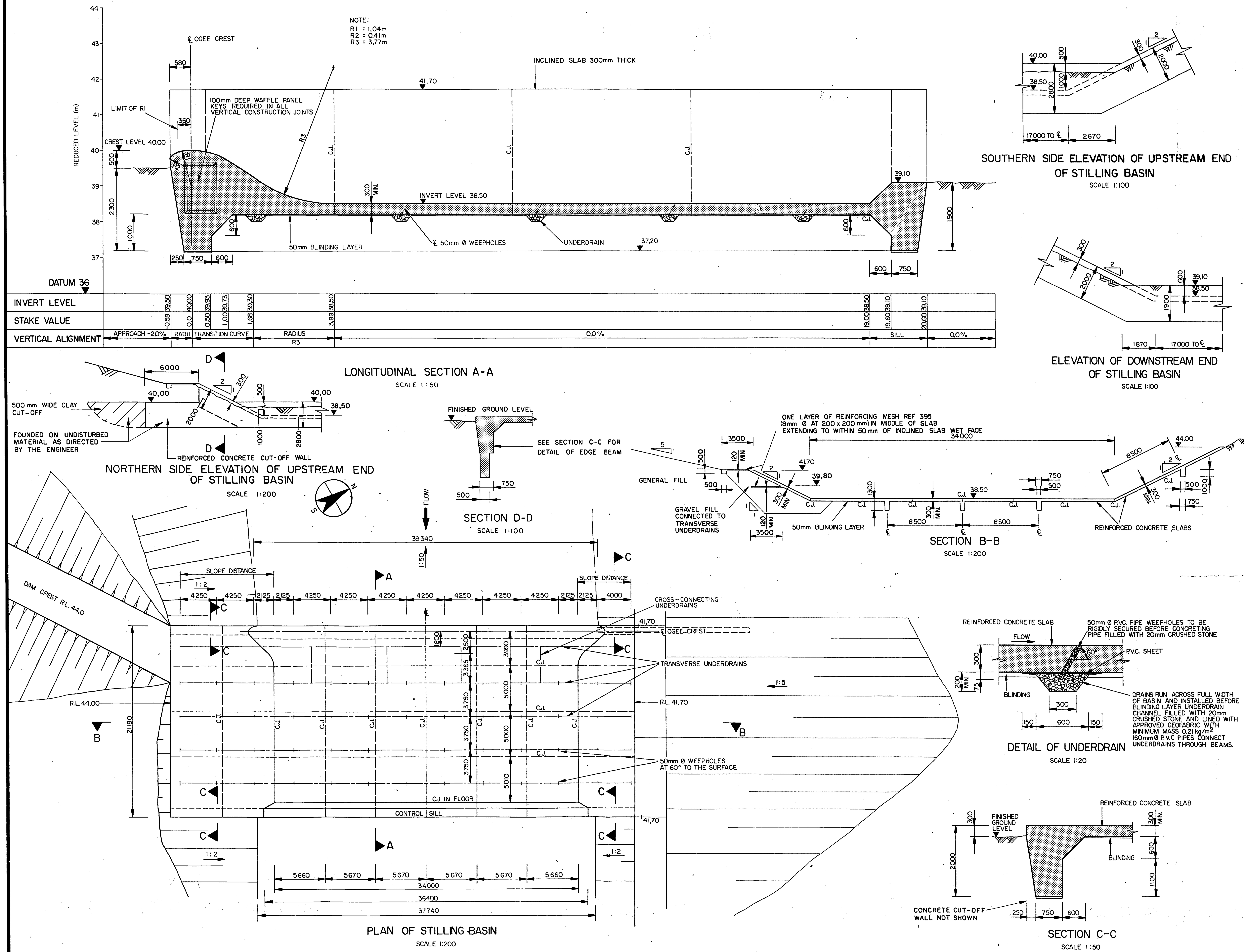
ROODEFONTEIN DAM
PLETTENBERG BAY

EMBANKMENT SECTION AND DOWNSTREAM VALVE CHAMBER DETAILS

SCALE AS SHOWN | SHEET OF SHEETS

DRAWING No.
4601 CT 3 Z

Reg. No 150197/xx



100 mm

NOTE:

1. C.J. = CONSTRUCTION JOINT
2. CONCRETE MIX
   : STRENGTH CONCRETE 30 MPa/40 mm
3. FORMWORK
   : SMOOTH TO ALL SURFACES
4. CONCRETE FINISH
   : WOOD FLOAT TO UNFORMED SURFACES
   : STEEL FLOAT TO ROUNDED SPILLWAY OVERFLOW CREST
5. CONCRETE TO BE CAST AGAINST TRIMMED EXCAVATION.

No.	AMENDMENTS	APPR.	DATE
Z	RECORD DRAWING		7/89
B	L.H.S. OF STILLING BASIN		2/89
A	CUT SLOPE DETAILS ON BB AND MINOR EXTRA NOTES	RCW	7/87

SURVEYED		BOOK No.	
PLOTTED		SURVEY CHECKED	
DRAWN	H.E.G.	DRAWING CHECKED	D.R.M.
DESIGNED	D.R.M.	DESIGN CHECKED	CVS & MJS

PROJECT ENGINEER Pr Eng
DATE 8/5/87.

FOR NINHAM SHAND Inc. Pr Eng
DATE 87-05-08

K&T KANTEY & TEMPLER CONSULTING ENGINEERS
IN ASSOCIATION WITH NINHAM SHAND CONSULTING ENGINEERS

THE GARDINER AND ROSS GROUP

ROODEFONTEIN DAM
PLETTENBERG BAY

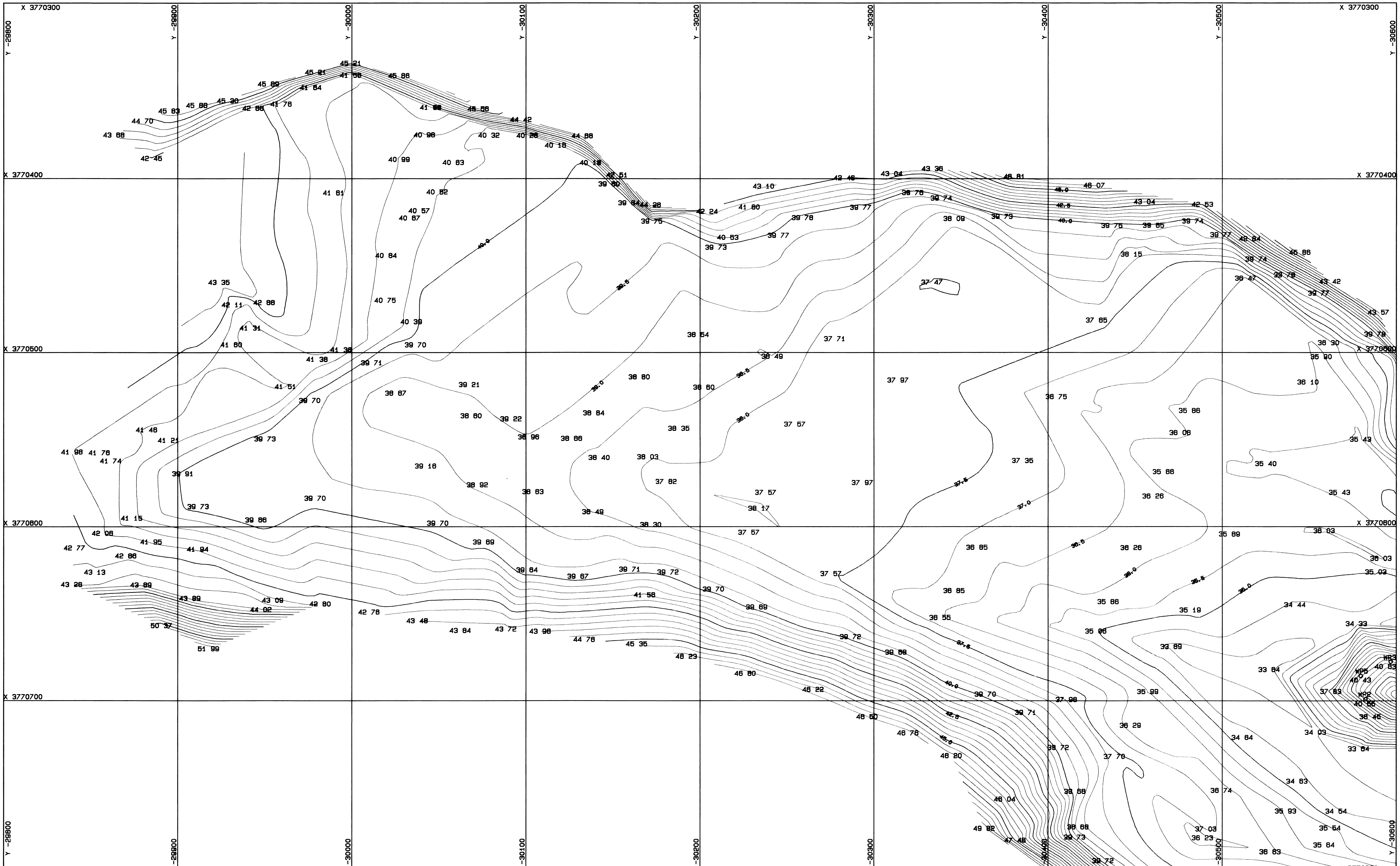
SPILLWAY

DETAILS OF STILLING BASIN

SCALE	SHEET OF SHEETS
AS SHOWN	

DRAWING No.
4601 CT 7Z

REG. NO. 150199/06



KOÖRDINAATSTELSEL LO.23°			
H.H. VANAF T114	144.9 METERS BO SEESPIEEL		
STASIE	Y	X	H.H.

CO-ORDINATE SYSTEM LO. °			
R.L. FROM	METRES ABOVE M.S.L.		
STATION	Y	X	R.L.

VELUITLEG/SHEET LAYOUT

1 2

N

0 10 20 40 60 80 100 120 140 m

SKAAL/SCALE 1 : 1000

DIREKTORAAT OPMETINGSDIENSTE — DIRECTORATE SURVEY SERVICES

PRIVAATSAK X313 PRIVATE BAG
PRETORIA
0001

OPGEMEET SURVEYED	WORC. REGION	4/98	NAGETREK TRACED	S NIEUWENHUIZEN	5/98
SAAMGESTEL COMPILED	E NIEUWENHUIZEN	4/98	TOPOGRAFIE NAGESIEN TOPOGRAPHY CHECKED		
FOTOGRAFIE PHOTOGRAPHY			KADASTRAAL NAGESIEN CADASTRAL CHECKED		

Nr./No	HERSIENING / REVISION		DATUM DATE

REPUBLIEK VAN SUID-AFRIKA
DEPARTEMENT VAN WATERWESE EN BOSBOU

REPUBLIC OF SOUTH AFRICA
DEPARTMENT OF WATER AFFAIRS & FORESTRY

PIESANGRIVIER — PIESANG RIVER

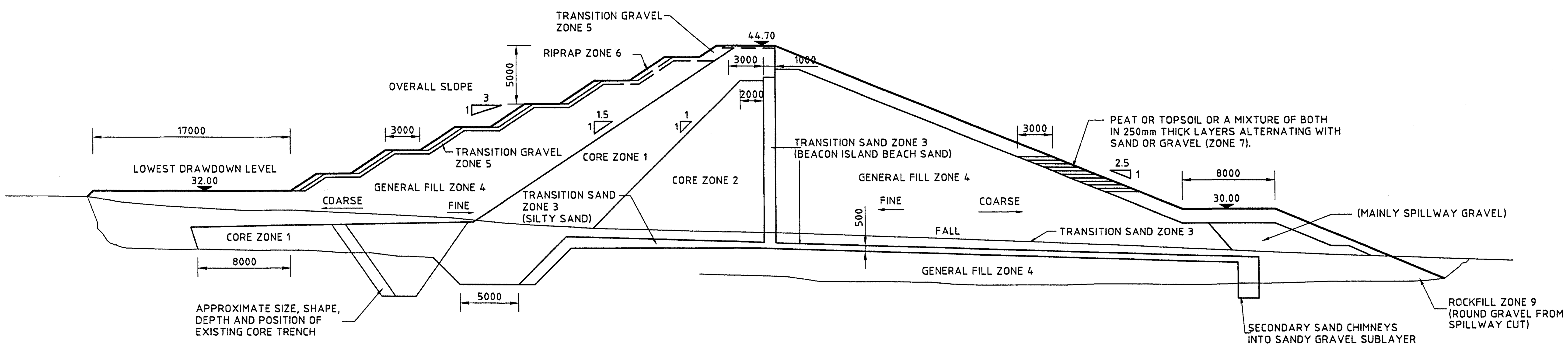
ROODEFONTEINDAM
KAPASITEITSOPMETING

ROODEFONTEIN DAM
CAPACITY SURVEY

DISTRIK DISTRICT	PLETTENBERGBAAI	VEL SHEET 1 OF 2	LOK. NR./LOC. NO.	REG. NR./REG. NO.
AREA	KODE CODE	CON	K602/02	~~KWS 22027~~

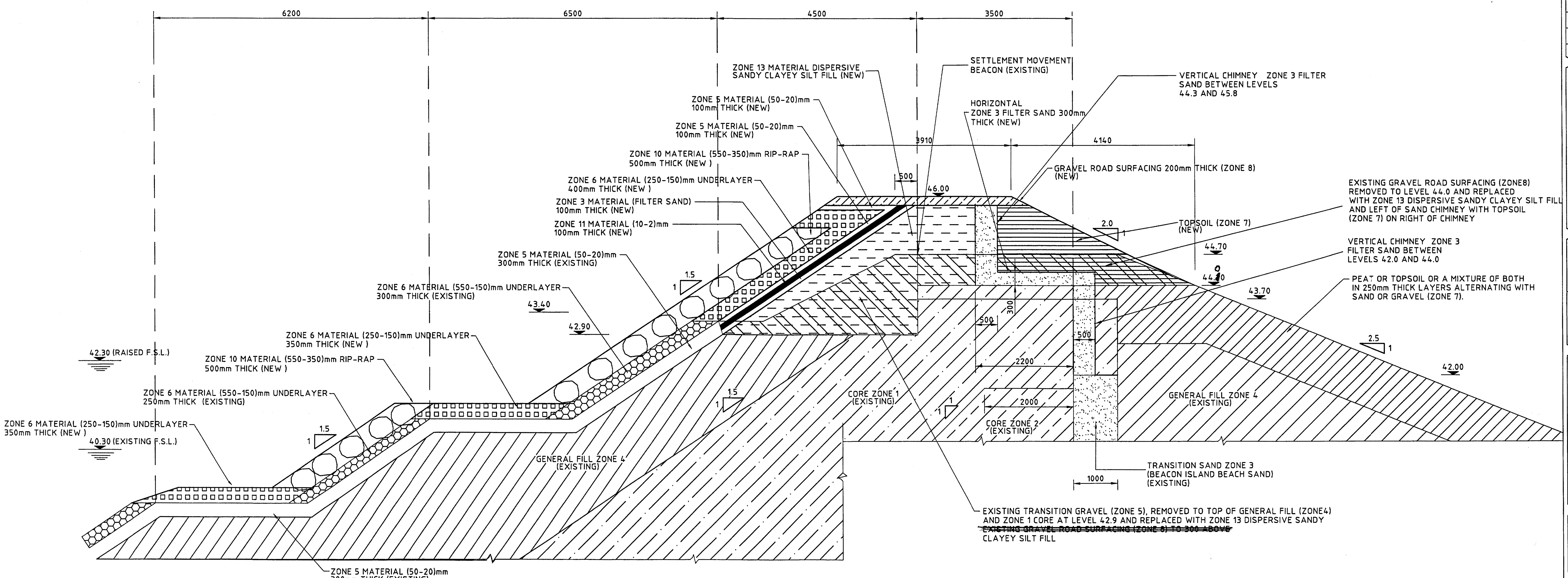
132418/99

150195.tif 2006/03/14



TYPICAL EMBANKMENT SECTION - EXISTING

SCALE 1:250



NOTE:
100mm THICK NEW ZONE 5 MATERIAL OVERLIES 100mm THICK NEW ZONE 11 MATERIAL WHICH OVERLIES 100mm THICK NEW ZONE 3 MATERIAL AS UPSTREAM FILTER FOR DISPERSIVE NEW ZONE 13 MATERIAL (SANDY CLAYEY SILT).

TYPICAL EMBANKMENT SECTION - PROPOSED ALTERNATIVE

SCALE 1:50

DRG. No.	REFERENCE DRAWINGS
WA166401/015	SPILLWAY RAISING LAYOUT
WA166401/016	SPILLWAY ELEVATION & LONG. SECTION
WA166401/017	SPILLWAY SECTIONS & DETAILS
WA166401/018	SPILLWAY SECTIONS & DETAILS

NOTE:
DETAILS OF EARTH EMBANKMENT OBTAINED FROM KANTEY & TEMPLER/ NINHAM SHAND DRAWING No. 4601 CT 3Z

REV	DESCRIPTION	DATE	CHKD	APPRV
1	VERT. FILTER TO 500 THICK	JUN. 2003		
0	ISSUED FOR CONSTRUCTION	APR. 2002		

DESIGNED	CW	01/10/2004
DRAWN	NJ	01/10/2004
CHECKED	JK	01/10/2004
PROJ. ENGINEER	KT	01/10/2004
PROJ. MANAGER	KT	01/10/2004

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6570
Tel: (044) 382 5293
Fax: (044) 382 6577
Email: ssikny@mweb.co.za

APPROVED: DATE

SCALES RELATE TO A SIZE A1 DRAWING

/APP. OCTOBER 2004

PROJECT/DRAWING TITLE
RODEFONTEN DAM PLETTENBERG BAY
ALTERNATIVE EMBANKMENT RAISING,
ROCK RIP-RAP REVETMENT

SCALE	SHT. No. 1 OF 1
AS SHOWN	
CONTRACT No.	PROJECT No. WA166401

DRAWING No. WA166401/020 REV

Reg. no. 150195/06

DRAWING SIZE : A1 PLOT SCALE : DATE PLOTTED : CAD REF :

Addendum C: Geology

(15)

Council for Geoscience

Private Bag X112 Pretoria 0001 SOUTH AFRICA 280 Pretoria Street Silverton Pretoria
Reception: +27 (0)12 841 1911 Internet: http://www.geoscience.org.za

Reference: K602/02
Enquiries: GN Davis
Tel: 012 336 7519
Fax: 012 336 8050
E-mail: gdavis@geoscience.org.za

24 March 2006

DEPARTMENT OF WATER AFFAIRS
AND FORESTRY
P/BAG X313
PRETORIA
0001

Chief Engineer: Civil Design Directorate
(Attention: Dr A Bester)

Roodefontein Dam: Engineering Geological Report for Dam Safety Purposes

The latest list of dams for which engineering geological reports for dam safety purposes are required, as received from the Civil Design Directorate, Department of Water Affairs and Forestry, refers.

