Client:	**Ngodwana Mill - SAPPI Paper and Paper Packaging SAPPI Southern Africa Ltd**
Ref #:	**Order # 2105-130638**
Project Description:	**2019 ANNUAL DAM SAFETY INSPECTION**
Date:	**30 September 2019**

NGODWANA DAM
ANNUAL DAM SAFETY REPORT
30 September 2019

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Approvals

Title:	**Ngodwana Mill - SAPPI Paper and Paper Packaging** **SAPPI Southern Africa Ltd** **Order # 2105-130638** **2019 Annual Dam Safety Inspection Report**
Author:	Altus de Beer PrEng [Approved Professional Person]
Status of Report:	Final
Company Reference:	Ngodwana Dam
Keywords:	

Approved by "Approved Professional Person":

30 September 2019

Altus de Beer PrEng
APP – Ngodwana Dam

Date

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Accepted by Owner:

Date

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Executive Summary

GENERAL

The 41,2 m high Ngodwana Dam, completed in 1983, is located on the Ngodwana River, a tributary of the Elands River, Mpumalanga Province, directly upstream from the N4 highway and the SAPPI Ngodwana Paper Mill, 40 km from Nelspruit.

The dam is classified with the Department of Water Affairs [DWA] Dam Safety Office as a Category III High Risk dam, in terms of the Dam Safety Regulations, Chapter 12 of the National Water Act 1998 (Act 36 of 1998).

DAMWALL

The dam wall was designed as an embankment dam with a sloping clay core and an un-zoned sloping chimney filter, connected to horisontal strip drains that terminates in collection toe-drains fitted with V-notch flow gauges. The upstream slope is 1:2,5 and the downstream slope is 1:2,0 and it has two equidistant berms.

SPILLWAY

The spillway is constructed on the right bank and consists of a 90 m wide OGEE control structure which is currently 5,62 m deep (381 mm loss of freeboard from the original 6 m freeboard, due to settlement of the crest), that discharges into a 140 m long reinforced concrete return channel that terminates in a plunge pool. There is also an effective 200 m long breaching section on the right flank, which is 1,4 m lower than the non-overspill crest. The total spillway discharge capacity (without the breaching section) is 2,563 m^3/s.

OUTLET WORKS

The outlet works consist of a reinforced concrete dry intake tower, with intakes at three levels. The intake tower is connected to a dry, reinforced concrete conduit, located under the dam wall, through which access is gained from the downstream toe of the wall. The conduit is partitioned with a central support wall to create two sub-conduits (left and right). There are two outlet pipes, each fixed to the bottom of the two sub-conduits.

FOUNDATION GEOLOGY

Ngodwana Dam wall is underlain by very complex and challenging foundations due to the occurrence of a variety of sedimentary, volcanic and metamorphic rocks, unconformities in the sedimentary succession, faulting, variable weathering, thick alluvium in the river section, colluvium on the left flank and large dislodged quartzite blocks on the right flank.

DAM WALL CONSOLIDATION, SETTLEMENTS & SLOPE STABILITY

Ngodwana Dam wall displayed large initial vertical and horizontal settlement, but settlements have now stabilised. Maximum vertical settlement (Beacon 7) of 200 mm occurred within the first year (1983-1984) after completion, then another 200 mm settlement during the next 15 years (1984-1999) and only 37 mm during the past 18 years (1999-2019). Total current vertical settlement is 439 mm.

Horizontal movement of the dam crest (downstream) under the water load, exactly followed the vertical settlement pattern. Total maximum horizontal movement (Beacon 6) is currently 246 mm, having stabilised to only 20 mm horizontal movement over the past 15 years (2004-2019).

The portion of the embankment between the riverbed and the left flank foundation contact showed significant erratic movements over the past 5 years, e.g. beacon SB15 on the crest near the foundation contact, underwent vertical settlement of 22 mm and horizontal (downstream) movement of 12 mm. However, during 2017 there was a vertical rebound (upwards) of 11 mm. The precision survey of 5'th-9'th August 2019 revealed that the entire extreme left portion of the embankment settled vertically by 7 mm and horizontally (downstream) by between 7-14 mm.

In the previous (2018) report, an additional analysis was conducted for the downstream slope stability. This analysis (Annexure I) found that the downstream slope does not meet the safety requirements i.r.o. the slope stability factor of safety (factor of safety = 1,5) as prescribed by the USBR Guidelines (Table 11.1) for a dam wall with this hazard potential and safety classification.