A dam safety report entitled *"Roodefontein Dam. Plettenberg Bay. Dam Safety Inspection. Geotechnical Report"* by Knight Hall Hendry and Associates (Reference SSI 95 G), dated 11 August 1993, has been submitted previously.

The main findings of this report, as well as other reports as listed below, may be summarized as follows;

* The geological conditions at the Roodefontein Dam appear well documented. Geological investigations were conducted prior to construction and this geological report was finalized after completion. A construction completion report was also compiled.

* The embankment is founded on a succession of alluvial materials, comprising clayey to sandy silt material with gravel layers, underlain by stiff clay and silt derived from weathering of the underlying Kirkwood Formation mudrock strata (*Note, however, the available geological maps indicate the strata as belonging to the Enon Formation*). The core contact is reportedly founded on siltstone across the full width of the valley.

  Note also the alluvium contained peat layers up to 4 m in thickness. While the exposed peat layers were reportedly removed and replaced with selected fill, doubts have been expressed that some of this compressible material could have remained beneath the embankment (Knight Hall Hendry, 1993).

* Major slope instability had occurred during construction on the northern slope flanking the spillway excavation, and failures were expected to continue. Detailed analysis concluded the slope is marginally stable with a minimum FOS of 1 (Kantey & Templer, 1996).

* Two areas of seepage were noted, namely 1) an area of stagnant water at the downstream toe, and 2) wet conditions at the base of the cut slope behind the valve chamber.

Additional geological / geotechnical information which has become available since the above-mentioned dam safety report includes the following;

- A final geotechnical report was compiled after repairs and rehabilitation of the spillway following erosion damage (Kantey & Templer, 1996[1])
- Results of an investigation into the seismic risk and the effects of a possible earthquake on the dam (Ninham Shand, 1996[2]). For the analysis a MCE was assumed to equal a peak ground acceleration of 0.07g. The report concluded the overall stability of the structure was unlikely to be affected, although minor settlement could occur.
- Recently published seismic hazard maps[3], however, indicate a Peak Ground Acceleration (PGA) of approximately 0.05g, with a 10% probability of being exceeded in a 50-year period. This might be considered a low level of seismic hazard.

Roodefontein Dam was visited by the undersigned, in the company of Mr Muvhuso Musetsho of DWAF, on 16th March 2006. At the time the reservoir level was very low (gauge plate reading 0.95 m). Observations from this site inspection, conducted in intermittent rain, include the following;

- A longitudinal crack is present at the break in slope immediately to the north of the spillway (Plate 1). It is not certain whether this is a new phenomenon, or is that referred to by Kantey & Templer (1996). Minor scarps just visible in the heavily wooded northern slopes do not appear to be new features and are considered to represent earlier slope movements. It is not known whether the installed inclinometer is still operational.
- Reed growth along the embankment toe indicates a measure of seepage, although there was no visible flow. The previously-noted area of seepage behind the valve chamber is still present; this in spite of the low reservoir level. It might be concluded that the seepage is not derived from the reservoir itself but rather from the right flank.
- Although trees growing on the embankment have been felled in the past, dense infestations of Port Jackson are again springing up – along the crest as well as on the downstream face near the spillway.
- There is no evidence of any erosion downstream of the concrete-lined chute. However, it is not known whether the dam has spilled since rehabilitation of the spillway.
- Of concern are a number of sub-parallel horizontal cracks near the crest of the downstream face of the embankment (Plate 2). Although partly obscured by the dense growth of Kikuyu grass, these cracks are up to 100 mm wide at surface and

---

[1] Kantey & Templer. 1996. Roodefontein Dam: Rehabilitation of Spillway. Final Geotechnical Report. Report No G5335 to Provincial Administration Western Cape. Community Services Branch.
[2] Ninham Shand (Cape). March 1996. Roodefontein Dam: The Effect of an Earthquake on the Dam. Report to Provincial Administration Western Cape. Community Services Branch. Report No 2488/4601.
[3] Kijko, A, Graham, G, Bejaichund, M, Roblin, D & Brandt, MBC. 2003. Probabilistic Seismic Hazard Maps for South Africa. Council for Geoscience.

might be traced for a significant length along the face (Plate 3). It is surmised that the origin of these cracks is linked to material placement and / or compaction during construction. These cracks suggest there has been some degree of downslope movement. At present the cracks provide for enhanced water ingress into the downstream face of the embankment. Note that the thick grass cover precluded detailed observation of the depth of these cracks, and it is not certain to what extent there has been any associated downslope movement. It might also be noted that the slope of the downstream face is estimated at approximately 30° (1:1.7), while available sections indicate a flatter gradient of approximately 22° (1:2.5).

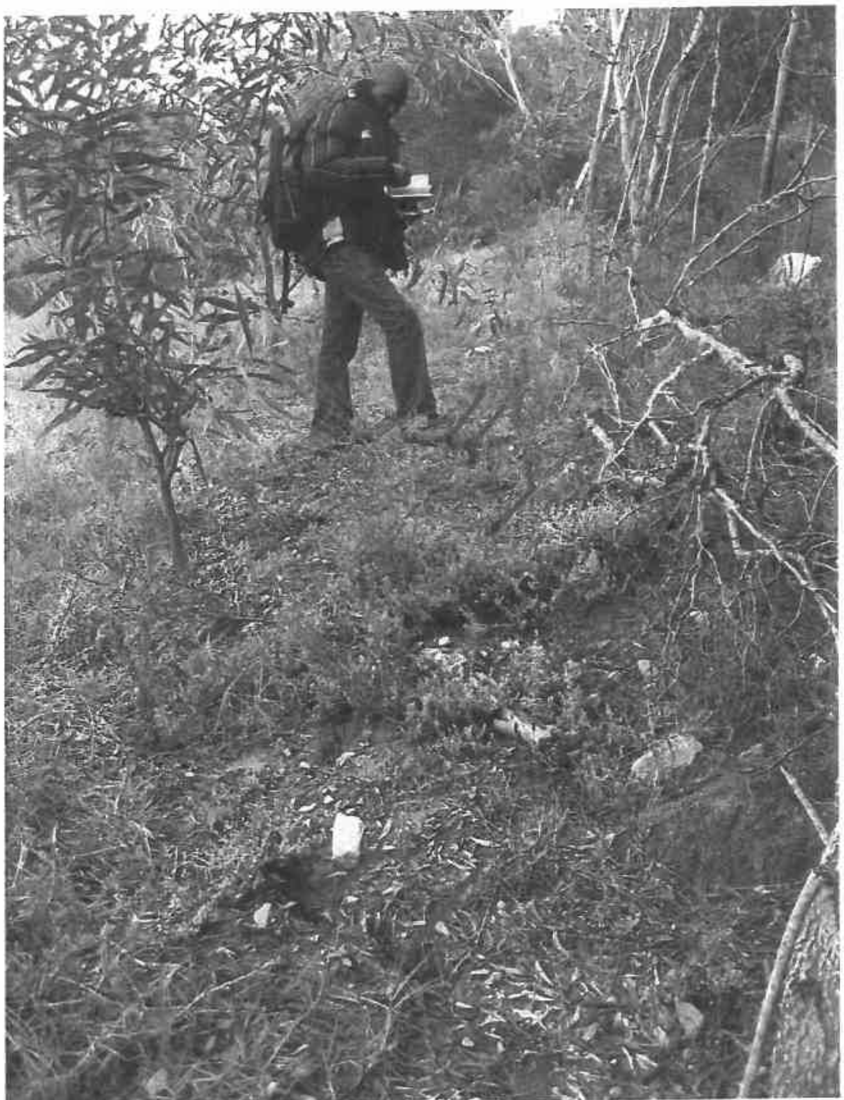

Plate 1: The longitudinal crack (indicated by the red lines) at the break in slope immediately to the north of the spillway, indicating some movement in the slope.

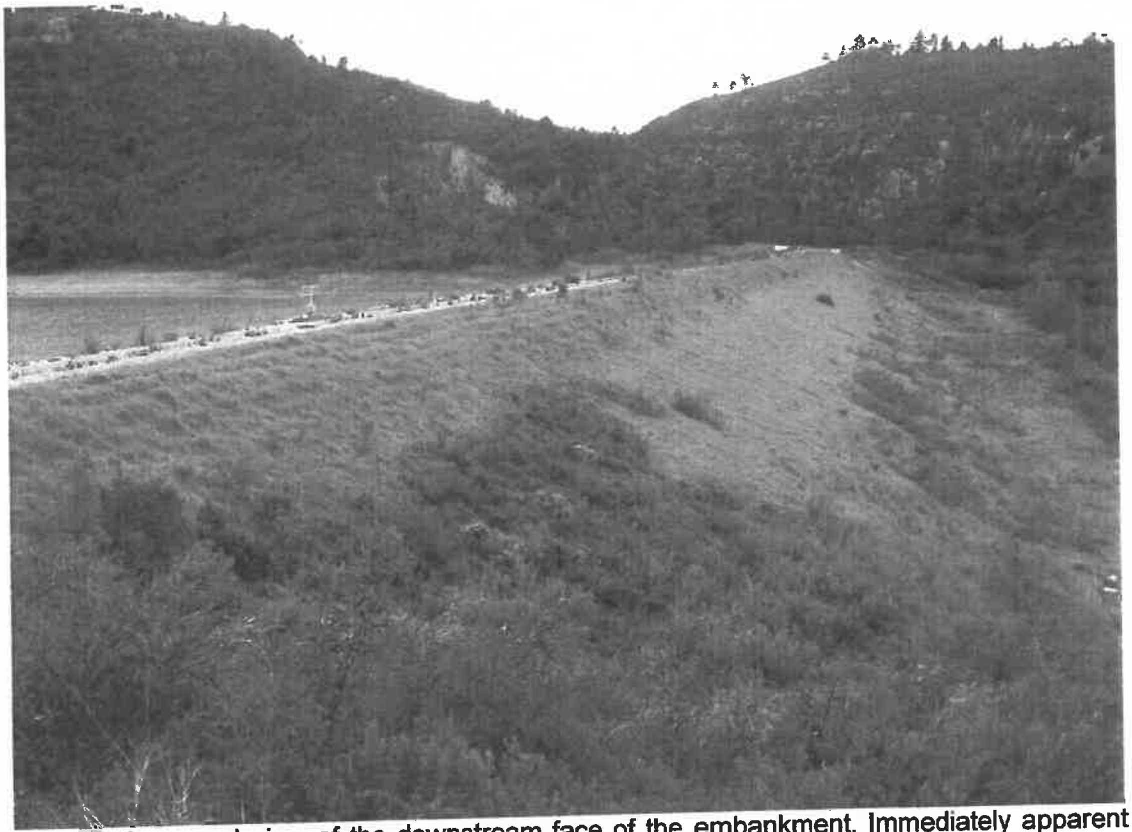

Plate 2: A general view of the downstream face of the embankment. Immediately apparent are the horizontal lines visible near the crest which closer inspection reveals to be prominent cracking.

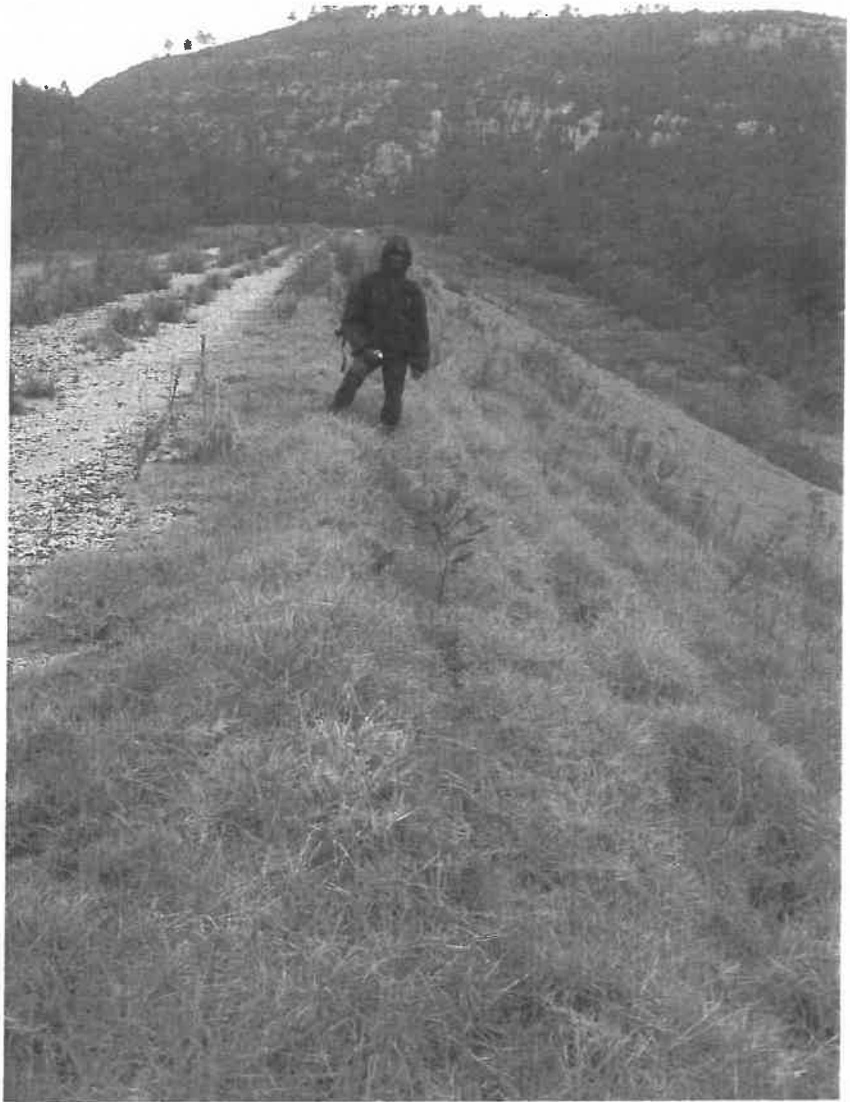

Plate 3: A closer view of the prominent cracking. Although largely hidden by the dense grass cover, at surface these cracks are almost 100 mm in width.

It is recommended that the longitudinal cracks on the downstream face be investigated more closely, including the embankment geometry and confirmation whether downslope movement is occurring. This would necessitate that the grass be cut short so as to enable detailed observation and surveying.

GN Davis
Manager: Engineering Geosciences

Addendum D: Hydrology Report

water & sanitation
Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

REPORT NO.: K600-R001-2018.02

FLOOD STUDIES

FLOOD FREQUENCY ANALYSIS

ESTIMATION OF FLOOD PEAKS FOR REQUIRED PROBABILITIES

Piesang river at

Roodefontein dam

February 2018

DEPARTMENT: WATER AND SANITATION

PROBABILISTIC, DETERMINISTIC AND EMPIRICAL EVALUATION
OF THE CATCHMENT, RAINFALL AND FLOW DATA
FOR THE ESTIMATION OF FLOOD PEAK PROBABILITIES,
FOR ROODEFONTEIN DAM

REPORT DETAILS

Title:	**Flood Frequency Analysis for Roodefontein Dam**
DWS no.:	**K602/02**
Report no.:	**K600-R001-2018.02**
Report status:	**Final**
Date:	**February 2018**
Prepared by:	**Directorate: Surface and Ground Water Information** **Component: Flood Studies**
Prepared for:	**Directorate: Strategic Asset Management** **Sub-directorate: Dam Safety Surveillance**

ENDORSEMENT

Flood Studies team:

P Rademeyer
Analyst and Reporter
Professional Scientist

D van der Spuy
Report Supervisor (APP)
Specialist Engineer

EXECUTIVE SUMMARY

BACKGROUND

The Sub-Directorate: Dam Safety Surveillance requested the Component: Flood Studies to do a flood frequency analysis on Roodefontein dam (K602/02) for dam safety purposes.

Roodefontein dam is situated in the Piesang river about 2.5 km west (as the crow flies) of the town Plettenberg Bay in the Western Cape Province. The dam was mainly constructed for domestic and industrial use.

THE CATCHMENT

Roodefontein dam has a total catchment area of 28 km^2. The catchment receives rainfall throughout the year and the mean annual precipitation was estimated at 930 mm.

The surface hydrology of the catchment consists of various streams of which the longest stream originates ≈280 m.a.s.l. in the most western part of the catchment. The longest stream measures ≈14.80 km in length and the time of concentration for the catchment was estimated at 3.5 hours.

The basic vegetation cover consists mainly of dense- and thin bush with some cultivated land areas. The catchment soils are largely deemed to be very permeable to semi-permeable.

THE DAM

Roodefontein dam was completed in 1989 and raised in 2004. The dam is an earthfill dam with an ogee spillway on the left hand side. The total crest width is 315 m. The dam has a wall height of approximately 18 m and a capacity of 2.069×10^6 m^3 at full supply level 43.91 m RL / 10.07 m GP. The surface area of the lake formed by the dam at FSL is ±37 hectares.

Roodefontein dam is classified as a category 3 dam of medium size with high hazard potential.

RECOMMENDED FLOOD FREQUENCIES

Exceedance probability (%)						
50	**20**	**10**	**5**	**2**	**1**	**0.5**
Flood peaks (m^3/s)						
8	20	30	45	65	80	95

SEF

In accordance with the SANCOLD guidelines, following this site specific analysis, it is recommended that a safety evaluation flood (SEF) equal to 530 m^3/s be considered for this catchment.

INDEX

REPORT

APPENDICES

INDEX OF TABLES AND FIGURES

REPORT

Tables

APPENDIX

Tables

Figures

LIST OF SYMBOLS AND ABBREVIATIONS

A	-	Catchment area (km^2)
ARF	-	Areal reduction factor
CAPA	-	Catchment parameter (2002) method
D	-	Storm duration
DRH	-	Direct Runoff Hydrograph method
DWS	-	Department of Water and Sanitation
EP	-	Exceedance Probability (%)
FSL	-	Full Supply Level (m)
GP	-	Gauge Plate level (m)
HG	-	Hydrograph
HRU 1/71	-	Midgley and Pitman (1971) method
i	-	Rainfall intensity (mm/hour)
K	-	RMF region
K_e	-	Effective Francou-Rodier value
k	-	Storm runoff factor
L	-	Longest watercourse (km)
MAP	-	Mean annual precipitation (mm)
MASL	-	Meters above sea level
MIPI	-	Midgley and Pitman (1972) method
P	-	Catchment rainfall (mm)
P_e	-	Effective catchment rainfall (mm)
Q	-	Flow (m^3/s)
RL	-	Relative Level (m)
RMF	-	Regional Maximum Flood (Kovacs, 1988)
S_A	-	Mean catchment slope
S_L	-	Average slope of longest watercourse
SANCOLD	-	South African National Committee on Large Dams
SAWS	-	South African Weather Services
SCS	-	Soil Conservation Service hydrograph generating technique
SEF	-	Safety Evaluation Flood (SANCOLD)
SRR	-	Smithers Regional Rainfall
SUH	-	Synthetic Unit Hydrograph method
tc	-	Time of concentration (hours)
TR	-	Technical Report (DWS)
WR2005	-	Water Resources of South Africa, 2005 Study
WRC	-	Water Research Commission

1. INTRODUCTION

1.1. Background

The Sub-Directorate: Dam Safety Surveillance requested the Component: Flood Studies to do a flood frequency analysis on Roodefontein dam (K602/02) for dam safety purposes.

Roodefontein dam is situated in the Piesang river about 2.5 km west (as the crow flies) of the town Plettenberg Bay in the Western Cape Province. The dam was mainly constructed for domestic and industrial use.

1.2. The catchment

Roodefontein dam has a total catchment area of 28 km^2. The catchment receives rainfall throughout the year and the mean annual precipitation was estimated at 930 mm.

The surface hydrology of the catchment consists of various streams of which the longest stream originates ≈280 m.a.s.l. in the most western part of the catchment. The longest stream measures ≈14.80 km in length and the time of concentration for the catchment was estimated at 3.5 hours.

The basic vegetation cover consists mainly of dense- and thin bush with some cultivated land areas. The catchment soils are largely deemed to be very permeable to semi-permeable.

Appendix A contains more information on the catchment characteristics.

1.3. The dam

Roodefontein dam was completed in 1989 and raised in 2004. The dam is an earthfill dam with an ogee spillway on the left hand side. The total crest width is 315 m. The dam has a wall height of approximately 18 m and a capacity of 2.069×10^6 m^3 at full supply level 43.91 m RL / 10.07 m GP. The surface area of the lake formed by the dam at FSL is ±37 hectares.

Roodefontein dam is classified as a category 3 dam of medium size with high hazard potential.

2. AVAILABLE INFORMATION

2.1. Results from previous analyses

To facilitate comparison the results of this study are also shown in the table below.

Table 2-1: Results from previous analyses

Report	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Flood peaks (m^3/s)						
Tshehla (2001)	15	25	40	50	70	90	105
Roux (2010)	10	25	35	50	70	90	105
Rademeyer (2018)	8	20	30	45	65	80	95

2.2. Extreme floods

For the period October 1995 to present, the following flood peaks were recorded.

Table 2-2: Flood peaks on record (> 25 m^3/s)

Catchment	Catchment Area (km^2)	Inflow peak (m^3/s)	Outflow peak (m^3/s)	Date
Roodefontein	28	34	25	02/08/2006
		35	33	23/11/2007
		27	24	01/09/2015

3. FLOOD FREQUENCY ANALYSIS

3.1. Introduction

Deterministic, empirical and statistical methods were used in the flood frequency analysis of the Roodefontein dam catchment.

3.2. Catchment rainfall

The catchment rainfall was estimated for different storm durations and exceedance probabilities by using site specific (MSR and DCR) and regional (SRR) approaches.

The catchment rainfall results of the DCR approach were used as input for the Rational, DRH, SUH and SCS methods. The results for tc and 1-day are shown below.

Table 3-1: Catchment rainfall

Duration	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Rainfall (mm)						
tc	43	59	71	83	101	114	129
1-day	60	83	100	117	141	160	181

Appendix B contains more information on the catchment rainfall.

3.3. Deterministic methods

The Rational, DRH, SUH and SCS methods were used to estimate flood peaks for different exceedance probabilities.

Table 3-2: Estimated flood peaks – Deterministic methods

Method	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Flood peaks (m^3/s)						
Rational	11	24	35	48	68	85	104
DRH	14	22	29	37	51	62	75
SUH	3	5	7	9	13	15	19
SCS	7	17	28	40	60	77	97

Appendix C contains more information on the deterministic methods.

3.4. EMPIRICAL METHODS

The MIPI, HRU 1/71 and CAPA methods were used to estimate flood peaks for different exceedance probabilities.

Table 3-3: Estimated flood peaks – Empirical methods

Method	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Flood peaks (m^3/s)						
MIPI	23	56	87	126	187	241	302
HRU 1/71	6	14	21	28	39	49	61
CAPA	19	35	49	66	94	115	135

Appendix D contains more information on the empirical methods.

3.5. STATISTICAL METHODS

Inflow flood peaks were estimated for Roodefontein dam, for the period 1995 to 2016, by using a level-pool-routing technique.

The LN, LP3 and GEV_{MM} distributions were considered in the statistical analysis of the compiled inflow flood peak record to estimate the flood peaks for the required probabilities.

Table 3-4: Estimated flood peaks – Statistical methods

Method	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Flood peaks (m^3/s)						
STATS	8	20	31	44	62	77	94

Appendix E contains more information on the statistical methods.

3.6. ASSESSMENT OF RESULTS

The data record used in the statistical analysis is 22 years in length and the quality of the data used seems good. The estimates produced by the statistical analysis should therefore be fairly accurate keeping in mind that the data are an observation of the actual stream-flow response of the catchment. Thus, the flood peak estimates produced by the statistical analysis (STATS) were used as the benchmark to compare the results of the deterministic and empirical methods with.

An overall assessment of the results shows that the STATS results compare very well and well with that of the SCS and Rational methods respectively, in the whole exceedance probability range. Both the SCS and Rational methods were specifically developed to estimate flood peaks in small catchments and when used correctly (given reliable input) the methods perform very well in catchments less than 30 km^2.

3.7. RECOMMENDED FLOOD PEAKS

The results of the statistical analysis are proposed for the recommended flood peaks for the Roodefontein dam catchment.

Table 3-5: Recommended flood peaks

Exceedance probability (%)						
50	**20**	**10**	**5**	**2**	**1**	**0.5**
Flood peaks (m^3/s)						
8	20	30	45	65	80	95

3.8. RECOMMENDED FLOOD VOLUMES

The observed hydrographs are recommended for routing purposes.

Appendix F contains more information on the hydrographs and volumes.

4. RMF and SEF

4.1. Assessment

The regional maximum flood (RMF) was estimated by applying the methodology described in TR137 and is shown in **Table 4-1**.

Table 4-1: RMF

RMF	Catchment Area (km^2)	Region	Francou-Rodier K	Flood peak (m^3/s)
	28	5.2	5.2	650

Observations from numerous site-specific analyses, taking into account the suggested screening criteria from the SANCOLD guidelines, led Flood Studies to adopt the following approach in order to evaluate the site-specific K-values for the regional maximum flood (RMF) and safety evaluation flood (SEF), respectively labeled as K_e and K_{SEF}:

- Determine the rounded K-value (one decimal) for the forecasted 0.01% (10 000 year) flood peak ($K_{0.01\%}$).
- Analytically evaluate $K_{0.01\%}$ also considering the regional K-value (K_{TR137}), observed flood peaks and any other relevant information. Subsequently determine a 'site-specific' K-value (K_e) that is sound and consistent with the site specific analysis.
- The site-specific probable maximum flood is determined by using K_e and the proposed SEF by using $K_{SEF} = K_e + \delta$ ($0.1 \le \delta \le 0.2$)

In the statistical analysis the estimate for the 10 000 year flood peak ($Q_{0.01\%}$) is used as an indicator for the maximum flood that is expected in a catchment. By applying Flood Studies adopted approach the forecasted $Q_{0.01\%}$ estimate of 210 m^3/s has a $K_{0.01\%}$ value of 4.0.

The properties of five dams situated in the K drainage region and near the coast were evaluated and are shown in **Table 4-2**.

Table 4-2: Properties of coastal dams in the K drainage region

Dam	Area (km^2)	$Q_{0.01\%}$ (m^3/s)	$K_{0.01\%}$	K_{TR137}
Hartebeeskuil	100	885	4.9	5.2
Klipheuwel	11	130	3.8	5.2
Wolwedans	123	1590	5.2	5.2
Ernest Robertson	17	380	4.9	5.2
Garden Route	36	635	5.1	5.2

After careful consideration and taking into account all of the above information, a sight specific K_e value of 4.8 is suggested for Roodefontein dam.

4.2. Recommendation

After applying the Flood Studies adopted approach (section 4.1) and in accordance with the SANCOLD guidelines, following this site specific analysis, it is recommended that a SEF value of 530 m^3/s (K_{SEF} = 5.0) be considered for this catchment.

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APPENDIX A

ROODEFONTEIN DAM AND CATCHMENT

A.1. CATCHMENT LOCATION

Roodefontein dam is situated in the Piesang river about 2.5 km west (as the crow flies) of the town Plettenberg Bay in the Western Cape Province. The dam was mainly constructed for domestic and industrial use.

See **Figure A-1** for the catchment location and layout.

A.2. THE CATCHMENT

Roodefontein dam has a total catchment area of 28 km^2. The catchment receives rainfall throughout the year and the mean annual precipitation was estimated at 930 mm.

The surface hydrology of the catchment consists of various streams of which the longest stream originates ≈280 m.a.s.l. in the most western part of the catchment. The longest stream measures ≈14.80 km in length and the time of concentration for the catchment was estimated at 3.5 hours.

The basic vegetation cover consists mainly of dense- and thin bush with some cultivated land areas. The catchment soils are largely deemed to be very permeable to semi-permeable.

The most important catchment characteristics are given in.

A.3. THE DAM

Roodefontein dam was completed in 1989 and raised in 2004. The dam is an earthfill dam with an ogee spillway on the left hand side. The total crest width is 315 m. The dam has a wall height of approximately 18 m and a capacity of 2.069×10^6 m^3 at full supply level 43.91 m RL / 10.07 m GP. The surface area of the lake formed by the dam at FSL is ±37 hectares.

Roodefontein dam is classified as a category 3 dam of medium size with high hazard potential.

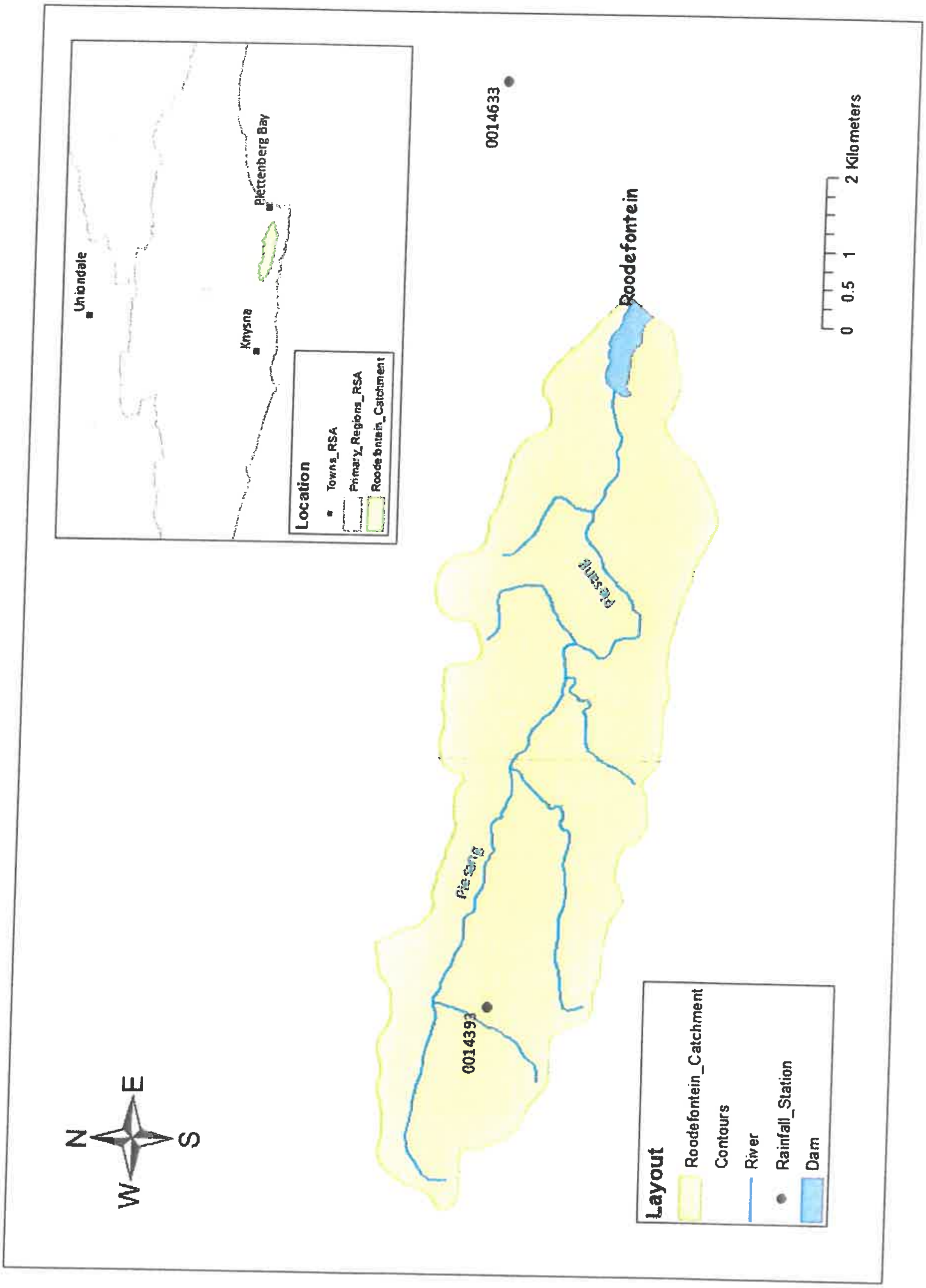


Figure A-1: Catchment location and layout

Table A-1: Catchment characteristics and information

IDENTIFICATION OF SITE

Place:	Roodefontein dam	Drainage Region:	K60G	Latitude:	-34.06667°
Watercourse:	Piesang river	DWS no.:	K602/02	Longitude:	23.33437°
Nearest town:	Plettenberg Bay	Hydro no.:	K6R001		

CATCHMENT CHARACTERISTICS

Catchment Area and Slope:

Total:	A	=	28	km^2	Dolomitic areas in effective catchment:				
Ineffective:	A_i	=	0.0	km^2	Part of A_e considered as dolomitic:	A_d	=	0.0	km^2
Effective:	A_e	=	28	km^2	Reduction factor	k	=	0.43	
Mean steepness of A_e	S_A	=	16.60	%	Flood peak reduction coefficient	f_d	=	1.00	

Watercourse properties:

Longest Watercourse:					Mean Slope:			
Natural channel	L_1	=	14.80	km	Mean slope of L_1	S_1	=	0.011683
Overland flow	L_2	=	0.00	km	Mean slope of L_2	S_2	=	0.000000
Total length	L	=	14.80	km	Mean slope of L_{total}	S_L	=	0.011683
Distance to centre of A	L_c	=	8.25	km	(distance, along channel, to a point closest to the centre of catchment)			

Time of concentration:

TYPE OF FLOW	TIME OF CONCENTRATION (hours)			where			
Natural channel	$t_{c1} = \tau\,[0.87{*}L_1^2 / (1000{*}S_1)]^{0.385}$	3.74	3.5	A_e	=	28	to calculate τ
Overland	$t_{c2} = 0{,}6 * [\,r * L_2 / (S_2)^{0.5}\,]^{0.467}$	0.00		τ	=	1.3	correction according to size of A
Artificial channel	$t_{c1a} = 0.278 * \Sigma\,(\,l_i / v_i\,)$			r	=	0.20	roughness factor
				l_i	=	length of a uniform reach	
				v_i	=	velocity in a uniform reach	

Typical r values	Pavement	0,02	Bare soil	0,1	Poor grass	0,3	Average grass & Cultivated land	0,4	Dense grass	0,8

Rainfall-Runoff properties:

Veld type zones (HRU 1/72 fig.F1)	1	
Relative Weight (%)	100%	

Catchment coverage as required for Rational method:

Rural	Urban	Lakes
100%	0%	0%

Soil Permeability:	%A	Land-Cover:	%A	Rainfall: (HRU 1/72 fig.C3) *	
Very Permeable (A)	30%	Forest, Dense bush & wood	81%	Mean Annual Rainfall (mm)	930
Permeable (B)	30%	Thin Bush, Cultivated land	16%	Extreme point rainfall region *	2
Semi-Permeable (C)	39%	Grassland	2%	Coastal (C) or Inland (I)	C
Impermeable (D)	0%	Bare surface	0%	Winter (W) or Summer (S)	S

ADDITIONAL INFORMATION FOR EMPIRICAL METHODS

MIPI Flood regions HRU 1/72 fig.B.1	3		
Relative proportion (%)	100%		

RMF region (TR137)	5.2
Francou-Rodier K_e	5.2

APPENDIX B
CATCHMENT RAINFALL

B.1. BACKGROUND

The Flood Studies component utilises four methods in calculating the rainfall for a specified catchment area. These include the maximum station-rainfall approach using catchment statistics (MSRcs), the maximum station-rainfall approach using station statistics (MSRss), daily catchment-rainfall approach (DCR) and the Smithers regional-rainfall approach (SRR). The methodology is summarised below.

B.2. RAINFALL ANALYSIS METHODOLOGY

Maximum Station-Rainfall approach (MSR)

Input:

- Point rainfall record at each available station
- Weighted representative catchment area of each rainfall station (Thiessen polygons)

MSR_{CS} (CS – Statistical analysis on Catchment Rainfall)

Note: Applicable only for patched data

Determine:

- The highest 1-day, 2-day, 3-day ... n-day point rainfall per annum, for each station
- The highest 1-day, 2-day, 3-day ... n-day weighted representative catchment rainfall per annum (Thiessen polygons)
- The highest ½tc, tc and 2tc (storm durations) catchment rainfall per annum
- The estimated storm duration events catchment rainfall, for all exceedance probabilities (statistical analyses)

The ARF-factor applicable to this approach will most probably differ from the existing ARF factor applicable to the next approach.

MSR_{SS} (SS – Statistical Analysis on Station Rainfall)

Note: Applicable for either patched or unpatched data

Determine:

- The highest 1-day, 2-day, 3-day ... n-day point rainfall per annum, for each station
- The highest ½tc, tc and 2tc (storm durations) point rainfall per annum, for each station
- The estimated storm duration events point rainfall per annum, for each station, for all exceedance probabilities (statistical analyses)
- The estimated storm duration events catchment rainfall, for all exceedance probabilities (Thiessen polygons)

Apply a suitable ARF-factor to determine the estimated effective catchment rainfall, for all considered storm events and appropriate exceedance probabilities.