SEEPAGE

Measured seepage through the embankment and under the dam wall has also significantly decreased from an initial 4.5 l/s to 0.470 l/sec in 2019. There is a small amount of additional unmeasured seepage in the open drain below the toe of the wall on the left flank. Maximum total seepage is estimated at less than 0,5 l/s. The initial large settlements and the subsequent decrease in settlement and seepage are ascribed to large-scale consolidation of the very thick, initially unconsolidated, foundations.

INSTRUMENTATION

The dam wall was initially equipped with adequate dam safety monitoring instrumentation, namely settlement beacons, V-notches for gauging seepage, standpipe piezometers for measuring the phreatic surface and pneumatic piezometers for gauging pore pressure in the upstream embankment. The pneumatic piezometers have in the meantime failed and the observed behaviour of the dam wall, in particular on the left bank where the "wet spot" also occurs, necessitated the installation of additional standpipe piezometers and a comprehensive analysis of the seepage through the dam wall as well as the slope stability of the downstream zone.

MAINTENANCE

The dam wall is well maintained by the owner and the owner arranges for regular instrumentation gauging to be undertaken by specialist contractors. The owner also arranges for annual interim dam safety inspections, in addition to the compulsory and comprehensive 7-yearly inspections.

ANNUAL DAM SAFETY INSPECTION

In this report, for which the inspection was held on Friday 13 September 2019, the following key recommendations are made.

SAPPI should maintain the safety of Ngodwana Dam by:

1. Promptly undertaking measures to improve the stability of the downstream slope of the embankment.
2. These measures should commence with a study to determine the most feasible and cost-effective way to achieve the desired slope stability. The desired slope stability can be achieved by adding a stabilizing berm at the toe of the dam with a filter between the berm and the existing embankment. The study would be required to procure materials and size the berm.
3. With regards to other dam safety issues, it should be mentioned that the Owner generally maintains Ngodwana Dam in a good condition. The

responsible person, Mr Carel van der Merwe, diligently kept records and ensured the proper maintenance of the dam.

4. Ongoing monitoring and maintenance activities should be continued, namely:
   i. monthly reading of the water table in the standpipe piezometers;
   ii. weekly gauging of seepage flow through V notches;
   iii. annual monitoring of dam wall deflections through precision survey of deflection beacons;
   iv. the embankment material leakage as well as the water seepage through the construction joints of the outlet conduit should be carefully monitored;
   v. termite nests seems to have been effectively poisoned but they should be regularly monitored for any further activity;
   vi. the Warning & Evacuation Plan should contain all the necessary warning and evacuation procedures as well as updated contact numbers of all affected people and organisations. The Plan is to be kept in an accessible location, which should be known to all key safety operational staff at SAPPI;
   vii. the SAPPI safety operational staff should, at least once per year, check whether they are able to contact all relevant affected individuals in a timely manner to take appropriate action in the event of a "dam break flood" event; and
   viii. the early warning and evacuation plan should be maintained and practised, in the event that a dam failure risk is detected or experienced.

CONTENTS

LIST OF APPENDICES

1 OWNER AND DAM SAFETY TEAM

The current team of dam safety specialists have been involved with safety inspections on Ngodwana Dam since at least the 3'rd compulsory inspection in July 1998. The current team is:

Designation	**Name**	**Contact Information**
Owner	Ngodwana Mill - SAPPI Paper and Paper Packaging SAPPI Southern Africa Ltd	
Owner's Representative	Carel van der Merwe	Carel.VanDerMerwe@sappi.com cell: 083 700 8733
APP and Structural Specialist	Altus de Beer PrEng	debeeraltus@gmail.com cell: 083 700 8733
Engineering Geologist	Prof A (Monte) van Schalkwyk	montevs@absamail.co.za cell: 083 922 3337
Flood Hydrology and Spillway Hydraulics Specialist	Dr WV (Bill) Pitman	pitmanwv@iafrica.com cell: 082 330 4630
Mechanical Engineer	Andre du Plessis PrEng (AR du Plessis & Associates)	andre@ardpconsulting.com cell: 082 850 4417
Settlement Beacon Surveyor	Peter Barnard (Barnard & Schneider)	sandd@tiscali.co.za tel: 011 704 0735
Piezometers	Carel van der Merwe SAPPI Ngodwana	Tel.: 013 734 4771 Cell.: 082 876 7496 Carel.VanDerMerwe@sappi.com

2 AVAILABLE INFORMATION

2.1 LOCALITY MAP

See Appendix A for maps and drawings.