Daily Catchment-Rainfall approach (DCR)

Note: Applicable only for patched data

Input:

- Point rainfall record at each available station
- Weighted representative catchment area of each rainfall station (Thiessen polygons)

Determine:

- For every day, in each year, the weighted representative catchment rainfall (Thiessen polygons)
- The highest 1-day, 2-day, 3-day ... n-day catchment rainfall per annum
- The highest ½tc, tc and 2tc (storm durations) catchment rainfall per annum
- The estimated storm duration events catchment rainfall, for all exceedance probabilities (statistical analyses)

As this method constitutes an analysis of the catchment rainfall for each day, there is no need to apply an ARF-factor.

Smithers Regional-Rainfall approach (SRR)

The Smithers regional-rainfall approach is a regional scale-invariance model developed for South Africa at the University of KwaZulu Natal. The model is applied through a software package which extrapolates n-day regional rainfall from a database of rainfall stations across South Africa, and provides n-day catchment rainfall frequencies specific to the limits of a defined rainfall area.

In implementing the model, the user defines the area over which the regional rainfall is required. The software generates a raster grid, from which it will then extrapolate n-day point rainfall data from within the specified area to provide point rainfall data for each point of the stipulated raster grid. The user is then able to calculate the average n-day regional catchment rainfall from the raster data obtained from the model.

B.3. RAINFALL ANALYSIS

After a thorough evaluation of the data of the rainfall stations situated in and outside the catchment area of Roodefontein dam, two rainfall stations were used in the site specific (MSR and DCR) rainfall analysis. A regional analysis (SRR) was also done.

Table B-1: Rainfall stations used

Station number	Data Record	MAP (mm)	Thiessen Polygon (km^2)	Station number	Data Record	MAP (mm)	Thiessen Polygon (km^2)
0014633	1900-2011	1005	22	0014393	1900-2011	645	6

The annual maximums of the downloaded rainfall data were converted to 24-hour depths and thereafter to ½tc, tc and 2tc rainfall depths according to the algorithm described in TR102.

The catchment rainfall was estimated for different storm durations and exceedance probabilities for both the site specific and regional approach. For ease of use only the estimates for tc are shown in **Table B-2** and **Table B-3**.

Table B-2: Catchment rainfall (ARF not applied)

Method	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Rainfall (mm)						
MSR_{SS}	46	64	77	91	109	124	140
MSR_{CS}	46	63	76	88	106	121	136
DCR	43	59	71	83	101	114	129
SRR	30	45	57	69	88	105	124

For both the MSR and SRR analyses, the appropriate factors for reduction in area (ARF) were applied. The ARF applicable for this catchment and storm duration tc is 0.915.

Table B-3: Catchment rainfall (ARF applied)

Method	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Rainfall (mm)						
MSR_{SS}	42	59	70	83	100	113	128
MSR_{CS}	42	58	70	81	97	111	124
DCR	43	59	71	83	101	114	129
SRR	28	41	52	63	81	96	113

From **Table B-3** it is clear that the results from the SRR approach (regional analysis) estimated lower results in the whole range of exceedance probabilities. The results for the MSR and DCR approaches (site specific analysis) compare well.

The rainfall estimates from the DCR approach (site specific analysis) was used as input for the deterministic methods (**Table B-4**).

Table B-4: Catchment rainfall

Duration	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Rainfall (mm)						
½tc	34	47	56	66	80	91	102
tc	43	59	71	83	101	114	129
2tc	51	72	86	101	122	138	156
1-day	60	83	100	117	141	160	181

C.1. DETERMINISTIC METHODS

The Rational, DRH, SUH and SCS methods were used to estimate flood peaks for different exceedance probabilities.

C.2. RATIONAL, DRH AND SUH PARAMETERS

The ½tc, tc and 2tc rainfall estimates from the DCR approach (site specific analysis) were used as input for the Rational, DRH and SUH methods and parameters were calculated.

Table C-1: Rainfall parameters for ½tc

Parameter	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Parameter value						
P (DCR)	34	47	56	66	80	91	102
ARF	1.000	1.000	1.000	1.000	1.000	1.000	1.000
k	0.118	0.146	0.159	0.174	0.194	0.210	0.224
P_e	4.0	6.8	8.9	11.5	15.5	19.1	22.9
i	19.429	26.857	32.000	37.714	45.714	52.000	58.286

Table C-2: Rainfall parameters for tc

Parameter	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Parameter value						
P (DCR)	43	59	71	83	101	114	129
ARF	1.000	1.000	1.000	1.000	1.000	1.000	1.000
k	0.139	0.164	0.181	0.199	0.223	0.240	0.259
P_e	6.0	9.7	12.9	16.5	22.5	27.4	33.4
i	12.286	16.857	20.286	23.714	28.857	32.571	36.857

Table C-3: Rainfall parameters for 2tc

Parameter	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Parameter value						
P (DCR)	51	72	86	101	122	138	156
ARF	1.000	1.000	1.000	1.000	1.000	1.000	1.000
k	0.152	0.183	0.203	0.223	0.250	0.270	0.290
P_e	7.7	13.2	17.4	22.5	30.5	37.2	45.3
i	7.286	10.286	12.286	14.429	17.429	19.714	22.286

Table C-4: Runoff coefficients for the Rational method

Parameter	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Parameter value						
C	0.116	0.182	0.222	0.258	0.302	0.334	0.363

C.3. SCS PARAMETERS

The 1-day rainfall estimates from the DCR approach (site specific analysis) were used as input for the SCS method. The following parameters were also used as input:

Table C-5: Parameters for the SCS method

Parameter name	Storm intensity distribution type	Hydrological response zone	Hydrological soil group	Catchment mean altitude (m)	MAP (mm)	Lag Time (Hours)	Curve number (final)
Parameter value	2	119	B	180	930	2.10	53.7

C.4. RESULTS

Flood peaks were calculated for different exceedance probabilities and storm durations.

Table C-6: Estimated flood peaks – Rational method

Storm duration	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Flood peaks (m^3/s)						
½tc	9	19	28	38	54	68	82
tc	11	24	35	48	68	85	104
2tc	7	15	21	29	41	51	63

Table C-7: Estimated flood peaks – DRH method

Storm duration	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Flood peaks (m^3/s)						
½tc	11	19	25	33	44	54	65
tc	14	22	29	37	51	62	75
2tc	12	20	26	34	46	56	68

Table C-8: Estimated flood peaks – SUH method

Storm duration	Exceedance probability (%)						
	50	**20**	**10**	**5**	**2**	**1**	**0.5**
	Flood peaks (m^3/s)						
½tc	3	5	6	8	10	13	15
tc	3	5	7	9	13	15	19
2tc	4	7	9	11	15	19	23

Table C-9: Estimated flood peaks – SCS method

Storm duration	Exceedance probability (%)						
	50	**20**	**10**	**5**	**2**	**1**	**0.5**
	Flood peaks (m^3/s)						
tc	7	17	28	40	60	77	97

D.1. EMPIRICAL METHODS

The MIPI, HRU 1/71 and CAPA methods were used to estimate flood peaks for different exceedance probabilities.

Technically speaking the storm duration for the flood peaks estimated by the empirical methods are unknown, but it is generally accepted that it coincide with storm duration tc.

Table D-1: Estimated flood peaks – Empirical methods

Method	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Flood peaks (m^3/s)						
MIPI	23	56	87	126	187	241	302
HRU 1/71	6	14	21	28	39	49	61
CAPA	19	35	49	66	94	115	135

D.2. RMF

The RMF in **Table D-2** was estimated by applying the methodology described in TR137.

Table D-2: RMF

RMF	Catchment Area (km^2)	Region	Francou-Rodier K	Flood peak (m^3/s)
	28	5.2	5.2	650

APPENDIX E
STATISTICAL METHODS

E.1. DATA

Roodefontein dam has a data record from October 1995 to present. The data of the dam (that include reservoir level data, capacity data, and rating data) were used to compile an inflow flood peak record for the period 1995 to 2016 by using a level-pool-routing technique.

E.2. FLOOD PEAK RECORD

Table E-1: Compiled inflow flood peak record for Roodefontein dam

Hydro year	Date	K6R001 Q_{in} (m^3/s)	K6R001 Q_{out} (m^3/s)	Q_{in} data used in analysis
1995/1996	30/11/1995	11	-	**11**
1996/1997	22/11/1996	20	5.4	**20**
1997/1998				**-99**
1998/1999				**-99**
1999/2000				**-99**
2000/2001				**-99**
2001/2002	10/09/2002	4.8	-	**4.8**
2002/2003	14/11/2002	3.2	-	**3.2**
2003/2004	24/07/2004	0.7	-	**0.7**
2004/2005	22/12/2004	23	-	**23**
2005/2006	02/08/2006	34	25	**34**
2006/2007	05/03/2007	2.6	-	**2.6**
2007/2008	23/11/2007	35	33	**35**
2008/2009	14/11/2008	6.2	-	**6.2**
2009/2010	15/07/2010	2.2	-	**2.2**
2010/2011	05/07/2011	14	12	**14**
2011/2012	14/07/2012	23	15	**23**
2012/2013	21/10/2012	5.1	3.8	**5.1**
2013/2014	01/11/2013	5.4	4.3	**5.4**
2014/2015	01/09/2015	27	24	**27**
2015/2016	01/11/2015	9.4	7.0	**9.4**
2016/2017	16/10/2016	1.8	1.1	**1.8**

-99 indicates permanent gap in data record

E.3. ESTIMATED FLOOD PEAKS

The combined data record used in the statistical analysis is 22 years in length and the quality of the data used seems to be good. The LN, LP3 and GEV_{MM} distributions were considered in the statistical analysis (STATS) of the compiled inflow flood peak record to estimate the flood peaks for the required probabilities (see **Figure E-1**).

The LP3 distribution was used for the statistical estimates.

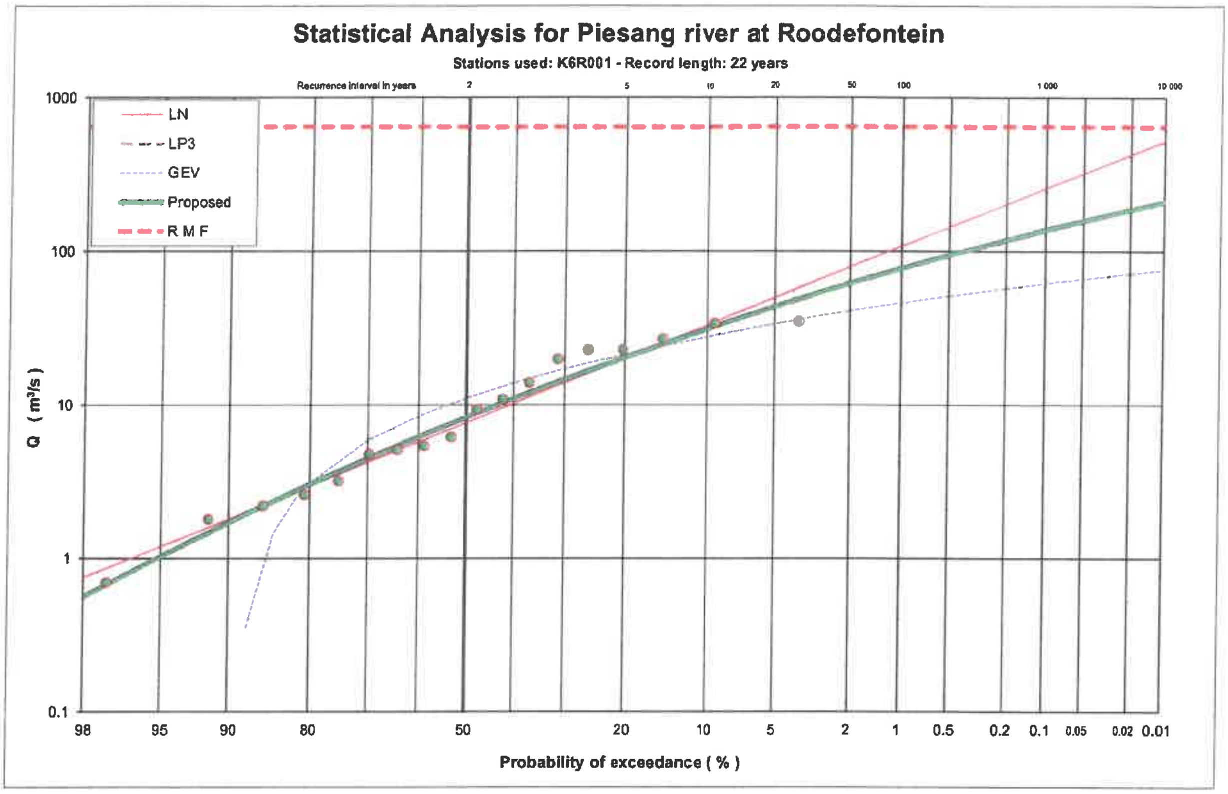


Figure E-1: Graphical comparison of data points and distributions

Table E-2: Estimated flood peaks – Statistical methods

Method	Exceedance probability (%)						
	50	20	10	5	2	1	0.5
	Flood peaks (m^3/s)						
STATS	8	20	31	44	62	77	94

Table E-3: Information about available record

		Continuous
Record Length (years)		22
Additional Record Length (years)		
Equivalent Record Length (years)		
Peaks higher than threshold$_{max}$	(outliers)	-
Valid peaks between thresholds		18
Peaks lower than threshold$_{min}$	(outliers)	-
Zero flows		-
Missing data (gaps in record)		4
Data not used in analysis (high; low; zero; missing)		4

Table E-4: Statistical properties

	Untransformed data	Log-transformed data
Median	7.8	
Mean	12.7	0.8871
s	11.4	0.4924
g	0.8273	-0.3919

Table E-5: Results of fitted distributions

AEP	RI	LN		LPIII		GEV$_{MM}$			Proposed
		WT	Q (m^3/s)	WT	Q (m^3/s)	WT	Par.	Q (m^3/s)	Q (m^3/s)
0.50	2	0.00	8	0.07	8	0.36		11	8
0.20	5	0.84	20	0.85	20	1.44	k	21	20
0.10	10	1.26	33	1.23	31	2.11	0.058	28	31
0.05	20	1.64	50	1.53	44	2.73		34	44
0.02	50	2.05	79	1.84	62	3.49		41	62
0.01	100	2.33	108	2.04	77	4.03	E(y)	46	77
0.005	200	2.58	143	2.21	94	4.56	0.970	51	94
0.002	500	2.88	201	2.41	118	5.21		57	118
0.001	1000	3.09	256	2.54	138	5.69		62	138
0.0005	2000	3.29	322	2.67	158	6.14	var(y)	66	158
0.0002	5000	3.54	427	2.81	187	6.71	0.005	72	187
0.0001	10000	3.72	523	2.91	210	7.13		76	210

APPENDIX F
HYDROGRAPHS

F.1. HYDROGRAPH PEAKS

The recommended flood peaks are based on storm duration tc. Hydrograph peaks were also calculated for storm durations ½tc and 2tc. The variation of flood peaks (*var*) with storm duration was taken into account in the following way:

- Calculate the variance (*var*) between the flood peaks of the storm durations ½tc and 2tc and tc respectively using the Rational method. The maximum value should be taken as *var* = 1.000.
- Correct *var* to obtain *var** = (*var* + 1) / 2. This is necessary to take into account the fact that the representative storm durations for the flood peaks estimated by the statistical and empirical methods are unknown.
- Multiply the recommended flood peaks for tc, and each exceedance probability, by the representative values of var* to get the proposed flood peaks for ½tc and 2tc.

Table F-1: Recommended flood peaks – Roodefontein dam

Storm duration	Exceedance probability (%)							
	50	20	10	5	2	1	0.5	RMF
	Flood peaks (m^3/s)							
½tc	7	18	27	40	58	72	85	582
tc	**8**	**20**	**30**	**45**	**65**	**80**	**95**	**650**
2tc	6	16	24	36	52	64	76	522

F.2. HYDROGRAPH VOLUMES

Stream flow data are available in the Roodefontein dam catchment and therefore the corresponding peaks and volumes of the observed hydrographs were compared with that estimated by the DRH and SUH methods.

F.2.1 DRH HYDROGRAPHS

The 1% exceedance probability hydrographs, as estimated by the DRH method, are shown below. The DRH hydrograph shapes (for each of the storm durations) will stay the same for the other exceedance probabilities. The ratio, of another exceedance probability flood peak and the 1% exceedance probability flood peak, must be applied to the 1% exceedance probability hydrograph coordinates.

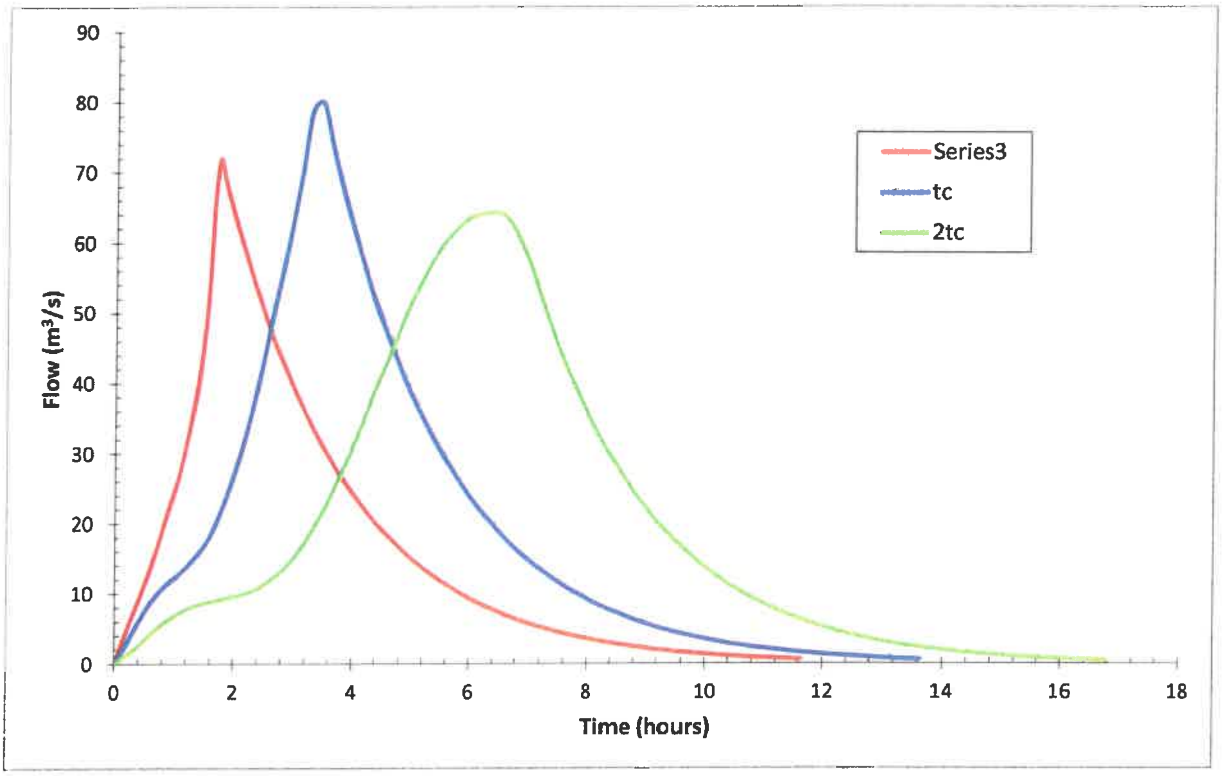


Figure F-1: DRH hydrographs

Hydrographs volumes, calculated for different exceedance probabilities and storm durations by using the DRH method, are shown in the table below.

Table F-2: DRH method volumes

Storm duration	Exceedance probability (%)							
	50	**20**	**10**	**5**	**2**	**1**	**0.5**	**RMF**
	Volumes (10^6 m^3)							
½tc	0.069	0.174	0.260	0.392	0.565	0.698	0.825	5.644
tc	**0.098**	**0.245**	**0.368**	**0.551**	**0.796**	**0.980**	**1.164**	**7.964**
2tc	0.117	0.297	0.444	0.667	0.961	1.183	1.405	9.610

F.2.2 SUH HYDROGRAPHS

The 1% exceedance probability hydrographs, as estimated by the SUH method, are shown below. The SUH hydrograph shapes (for each of the storm durations) will stay the same for the other exceedance probabilities. The ratio, of another exceedance probability flood peak and the 1% exceedance probability flood peak, must be applied to the 1% exceedance probability hydrograph coordinates.

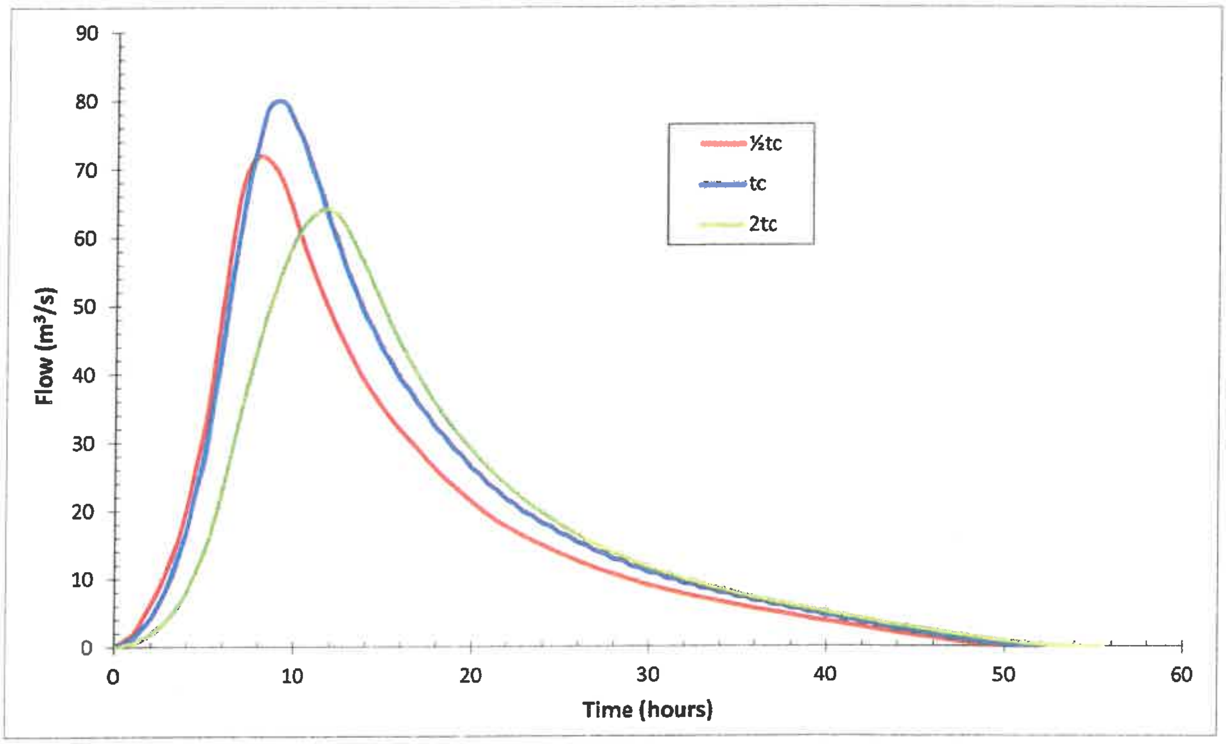


Figure F-2: SUH hydrographs

Hydrographs volumes, calculated for different exceedance probabilities and storm durations by using the SUH method, are shown in the table below.

Table F-3: SUH method volumes

Storm duration	Exceedance probability (%)							
	50	20	10	5	2	1	0.5	RMF
	Volumes (10^6 m^3)							
½tc	0.345	0.864	1.291	1.943	2.802	3.461	4.092	28.000
tc	**0.397**	**0.993**	**1.490**	**2.235**	**3.228**	**3.973**	**4.717**	**32.277**
2tc	0.359	0.906	1.355	2.036	2.933	3.613	4.289	29.345

F.2.3 OBSERVED HYDROGRAPHS

The hydrographs of the biggest flood events (biggest peak volume ratios) that occurred in the catchment, and recorded at the dam, are displayed below. The dashed lines indicate the boundaries for the volume calculations.

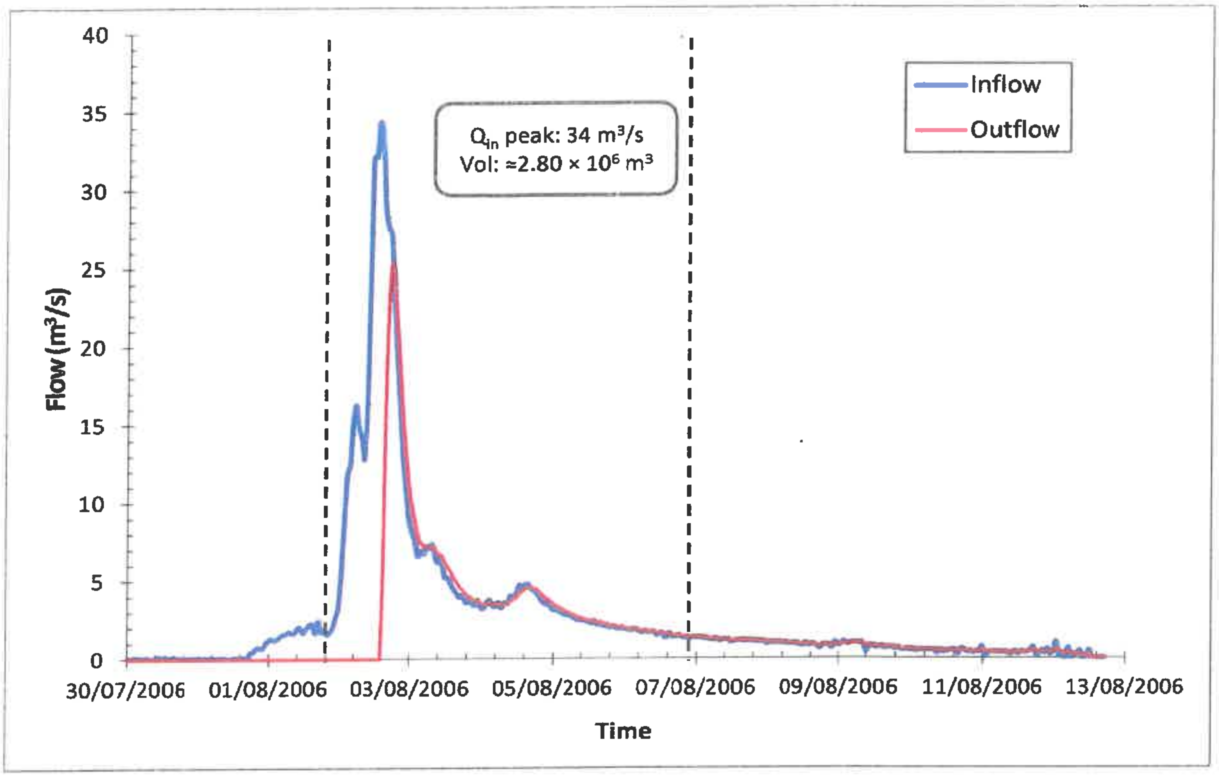


Figure F-3: Flood hydrograph of August 2006

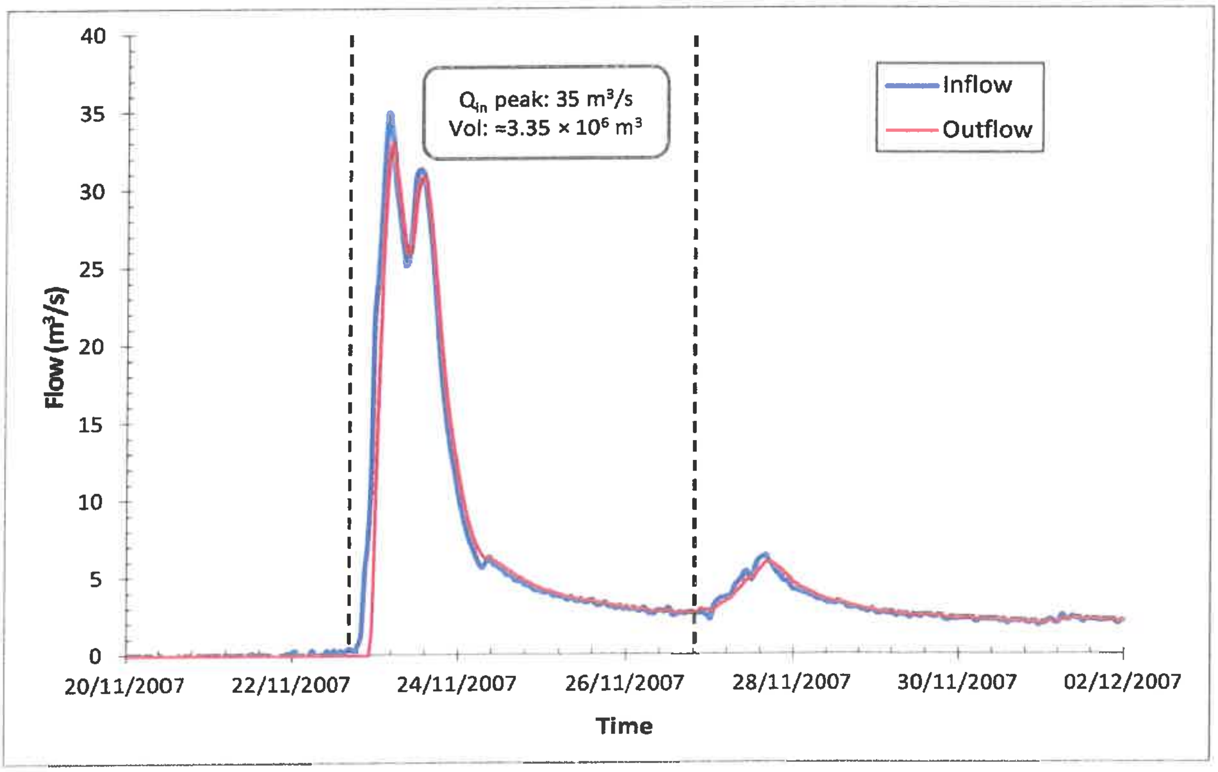


Figure F-4: Flood hydrograph of November 2007

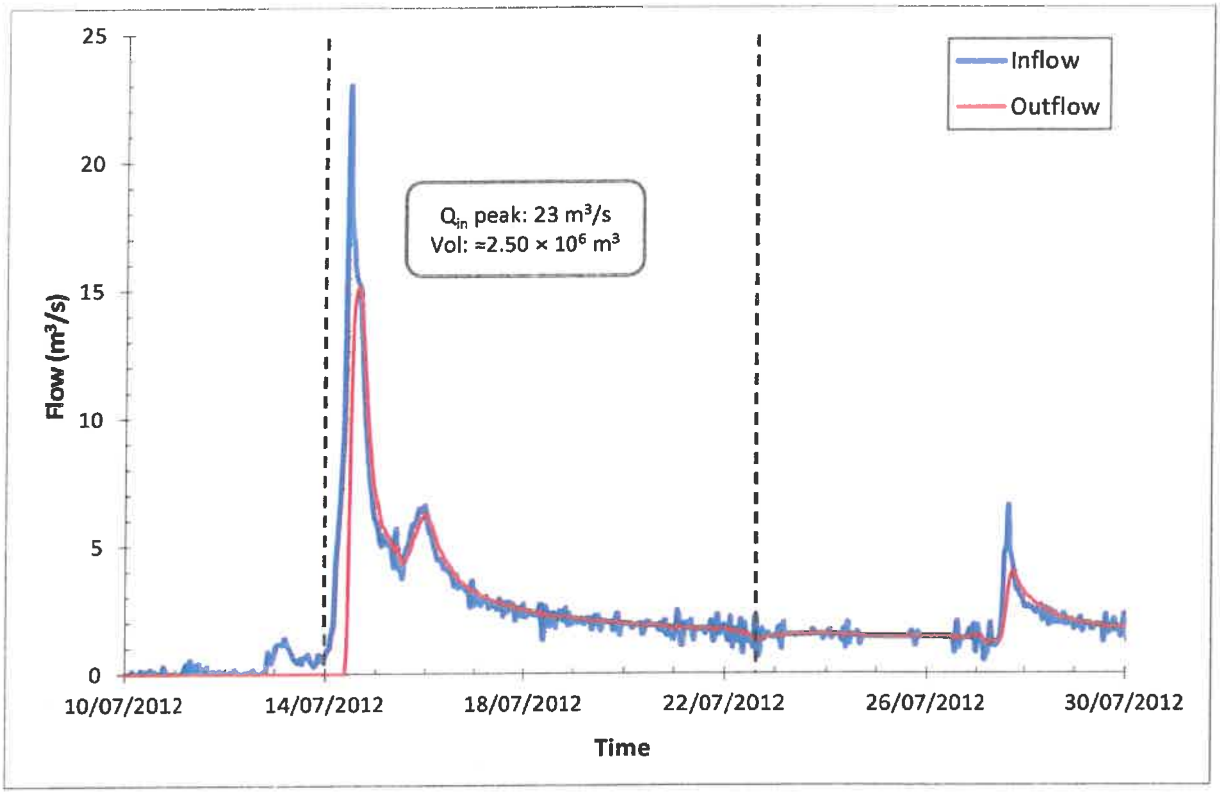


Figure F-5: Flood hydrograph of July 2012

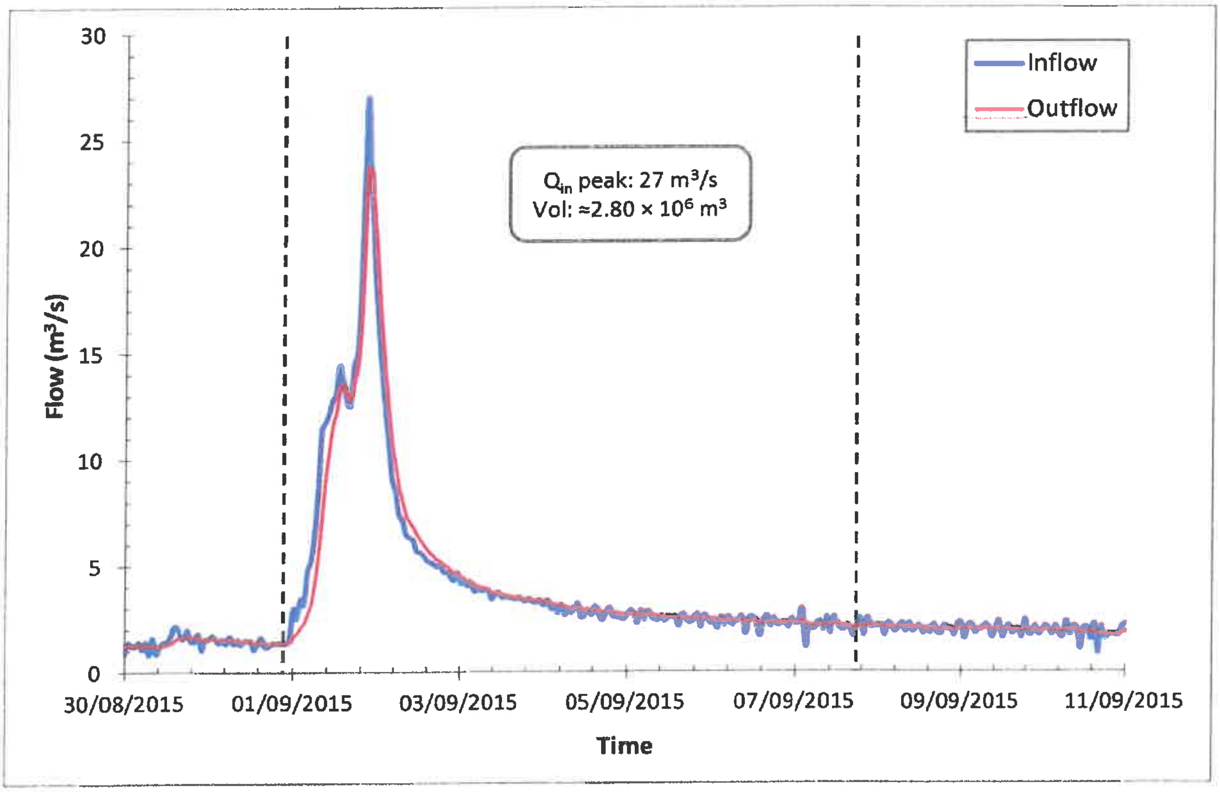


Figure F-6: Flood hydrograph of September 2015

F.2.4 HYDROGRAPH DATA COMPARED

The observed (recorded) hydrographs peak/volume pairs were plotted against that estimated by the DRH and SUH methods (see **Figure F-7** and **Figure F-8**).

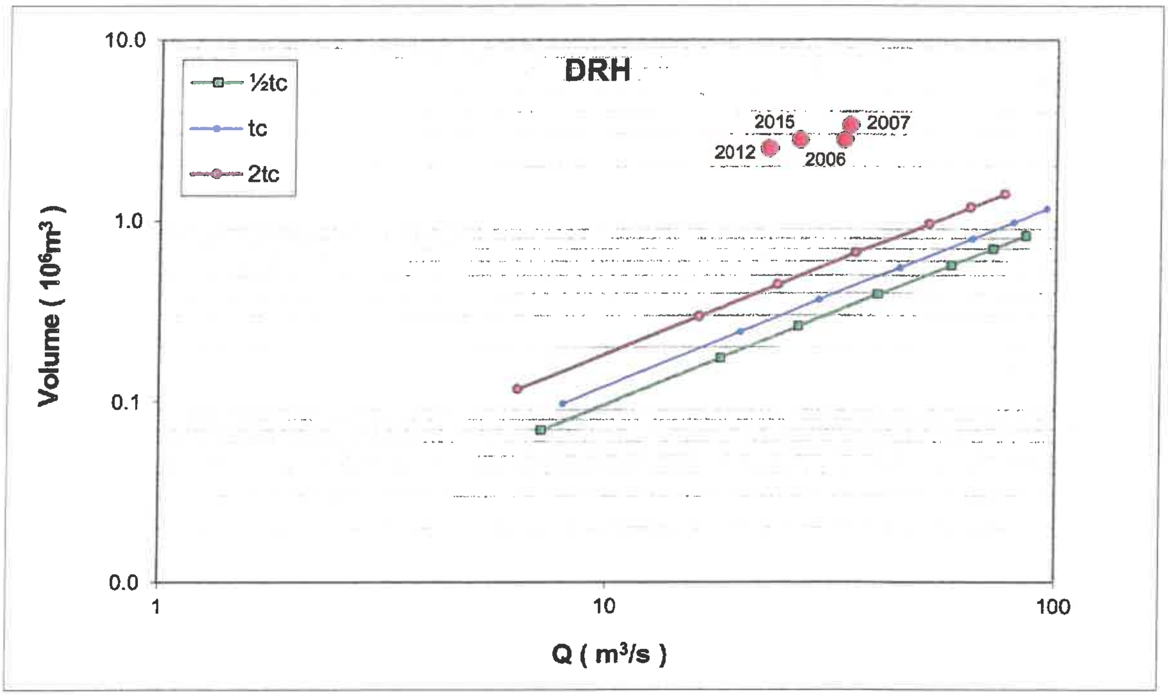


Figure F-7: DRH vs observed hydrographs

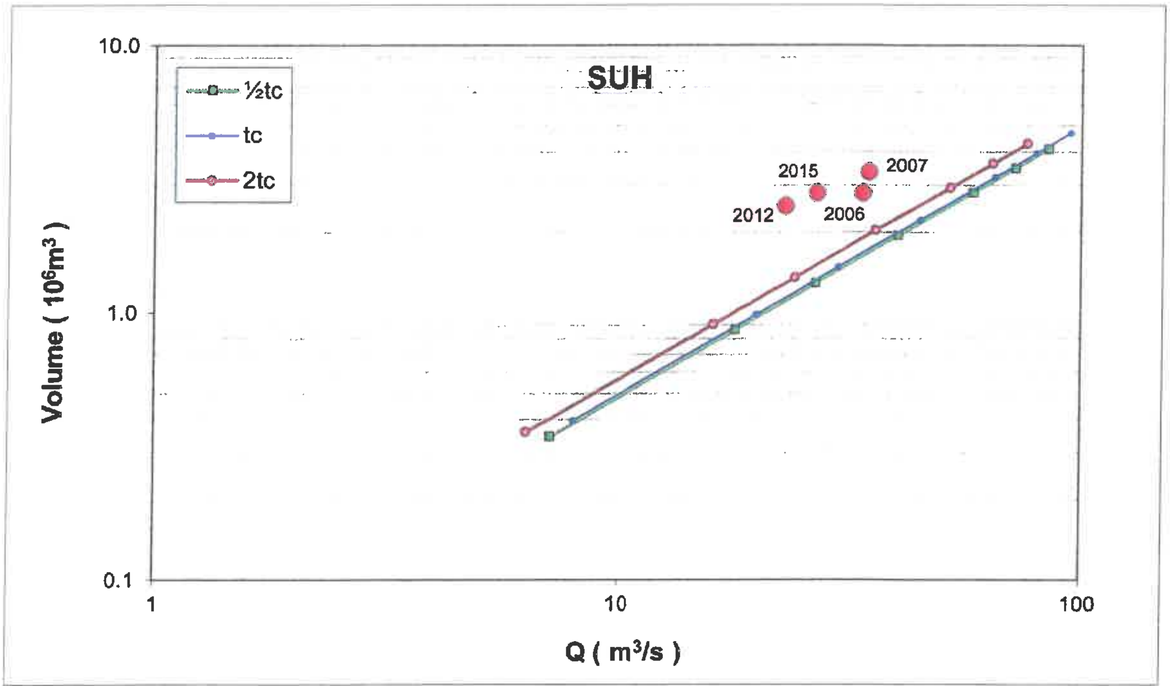


Figure F-8: SUH vs observed hydrographs

From **Figure F-7** and **Figure F-8** above it is clear that the hydrographs estimated by the SUH method compare better with the observed hydrograph than that of the DRH method.

F.2.5 RECOMMENDED HYDROGRAPHS

The observed hydrographs are recommended for routing purposes.

F.2.6 HYDROGRAPH COORDINATES

Table F-4: Coordinates of the August 2006 hydrograph

Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)
0	1.6	38	6.2	76	3.1
1	1.8	39	6.6	77	3.1
2	2.1	40	5.3	78	3.1
3	2.8	41	5.3	79	2.9
4	3.4	42	4.8	80	2.8
5	5.8	43	4.5	81	2.8
6	9.0	44	4.2	82	2.8
7	11.9	45	4.0	83	2.6
8	12.5	46	4.1	84	2.5
9	15.3	47	3.6	85	2.5
10	16.3	48	3.8	86	2.5
11	14.8	49	3.6	87	2.4
12	14.1	50	3.5	88	2.4
13	12.9	51	3.8	89	2.3
14	16.5	52	3.4	90	2.4
15	24.0	53	3.2	91	2.2
16	32.1	54	3.6	92	2.3
17	32.1	55	3.8	93	2.1
18	34.4	56	3.4	94	2.2
19	33.8	57	3.4	95	2.0
20	28.9	58	3.7	96	2.1
21	27.6	59	3.3	97	2.0
22	27.3	60	3.6	98	1.9
23	23.4	61	3.6	99	2.0
24	19.8	62	4.1	100	1.9
25	16.6	63	4.0	101	1.9
26	13.3	64	4.2	102	1.8
27	11.1	65	4.7	103	1.9
28	9.2	66	4.6	104	1.9
29	8.2	67	4.8	105	1.8
30	7.7	68	4.8	106	1.7
31	6.6	69	4.5	107	1.8
32	7.0	70	4.4	108	1.7
33	6.7	71	4.2	109	1.7
34	7.3	72	3.9	110	1.7
35	7.1	73	3.6	111	1.7
36	7.4	74	3.5	112	1.6
37	6.5	75	3.4	113	1.6

Table F-5: Coordinates of the November 2007 hydrograph

Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)
0	0.6	38	6.3	76	3.0
1	1.4	39	5.9	77	3.1
2	5.2	40	5.8	78	2.9
3	7.5	41	5.6	79	3.0
4	11.7	42	5.4	80	2.9
5	21.5	43	5.2	81	2.8
6	24.4	44	5.2	82	3.1
7	27.9	45	4.9	83	2.8
8	32.4	46	4.8	84	2.7
9	35.0	47	4.8	85	2.7
10	33.8	48	4.5	86	2.9
11	30.5	49	4.5	87	2.9
12	28.8	50	4.4	88	2.8
13	26.9	51	4.2	89	3.1
14	25.2	52	4.3	90	2.6
15	25.6	53	4.1	91	2.6
16	28.1	54	4.0	92	2.7
17	31.0	55	4.1	93	2.7
18	31.4	56	3.8	94	2.7
19	31.0	57	3.9	95	2.8
20	29.2	58	3.8	96	2.7
21	27.1	59	3.8	97	2.7
22	24.6	60	3.6	98	2.7
23	21.8	61	3.7	99	2.7
24	18.8	62	3.6	100	2.5
25	16.6	63	3.5	101	2.3
26	14.6	64	3.6		
27	13.0	65	3.4		
28	11.8	66	3.4		
29	10.4	67	3.6		
30	9.3	68	3.4		
31	8.3	69	3.2		
32	7.5	70	3.3		
33	7.0	71	3.4		
34	6.4	72	3.1		
35	5.9	73	3.2		
36	5.7	74	3.0		
37	6.0	75	3.1		

Table F-6: Coordinates of the July 2012 hydrograph

Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)
0	1.6	46	6.0	91	2.6	136	1.7
1	2.1	47	5.6	92	2.3	137	2.1
2	4.2	48	5.5	93	2.7	138	2.3
3	5.4	49	5.1	94	2.2	139	1.5
4	6.6	50	4.8	95	2.3	140	2.0
5	8.2	51	4.5	96	2.5	141	2.0
6	10.0	52	4.5	97	2.4	142	2.0
7	16.2	53	4.4	98	2.3	143	2.0
8	28.0	54	4.2	99	2.4	144	2.0
9	18.2	55	4.1	100	2.3	145	1.8
10	17.0	56	4.1	101	2.7	146	1.9
11	15.8	57	4.0	102	1.3	147	1.9
12	15.4	58	3.9	103	2.7	148	1.9
13	15.2	59	3.5	104	2.5	149	2.0
14	13.6	60	3.9	105	1.8	150	1.6
15	11.7	61	3.4	106	2.3	151	1.6
16	10.1	62	3.4	107	2.4	152	2.0
17	8.9	63	3.5	108	1.8	153	2.0
18	7.8	64	3.4	109	2.3	154	1.9
19	7.1	65	3.3	110	2.5	155	1.9
20	6.3	66	3.4	111	2.3	156	1.9
21	6.0	67	2.6	112	2.1	157	1.9
22	5.8	68	3.7	113	2.6	158	1.7
23	5.2	69	2.7	114	1.9	159	2.0
24	5.0	70	3.2	115	2.4	160	1.7
25	5.5	71	3.0	116	1.9	161	2.1
26	5.2	72	2.7	117	2.2	162	1.5
27	5.1	73	2.8	118	2.2	163	1.9
28	5.2	74	2.9	119	2.0	164	1.8
29	4.6	75	3.0	120	2.2	165	1.4
30	4.2	76	2.6	121	2.2	166	1.9
31	5.7	77	2.8	122	2.0	167	2.5
32	4.1	78	2.7	123	2.1	168	1.8
33	4.1	79	2.7	124	2.2	169	1.1
34	3.8	80	2.5	125	2.0	170	2.0
35	4.6	81	2.9	126	1.7	171	1.8
36	4.9	82	2.8	127	2.3	172	1.8
37	5.1	83	2.3	128	2.0	173	2.1
38	5.6	84	2.8	129	2.3	174	1.3
39	5.9	85	2.4	130	1.7	175	1.8
40	5.8	86	2.8	131	2.1	176	1.8
41	6.2	87	2.5	132	2.0	177	1.8
42	6.5	88	2.8	133	2.0	178	1.8
43	6.4	89	2.1	134	2.0	179	1.8
44	6.4	90	2.6	135	2.0	180	1.6
45	6.6						

Table F-7: Coordinates of the September 2015 hydrograph

Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)	Time (hrs)	Flow (m^3/s)
0	1.6	41	5.1	82	3.0	123	2.1
1	3.1	42	5.0	83	3.2	124	2.6
2	2.5	43	5.1	84	2.6	125	2.3
3	3.5	44	4.8	85	2.8	126	2.6
4	3.2	45	4.7	86	2.9	127	2.3
5	4.8	46	4.5	87	2.7	128	2.4
6	5.3	47	4.4	88	3.1	129	2.4
7	6.5	48	4.7	89	2.7	130	2.6
8	8.6	49	4.2	90	2.5	131	1.8
9	11.5	50	4.3	91	2.8	132	2.6
10	11.8	51	4.1	92	3.0	133	2.6
11	12.2	52	4.2	93	2.7	134	2.4
12	12.9	53	3.8	94	2.6	135	1.7
13	13.1	54	3.9	95	2.6	136	2.4
14	14.5	55	3.9	96	2.6	137	2.7
15	13.8	56	3.9	97	2.7	138	2.2
16	12.7	57	3.9	98	2.9	139	2.6
17	12.6	58	3.5	99	2.8	140	2.3
18	14.5	59	3.7	100	2.4	141	2.3
19	15.1	60	3.8	101	2.9	142	2.3
20	18.1	61	3.5	102	2.5	143	2.1
21	23.2	62	3.6	103	2.9	144	2.5
22	27.1	63	3.5	104	2.7	145	2.3
23	23.4	64	3.5	105	2.5	146	2.3
24	19.3	65	3.5	106	2.5	147	3.0
25	16.6	66	3.5	107	2.7	148	1.2
26	14.1	67	3.4	108	2.4	149	2.2
27	12.2	68	3.4	109	2.6	150	2.0
28	10.6	69	3.5	110	2.6	151	2.2
29	9.1	70	3.3	111	2.6	152	2.4
30	8.5	71	3.3	112	2.2	153	2.0
31	7.5	72	3.4	113	2.7	154	2.6
32	7.2	73	3.2	114	2.6	155	1.9
33	6.5	74	3.4	115	2.4	156	2.6
34	6.4	75	3.0	116	2.5	157	2.3
35	6.3	76	3.1	117	2.7	158	2.1
36	5.8	77	2.8	118	2.2	159	2.0
37	5.7	78	3.2	119	2.7	160	2.2
38	5.5	79	2.7	120	2.5	161	2.0
39	5.3	80	2.9	121	2.3	162	1.7
40	5.2	81	2.8	122	2.5		

Addendum E: Flood routing

Addendum F: Freeboard Calculations

FREEBOARD CALCULATIONS

1. METHODOLOGY

a) Select wind speeed from fig 2.3 -4 of SANCOLD freeboard guidelines or a suitable source
b) Determine fetch (Km). Either use the SANCOLD recommendations or the USBR approach
c) Calculate the Significant Height (H_s), use Saville Method or the Donelan et al method.
d) Calculate Wave Period and Length.
e) Calculate Wave Runup.
f) Calculate Wind Setup.
g) Determine miximum stage corrosponding to maximum discharge of RDD through flood routing
h) Calculate Surge and Seich
i) Calculate Earthquake Wave
j) Calculate land slide wave height
k) Calculate the influence of 25% inoperation of the outlet works
l) Use table 3.1 of SANCOLD Guidelines on Freeboard to calculate required freeboard.
m) Compare the obtained freeboard value with values given in table 3.3 of the SANCOLD guidelines and the existing freeboard at the dam of interest.

2. DESIGN STANDARDS:

a) USBR Design Standard No.13: Freeboard, 2012. (USBR - DS No.13:FB)
b) South African National Committee On Large Dam: Freeboard Guidelines, 2011 (SANCOLD:FB)
c) The Rock Manual

3. PROPERTIES OF THE DAM

Input parameters (variables)

Fetch…………………..……..; F = 1110 m
Wind Speed ………….……...; U_{10} = 24 m/s
Upstream Slope Angle …. ..; α = 18.4°
Density of air…………………; $\rho_{\alpha\iota\rho} = 1.2$ kg/m^3
Density of water………….….; $\rho_{\omega} = 1000$ kg/m^3
Volume of basin at FSL…….; A_{FSL} = 37010000 m^2
Area of basin at FSL………..; V_{FSL} = 2063000 m^3
Air/Water drag coefficient ….; C_D = 0.005
Gravitational acceleration…..; g = 9.81 m/s^2
Wave direction ………………; θ = 0°
Wind direction ……………….; ϕ_w = 0°
RL of Spillway Crest…………; $RL_{Spillway\ Crest}$ = 42.3 m
RL of NOC..…………………..; RL_{NOC} = 46 m

4. CALCULATIONS

4.1. Minimum duration (t_{min})

$$\frac{g\,t_{min}}{U_{10}\cos(\theta-\phi_w)}=30.1\left(\frac{g\,F_\theta}{\left(U_{10}\cos(\theta-\phi_w)\right)^2}\right)^{0.77}$$ ………………………………………..…………….*Rock Manual*

; $t_{min} = (30.1 / g) \times U_{10} \times \mathrm{Cos}(\theta - \phi_w) \times (g \times F / (U_{10} \times \mathrm{Cos}(\theta - \phi_w))^2)^{0.77}$ = **708.055** s

4.2. Wave Height (Significant Wave Height: H_s)

$$\frac{g\,H_s}{\left(U_{10}\cos(\theta-\phi_w)\right)^2}=0.00366\left(\frac{g\,F_\theta}{\left(U_{10}\cos(\theta-\phi_w)\right)^2}\right)^{0.38}$$ ……………………………………….……….………….*Rock Manual*

$H_s = (0.000366\ \mathrm{m}^{-0.38} / g) \times (U_{10} \times \mathrm{Cos}(\theta - \phi_w))^2 \times (g \times F / (U_{10} \times \mathrm{Cos}(\theta - \phi_w)))^{0.38}$ = **0.220** m

4.3. Wave Period (T)

$$\frac{g\,T_p}{U_{10}\cos(\theta-\phi_w)}=0.542\left(\frac{g\,F_\theta}{\left(U_{10}\cos(\theta-\phi_w)\right)^2}\right)^{0.23}$$ …………………………………………………………...*Rock Manual*

$T = (0.542 / g) \times U_{10} \times \mathrm{Cos}(\theta - \phi_w) \times (g \times F / (U_{10} \times \mathrm{Cos}(\theta - \phi_w))^2)^{0.23}$ = **2.607** s

4.4. Wave Length (L)

$L = 1.56\ \mathrm{m/s^2} \times T^2$ = **10.603** m

4.5. Wave Run-up (R)

a) Steepness of the peak wave (S_p)

$S_p = H_s/L$ = **0.021**

b) Surf Similarity Factor (ξ_p)

$\xi_p = \tan(\alpha) / \sqrt{(H_s/L)}$ = **2.311**

R : $H_s \times (A \times \xi_p + C) \times \gamma_r \times \gamma_b \times \gamma_h \times \gamma_\beta$

Type of slope surface	Yr
Smooth, concrete, asphalt	1
Smooth block revetment	1
Grass (3 centimeters in length)	0.90 – 1.0
One layer of rock, diameter D, (Hs/D = 1.5 – 3.0)	0.55 – 0.6
Two or more layers of rock, (Hs/D = 1.5 – 6.0)	0.50 – 0.55

Figure 1: Surface roughness reduction factors (USBR Table B-3:2011)

ξ_p - Limits	A	C
$\xi_p \leq 2.5$	1.6	**0**
$2.5 < \xi_p < 9$	-0.2	**4.5**

Figure 2: Values of variable A and B (USBR Table B-4:2012)

Run-Up parameters

$\gamma_r = 0.55$
$\gamma_b = 1$
$\gamma_h = 1$
$\gamma_\beta = 0.85$
A = 1.6
C = 0

$R = H_s \times A \times \xi_p \times \gamma_r \times \gamma_b \times \gamma_h \times \gamma_\beta$ = **0.380** m

4.6. *Wind Setup (n_w)*

$$n_w = \frac{1}{2} \cdot \frac{\rho_{air}}{\rho_w} \cdot C_D \cdot \frac{U_{10}^2}{gh} \cdot F$$

……………………………………………………………….*Rock Manual*

Average Depth of basin; $h_{ave} = V_{FSL} / A_{FSL}$ = **0.056** m

; $n_w = 0.5 \times (\rho_{\alpha\iota\rho} / \rho_\omega) \times C_D \times (U_{10})^2 / (g \times h_{ave}) \times F$ = **3.508** m

4.7. *Recommended Design Discharge (RDD)*

From Flood Routing;…………………………; RL_{RDD} = 447.5 m

Height of water above spillway crest……… ; h_{RDD} = 1.5 m

4.8. *Surge and Seiche*

; h_{surge} = 0 m

4.9. *Eathquake Wave*

; h_{EQ} = 0 m

4.10. Land Slide

; $h_L = 0$ m

4.11. Flood Outlet

; $h_{FO} = 0$ m

4.12. Freebord Combinations

Combination number	RDD Surchage	Wind wave and run-up 100-year event	Wind set-up	Surges and seiches	Earth-quake wave	Land-slide wave	Flood outlets
1	X	X					
2	X	X	X	X			
3					X		
4						X	
5	X	X	X	X			X

Figure 3: Proposed Design Considerations of Freeboard condition to be considered with RDD Surcharge

4.12.1. Combination 1: FB_1

$FB_1 = h_{RDD} + R =$ **1.880** m

4.12.2. Combination 2: FB_2

$FB_2 = h_{RDD} + R + n_w =$ **5.387** m

4.12.3. Comination 3: FB_3

$FB_3 = h_{EQ}$

4.12.4. Combination 4:FB_4

$FB_4 = h_L$

4.12.5. Combination 5:FB_5

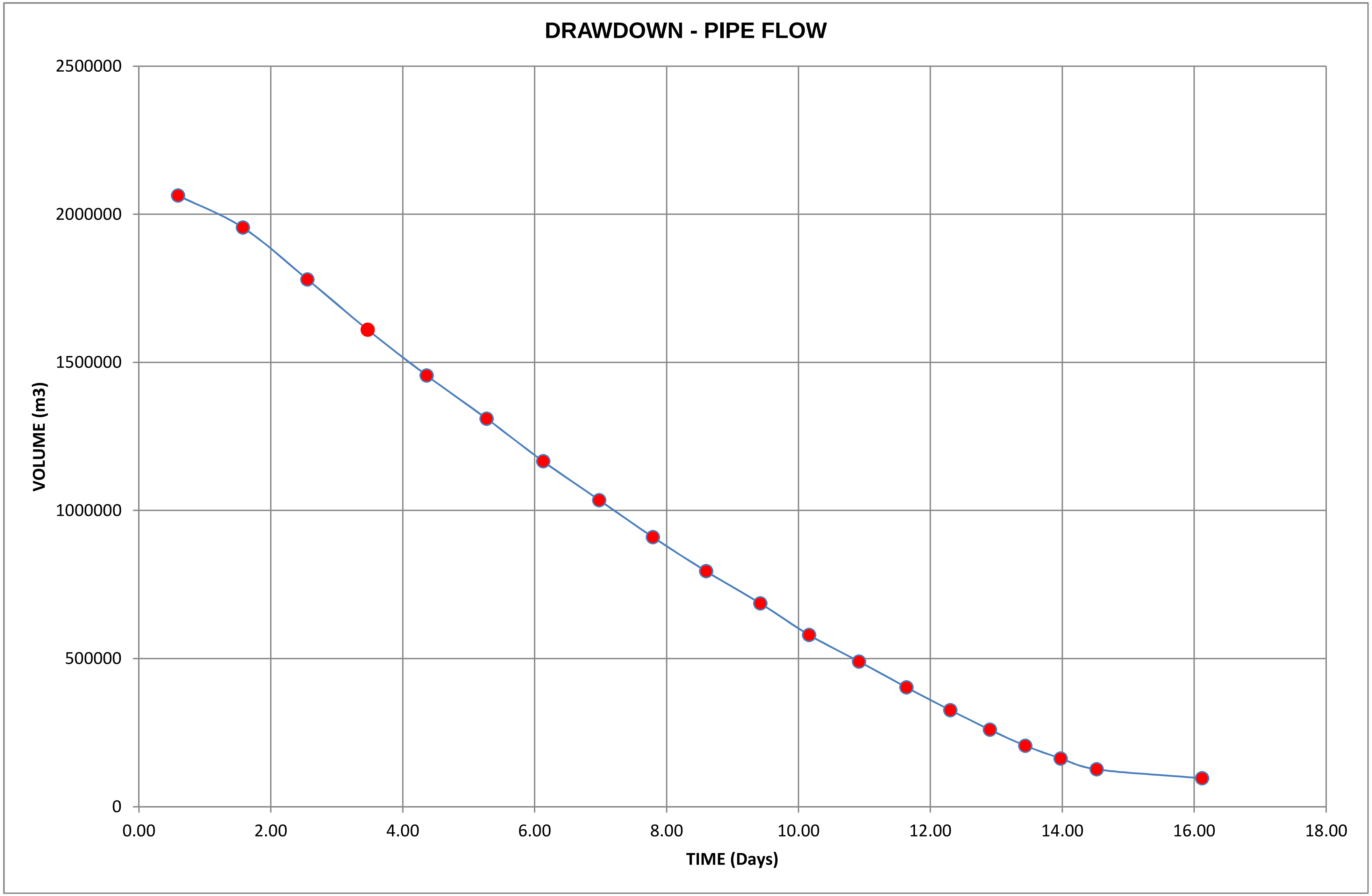
$FB_5 = h_{RDD} + R + n_w + h_{surge} + h_{FO} =$ **5.387** m

Addendum G: Drawdown Capacity

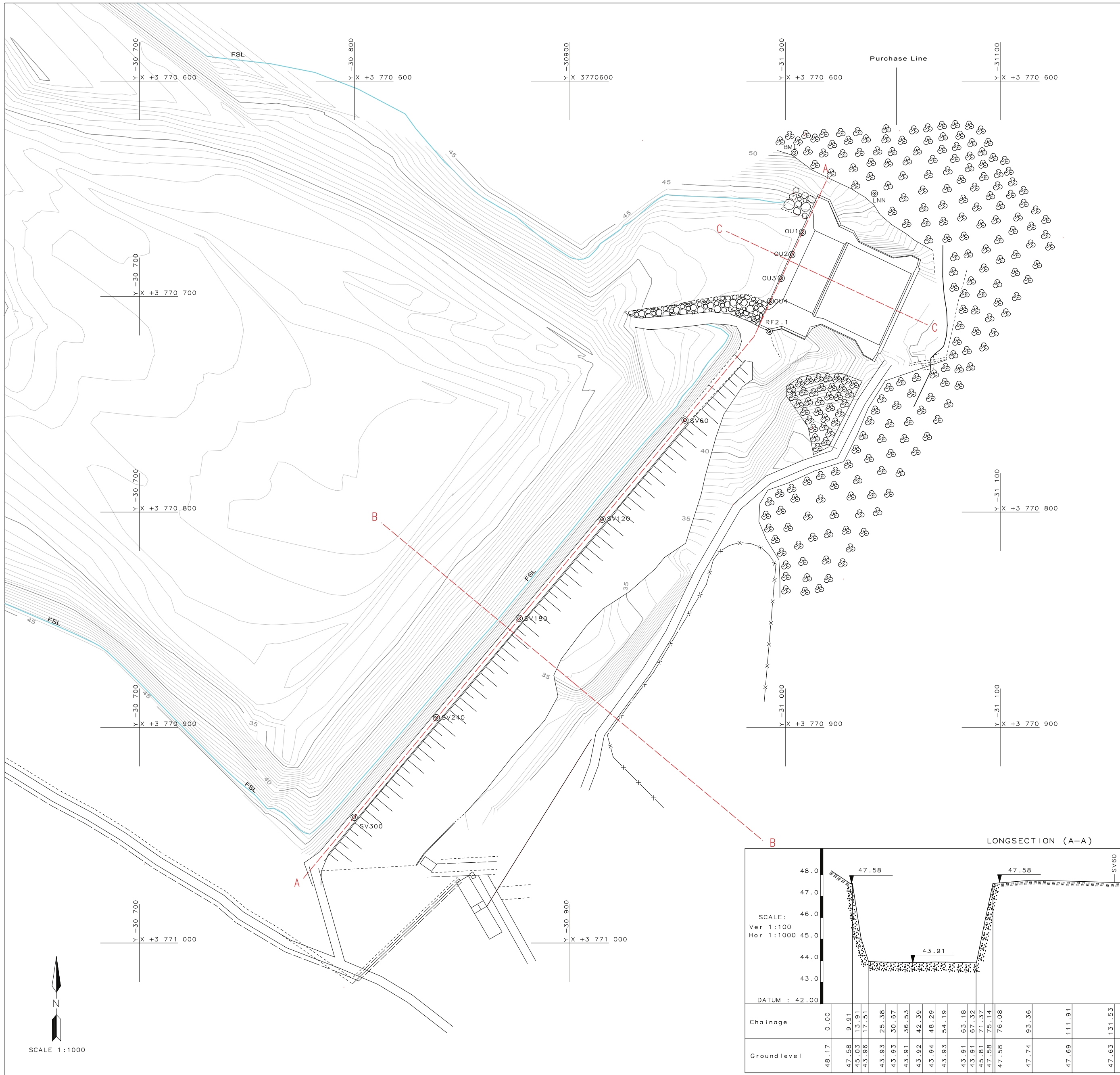
PARAMETER	VALUE
Invert Level of the pipe - RL (m	32
Diameter of pipe (m)	0.7
Length of pipe (m)	110
Reynolds number	1000000
Pipe Roughness €	0.03
Entrey losses	0.5
Exit losses	1
Gate valve losses	0.2
Butterfly valve losses	1
Discharge Coefficient	0.61
Gravitational accelaration	9.81
Surface Area of reservoir (m2)	371000
Total height of the embankmer	10.3

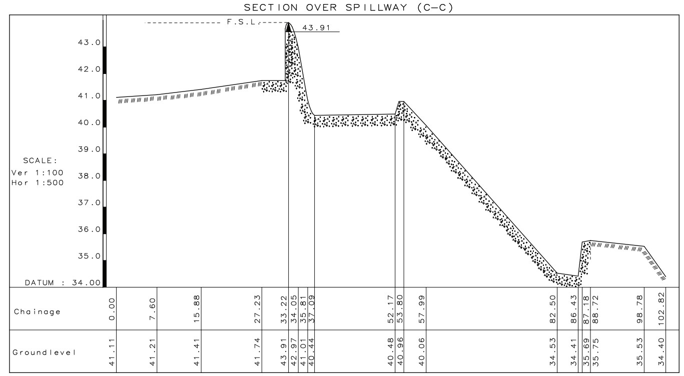
Installed drawdown rate - Qd	
m^3/s	2.11
%H/day	4.8
mm/day	491
Top 33% of H (m)	3.4
RL of 33%H	38.9
Time to empty top 33% of H	7.79
Water Level Height - H (m)	18
Total length of the embankment at base - L (m	20
Hydraulic Gradient (I) - H/L	0.9

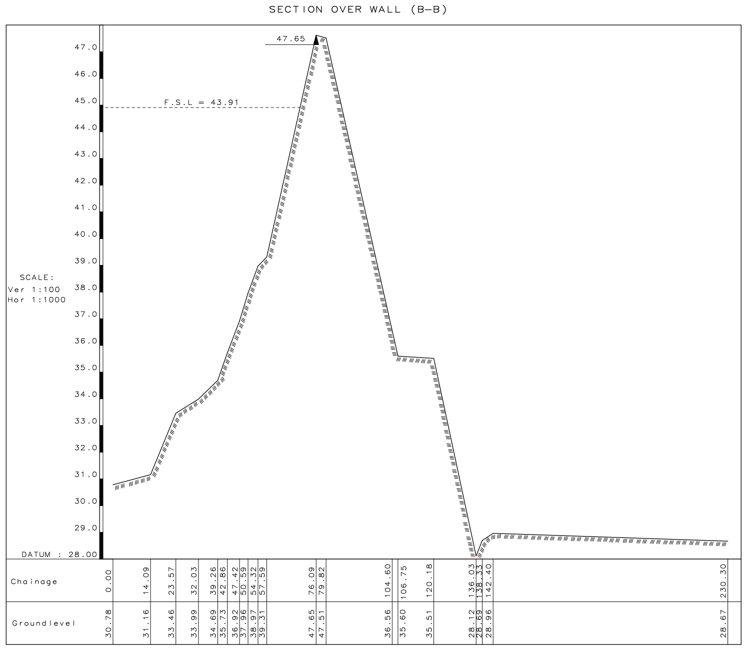
Resevoir Water Level	Resevoir Volume below depth	Increamental Volume	Average Head over outlet for the depth	Inflow pass-through allowance	ORIFICE FLOW			PIPE FLOW		
					Average discharge rate	Time to empty this depth increame	Cumulative time	Average discharge rate	Time to empty this depth increament	Cumulative time
m	*m^3*	*m^3*	*m*	*m^3/s*	*m^3/s*	*Hrs*	*Days*	*m^3/s*	*Hrs*	*Days*
42.3	2063000	108000	9.8	0	3.26	9.22	0.38	2.11	14.24	0.59
42	1955000	175000	9.4	0	3.19	15.25	1.02	2.06	23.56	1.57
41.5	1780000	170000	8.9	0	3.10	15.22	1.65	2.01	23.52	2.55
41	1610000	154000	8.4	0	3.01	14.19	2.25	1.95	21.93	3.47
40.5	1456000	146000	7.9	0	2.92	13.88	2.82	1.89	21.44	4.36
40	1310000	144000	7.4	0	2.83	14.14	3.41	1.83	21.85	5.27
39.5	1166000	131000	6.9	0	2.73	13.32	3.97	1.77	20.58	6.13
39	1035000	125000	6.4	0	2.63	13.20	4.52	1.70	20.39	**6.98**
38.5	910000	115000	5.9	0	2.53	12.65	5.04	1.63	19.54	7.79
38	795000	109000	5.4	0	2.42	12.53	5.57	1.56	19.36	8.60
37.5	686000	106000	4.9	0	2.30	12.79	6.10	1.49	19.76	9.42
37	580000	90000	4.4	0	2.18	11.46	6.58	1.41	17.71	10.16
36.5	490000	87000	3.9	0	2.05	11.77	7.07	1.33	18.18	10.92
36	403000	77000	3.4	0	1.92	11.16	7.53	1.24	17.23	11.64
35.5	326000	66000	2.9	0	1.77	10.35	7.96	1.15	15.99	12.30
35	260000	54000	2.4	0	1.61	9.31	8.35	1.04	14.39	12.90
34.5	206000	43000	1.9	0	1.43	8.33	8.70	0.93	12.87	13.44
34	163000	37000	1.4	0	1.23	8.35	9.05	0.80	12.91	13.98
33.5	126000	30000	0.9	0	0.99	8.45	9.40	0.64	13.05	14.52
33	96000	36000	0.15	0	0.40	24.83	10.43	0.26	38.36	16.12
32	60000									

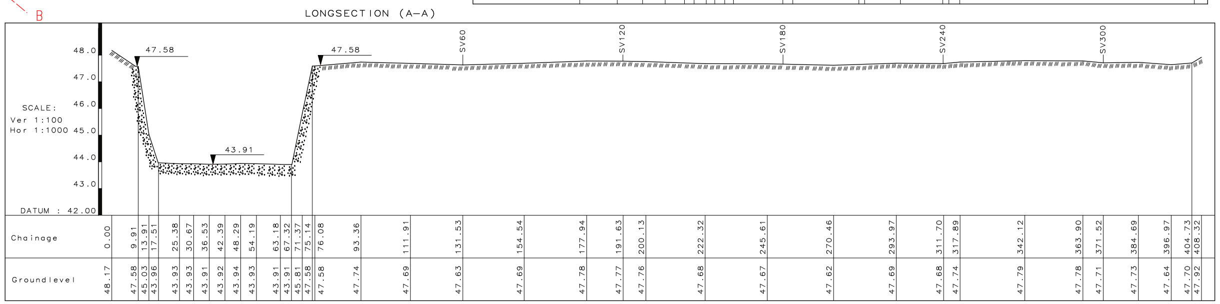

DRAWDOWN - PIPE FLOW
2500000
2000000
1500000
1000000
500000
0
VOLUME (m3)
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
16.00
18.00
TIME (Days)

Addendum H: Monitoring Data









CO-ORDINATE SYSTEM LO 23° CAPE DATUM

R.L. FROM METERS ABOVE M.S.L.

STASIE	Y	X	H.H.	STATION	Y	X	R.L.
◎LNN	−31 041.35	+3 770 652.19	47.044	◎SV300	−30 800.	+3 770 942.	47.731
◎BML1	−31 004.25	+3 770 633.28	51.570				
◎OU1	−31 007.95	+3 770 670.19	43.908				
◎OU2	−31 003.05	+3 770 680.55	43.897				
◎OU3	−30 998.11	+3 770 691.51	43.921				
◎OU4	−30 993.09	+3 770 702.18	43.915				
◎RF2.1	−30 992.53	+3 770 761.05	47.569				
◎SV60	−30 953.40	+3 770 757.34	47.666				
◎SV120	−30 914.93	+3 770 803.35	47.689				
◎SV180	−30 876.45	+3 770 849.36	47.704				
◎SV240	−30 837.95	+3 770 895.41	47.716				

SHEET LAYOUT

1

0 10 20 40 60 80 100 120 140 m

PLAN SCALE 1:1000

SOUTHERN OPERATIONS (NWRI)

PO BOX 5501
WALMER
6065

SURVEYED: SOUTHERN OPERATIONS (NWRI)	11/19	TRACED: SOUTHERN OPERATIONS (NWRI)	02/20
COMPILED: SOUTHERN OPERATIONS (NWRI)	02/20	TOPOGRAPHY CHECKED:	
PHOTOGRAPHY:		CADASTRAL CHECKED:	

No.	REVISION	DATE

REPUBLIC OF SOUTH AFRICA

DEPARTMENT WATER AND SANITATION

ROODEFONTEIN DAM

DETAIL SURVEY OF DAMWALL

DISTRICT	KNYSNA	SHEET 1 OF 1		LOC. NO. K600	REG. NO.
AREA	Ha	CODE			

Addendum I: Stability Analysis

1. Soil Parameters

The relevant soil properties for stability analysis are strength parameters; Cohesion, Angle of frication and Density. Strength parameter values used in stability analysis for Roodefontein Dam were obtained from appendix 4 of a 2002 Design Report (*Roodefontein Dam: Raising the full supply level of the dam by 2.0 m by means of solid raising*) and section 4.4.1 of the 1987 Design Report. These parameters are summarized in Table 1.

Table 1: Shear strength parameters - Original Embankment

SOIL PROPERTIES USED FOR STABILITY ANALYSIS				
	Unit Weight (kN/m^3)		Cohesion	Angle of Friction
	Saturated	Unsaturat	C	ϕ
Core Material 1	22	20	5	24
Core Material 2	22	20	5	24
General Fill	22	20	2	24
Rockfill/Gravel Toe	22	20	0	40
Sand Filter	22	20	0	35
Foundation Material	18	16	0	30

2. Analysis Criteria

The main criteria for Ultimate Limit State (ULS) design is governed by equilibrium principle and that is; driving moments (M_A) should be less or equal to the resisting moments (M_R). This criterion is expressed quantitatively in equation 1.

$$M_A \leq M_R \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$Safety\ Factor\ (F) = \frac{M_R}{M_A} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The second criteria is based on USBR (United States Department of Interior Bureau of Reclamation) Design Standard No. 13, Chapter 4, section 4.2.4 and (ADEDCR) Advance Dam Engineering for Design, Construction and Rehabilitation, page 275. The ADEDCR criteria is summarized in Table 2 and the USBR criteria is in Table 3.

Table 2: Safety Factor Criteria as per ADEDCR.

SAFETY FACTOR CRITERIA - ADEDCR		
Loading Condition	Side of embankment	Minimum Safety
1. End of Construction	Upstream	1.25
	Downstream	1.25
2. End of Construction + earthquake Loading	Upstream	1.0
	Downstream	1.0
3. Steady Seepage at Partial Pool	Upstream	1.5
	Downstream	1.5
4. Steady Seepage at Partial Pool + Earthquake Loading	Upstream	1.25
	Downstream	1.25
5. Rapid drawdown	Upstream	1.25
6. Rapid drawdown + Earthquake Loading	Upstream	1.0

Table 3: USBR Safety factor criteria (Ashok Chung, P.E, 2011).

SAFETY FACTOR CRITERIA - USBR			
Loading Condition	Shear Strength parameters	Pore pressure characteristics	Min SF
1. End of Construction	Effective	Generation of excess pore pressures in embankment and foundation materials with laboratory determination of pore pressure and monitoring during construction.	1.3
		Generation of excess pore pressures in embankment and foundation materials where there is no field monitoring during construction and no laboratory determination determination.	1.4
		Generation of excess pore pressures in embankment only with or without field monitoring during construction and no laboratory determination.	1.3
	Undrained Strength		1.3
2. Steedy State Seepage	Effective	Steady-state seepage under active conservation pool	1.5
3. Operational Conditions	Effective or Undrained	Steady-state seepage under maximum reservoir level (during a probable maximum flood)	1.2
		Rapid drawdown from normal water surface to inactive water surface	1.3
		Rapid drawdown from maximum water surface to active water surface (following a probable maximum flood)	1.2
4. Other	Effective or Undrained	Drawdown at maximum outlet capacity (Inoperable internal drainage; unusual drawdown)	1.2
		Construction modifications (applies only to temporary excavation slopes and the resulting overall embankment stability during construction),	1.3

3. Design Standards & References:

a) USBR Design Standard No.13, Chapter 4: Static stability analysis.

b) Craig's soil mechanics, 8th ed, Chapter 12: Stability of self-supporting soil masses.

c) Advance Dam Engineering for Design, Construction and Rehabilitation.

d) 2012 SANCOLD guidelines on Freeboard (to select earthquake loads).

4. Analysis Methodology

a) Fellenius/Swedish (ordinary) method of slices based on limit equilibrium is preferred for analysis in this fourth dam safety evaluation. The method is quantified by equation 3 below.

$$F = \frac{C'L_a + \tan\emptyset' \sum_i (W_i \cos\alpha_i - U_i l_i)}{\sum_i W_i \sin\alpha_i} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (3)$$

b) Using Fellenius Method of analysis, safety factor of the downstream slope was determined and then a computer Program called SLIDE@ (a product of Rock-science) was used to perform detailed analysis of different scenarios.

c) The results obtained using hand calculations were compared with those of SLIDE@ to verify the accuracy of the Program.

d) SLIDE was thereafter used to evaluate safety factors for both the upstream and downstream slopes of the embankment under different load cases.

e) Results based on Bishop's Method and Fellenius Method was compared as shown in Table 5.

5. Input Parameters & Loads

The trial slip circle used to determine the safety factor for the downstream slope is shown in Figure 1 and the used input parameters relevant to the slip circle are listed below.

5.1. input parameters/variables

Downstream Slope...………..;$\beta_D = 1:2$

Upstream slope …………….;$\beta_U = 1:3$

Radius………………………..;$R = 30\ m$

Arc Length…………….…….;$L_a = 26.2\ m$

Radius Angle……………….;$\theta = 50°$

Unit weight of water……..….;$\gamma_w = 10\ kN/m^3$

Unit weight of soil…………..;$\gamma_s = 20\ kN/m^3$

Cohesion ………………...….;$C = 2\ kPa$

Angle of friction…………..….;$\emptyset = 24°$

5.2. Load cases

The following load cases were evaluated:

a) Steady state seepage.
b) Rapid drawdown.
c) Steady state combined with Earthquake.
d) Rapid drawdown combined with earthquake.

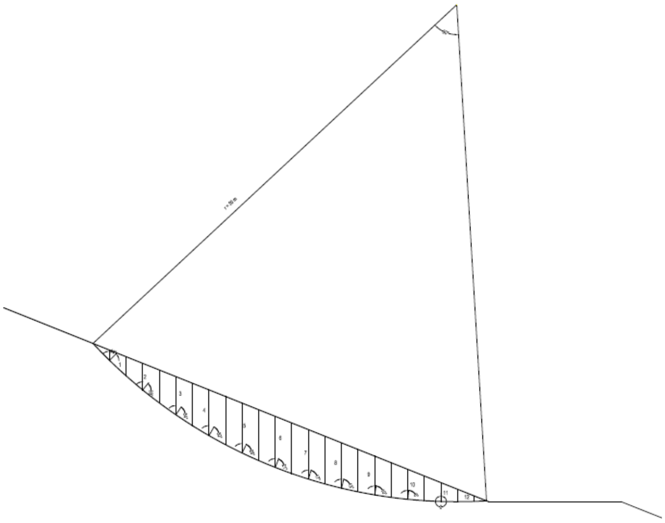


Figure 1: Trial slip circle

5.2.1. Earthquake loading

Earthquake prevalence in the Western Cape Province is relatively low. Figure 2.8-1 in 2012 SANCOLD guidelines forms a basis for selecting gravitational acceleration (g) for the dam, this figure is reproduced and shown in Figure 2 below for convenience of the reader. Gravitational acceleration for Roodefontein Dam was selected as 0.085g.

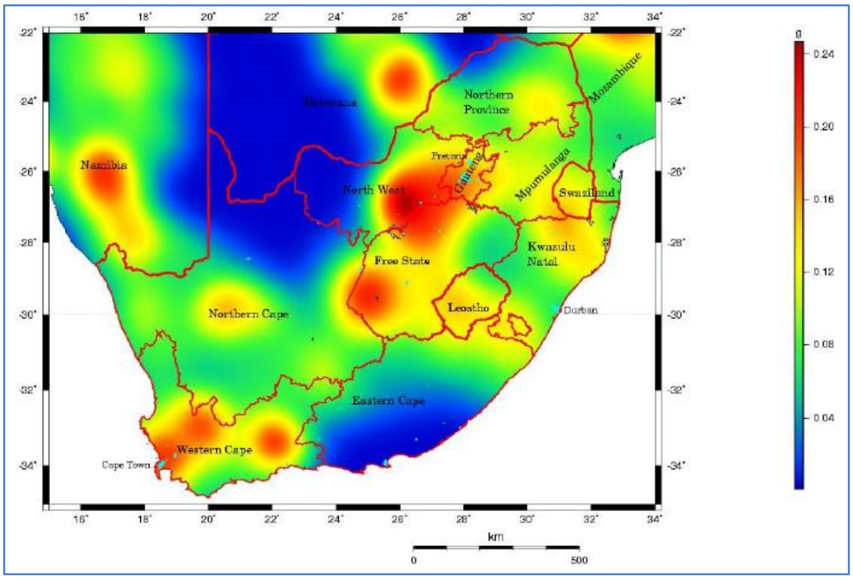


Figure 2: Seismic Hazard map for Council of Geoscience (Bosman, D.E etl, 2012)

6. Assumptions

The Fellenius/Swedish solution assumes that the resultant force for inter-slice is zero.

7. Calculations

Table 4 shows contribution of each slice to fulfill equation 2 and their summary is as follows:

$$F = \frac{C' L_a + \tan \emptyset' \sum_i (W_i \cos \alpha_i - U_i l_i)}{\sum_i W_i \sin \alpha_i}$$

$$F = \frac{2\ x\ 26.2 + \tan 24\ x\ 936.08}{348.44}$$

$$F = \frac{469.17}{348.44}$$

$$\boldsymbol{F = 1.35}$$

Table 4: Calculation results

Slice No.	**h_i (m)**	**b *(m)***	**α_i (°)**	**α_i (rad)**	**W_i *(kN/m)***	**l_i *(m)***	**γ_w (m)**	**u_i *(kPa)***	**($W_i \cos\alpha_i$ - $u_i l_i$) *kN/m***	**$W_i \sin\alpha_i$ *(kN/m)***
					$\gamma_s b_i h_i$	$b/\cos\alpha_i$		$\gamma_w Z_w$	α_i (rad)	α_i (rad)
1	0.62	2	44.3	0.77	24.8	2.79	0	0	17.75	17.32
2	1.6	2	39.2	0.68	64	2.58	0	0	49.60	40.45
3	2.3	2	34.4	0.60	92	2.42	0	0	75.91	51.98
4	2.76	2	29.9	0.52	110.4	2.31	0	0	95.71	55.03
5	3	2	25.6	0.45	120	2.22	0	0	108.22	51.85
6	3.1	2	21.4	0.37	124	2.15	0	0	115.45	45.24
7	3	2	17.4	0.30	120	2.10	0	0	114.51	35.88
8	2.74	2	13.4	0.23	109.6	2.06	0	0	106.62	25.40
9	2.34	2	9.5	0.17	93.6	2.03	0	0	92.32	15.45
10	1.81	2	5.6	0.10	72.4	2.01	0	0	72.05	7.07
11	1.1	2	1.8	0.03	44	2.00	0	0	43.98	1.38
12	0.35	2	-2	-0.03	14	2.00	0	0	43.98	1.38
								TOTAL	**936.08**	**348.44**

Table 5 below, shows the calculated safety factors for six (6) different load cases.

Table 5: Slope stability calculation - results

EMBANKMENT SAFETY FACTORS					
Side of Slope	Load Condition		Safety Factors		Limit
			Fellenius	Bishops	
Downstream	FSL	Load case 1	1.27	1.30	1.5
	FSL + EQ	Load case 2	1.00	1.03	1.25
Upstream	FSL	Load case 3	1.56	1.7	1.5
	FSL + EQ	Load case 4	0.88	1.11	1.25
	RDD	Load case 5	0.96	1.07	1.25
	RDD + EQ	Load case 6	0.64	0.74	1.0

8. Evaluation Of The Results

Stability results as shown in Table 5 reveal that Fellenius Method is more conservative than Bishops Method of slices. All slopes under load case 1 to case 5 satisfy principles of equilibrium when evaluated using Bishop's Method. The only condition satisfying USBR and ADEDCR criteria is load case 3 with Safety factor of 1.56 and 1.7 using Fellenius and Bishop Method, respectively. The other conditions do not comply with the two criteria. Fellenius results for load case 4 to 6 suggest that Roodefontein Dam will experience instability when there is an Earthquake or when the dam is rapidly drawn down.

Addendum J: Registration Information

#Name?

Department of Water and Sanitation - Dam Safety Office

Registration Details of a Dam Registered in terms of Dam Safety Legislation of Chapter 12 of the National Water Act (Act No. 36 of 1998)

(Please note that registration for dam safety legislation is not an entitlement for water use in terms of Chapter 4 of the National Water Act)

Departmental File No. : 12/2/ K602/02 *WARMS Dam ID:*

Water management area 8 *Dam Status:* REG *Drainage Nr:* K60G

Name of dam ROODEFONTEIN DAM

Latitude 34 4 1 *Longitude* 23 20 1 *Lat sec:* 1.00

Long sec: 1.00

Town nearest: PLETTENBERG BAY

Distance from town (km) 2 *WMA* Breede-Gouritz

Name of farm ROODEFONTEIN 440 PORTION 82

Magisterial District KNYSNA

Province: WESTERN CAPE *Water Management Region:* WESTERN CAPE

Date of completion 1989

Raising or Alteration Date 2004

River PIESANG

Wall type EARTHFILL

Wall height (m) 18

Crest length (m) 315

Spillway UNCONTROLLED OGEE

Capacity (1000 cub. m) 2003

Surface area of water (ha) 37 *Catchment area (sq km)* 28

Purpose DOMESTIC SUPPLY & INDUSTRIAL USE

Owner	*Person in Control (if not the same as the owner)*
	DIRECTOR: SOUTHERN OPERATIONS
DEPT. OF WATER & SANITATION	MR. H. W. GELDENHUYS
PRIVATE BAG X 313	BITOU MUNICIPALITY
PRETORIA	PLETTENBURG
0001	6600
Tel no. (012) 336 7500	*Tel no.* (044) 501 3265
Cell no.	*Cell no.*
Email / Fax	*Email / Fax*

Designer NINHAM SHAND (CAPE) INC.

Contractor HERBST BROS (PTY) LTD.CERES

Registration date: 1/29/1990 *Status* Dam Registered as a Dam with a Safety Risk

Size Medium *Hazard Rating:* High *Category* 3

Classification date: 9/2/1987 *Date Last DSE* 8/16/2010

Date Completion Report: 3/15/2005 *Number Last DSE:* 3

Addendum K: Mechanical Report

ROODEFONTEIN DAM

DAM SAFETY INSPECTION OF MECHANICAL AND ELECTRICAL PLANT AT WESTERN CAPE REGION

The operation, maintenance and general condition of the mechanical and electrical plant and equipment was evaluated on 11TH February 2020.

INSPECTION

The outlet works were found to be in a reasonable operating condition and capable of performing the function of which they were intended. The required maintenance was not performed due to the fact that there's no maintenance contract in place.

RECOMMENDATIONS

The following recommendations are made with regard to the maintenance of the equipment.

		Priority	Picture
	Levels of Priority		
	Rectify as soon as possible	1	
	Rectify within two years	2	
	Rectify within five years	3	
	Rectify as soon as opportune	4	
	No action required	5	
1	**ISOLATING VALVES – GATE VALVES**		
1.1	The shaft on the insolating gate valve is bent and it is difficult to operate. It must be repaired or replaced.	1	A1
1.2	Upstream gate valve is not sealing. It must be inspected and repaired.	2	A2
1.3	The indication on the wheel (open-close) is missing and must be rectified.	1	A3
2	**CONTROL VALVE – SLEEVE VALVE**		
2.1	The front seal rubber on the sleeve valve is cut and must be replaced.	1	
2.2	The inside of the sleeve valve has plenty of visible corrosion marks. Some paint is gone and there are plenty of blisters. General condition of the valve is poor and must be cleaned and refurbished.	1	B1, B2
2.3	The mild steel grease nipple on the sleeve valve must be replaced with stainless steel ones.	1	
2.4	Open and closed indicators are missing on the sleeve valve wheels. Indicators must be added.	1	B3, B4
3	**OUTLET PIPES**		
3.1	The paint is flaking off from under water manifold with five inlets the corrosion protection must be repaired.	2	C1
3.2	The sacrificial anodes on the manifold must be checked and corroding anodes holders must be repaired.	1	
3.3	The rusted mild steel pipe protruding out of the concrete in the valve house has been refurbished from outside, lining has been spot repaired with Hycote 151.	5	
3.4	New possibilities must be investigated for the manifold outlet works.	3	
4	**GENERAL**		
4.1	The log books must be introduced as soon as possible to ensure that a good maintenance program is in place.	1	
4.2	The drawings and information on the new outlet works should have been submitted to the Directorate Strategic Asset Management for approval before the extension were done. Drawings must be submitted to update the departmental records.	1	

4.3 The technical information and a copy of the operating manual must be submitted to Strategic Asset Management. 1

4.4 Roof damage and security doors must be fixed to ensure the outlet works room remains dry and secure. 1

5 OPERATING AND MAINTENANCE PROCEDURES

5.1 To enable the establishment and maintaining the required high standards of the condition of the dams the final responsibility of the dams must be clearly defined.

5.2 It is considered to be the responsibility of the Operational Personnel to implement the use of and ensure adherence to the procedures specified in the logbook and to inform the main maintenance personnel of the faults and maintenance procedures specified in the logbook.

5.3 Inspections by Competent Artisans must be done at regular intervals as also called for in Logbooks to ensure reliability of the plant and equipment and to comply with the Occupational Health and Safety Act.

5.4 Provision is made in the logbooks for senior (Supervisory) Personnel to certify that the condition of the plant recorded in the Logbooks is a true reflection of the condition of the plant and equipment. This is essential to ensure that the plant and equipment is correctly operated and adequately maintained and complies with the Occupational Health and Safety Act at all times.

5.5 All lifting equipment to be performance tested every twelve (12) months to comply with the regulations in terms of section 43 of the OHS Act.

J. KOLAROVIC
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STRATEGIC ASSET MANAGEMENT
DATE: 04-03-2020

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