2.2 DAM WALL LAY-OUT & SECTIONS DRAWINGS

See Appendix A for maps and drawings.

2.3 LIST OF PREVIOUS REPORTS, DRAWINGS, ETC.

2.3.1 Original Design Reports (as listed in 1987 DS Report)

2.3.1.1 Geology

1. Webb and Partners; The geology at the site of a proposed dam for Sappi Fine Papers Limited - Ngodwana, Report 8027, June 1981, 9pp.
2. Brink and Matthews; Engineering geological assessment of foundation conditions below left flank of proposed embankment dam Ngodwane River, Report LOC BM 72, February 1982, 11pp.
3. Seismic Survey (Pty) Ltd; Report on seismic refraction survey - Ngodwane Dam Project - Eastern Transvaal, Job 504/03, May 1982 5pp.
4. Matthews and Associates: Engineering geological assessment of foundation' conditions within stilling basin and cut-off of left flank - Embankment Dam - Ngodwane River, Report M20/6/82, July 1982, 8pp.
5. Matthews, and Associates; Ngodwane Dam - Report on spillway slope stability, Report M20/7/83, June 1983, 8pp.
6. Report on the geology of the foundations of the Ngodwane Dam built for Sappi Fine Papers Limited, Ngodwana. Report prepared by Matthews and Associates, Engineering Geologists and Geotechnical Engineers. Report M40/8/83, August 1983.

2.3.1.2 Hydrology

1. Water Resources. Report by Prof D C Midgley. (Appendix 1)

2. Floods. Report by Prof D C Midgley. (Appendix 1)

3. Files. B G A Lund & Partner.

2.3.1.3 Dam Design

1. Files. B G A Lund & Partner.

2. Soil test results. Files, B G A Lund & Partner and W H Luwes Pr.Eng. who carried out tests.

3. Paper "The Design of Earth Embankments with reference to the Choice of Shear Strength Parameters and Control of Earth Fill Operations", by B G A Lund. 11th African Regional Conference on Soil Mechanics and Foundation. Engineering 1963.

4. Estimating Construction Pore Pressures in Rolled Earth Dams. H W Hilf, USBR.

5. SLOP 4. Computer Program for Slope Stability Analysis, CSIR.

6. Earth dam design report prepared for this study. (Appendix 2)

7. Monitoring. of Ngodwane Dam. Report prepared for this study. (Appendix 3)

2.3.1.4 Drawings

These are new drawings or adapted from existing drawings.

Drawing No	Description / Title
SK/1/1	Project site plan
SK/1/2	Leakage survey
SK/1/3	General arrangement
SK/1/4	Monitoring section through standpipe piezometers
SK/1/5	Dam break study - area inundated by flood
SK/1/6	Breaching section (Computations in Appendix 4)
SK/1/7	Monitoring - leakage study
SK/1/8	Test result of monitoring
SK/1/9	Monitoring of dam settlement
SK/1/10	Plan - cut off trench, grout curtain and monitoring pts.

SK/1/11	Longitudinal Section - Ngodwane, Elands & Crocodile Rivers.

2.3.1.5 Specifications

Files. B G A Lund & Partner.

2.3.1.6 Additional Drawings Listed in 1998 DS Report

These are new drawings or adapted from existing drawings.

Drawings No	**Description**
SK1/1	Project site plan
Sk1/2	Leakage survey
SK/1/3	General arrangement
SK/1/4	Monitoring section through standpipe piezometers
SK/1/5	Dam breaks-study area inundated by flood
SK/1/6	Breaching section
SK/1/7	Monitoring leakage study
SK/1/8	Test result of monitoring
SK/1/9	Monitoring of dam settlement
SK/1/9A	Ditto 6-94 to 6-98
SK/1/10	Plan – cut-off trench, grout curtain and monitoring pts
SK/1/11	Long section Godwana, Elands and Crocodile Rivers
SK/1/12	Remedial Work Outlet to Culvert
SK/1/13	Remedial Work Subsurface Drain (at Wet spot)
SK/1/14	Cancelled
Sk/1/15	Seepage Records
SK/1/165A	Inst. Monitoring Standpipe Piezometers
SK/1/16B	Ditto Pneumatic Piezometers Section A
SK/1/16C	Ditto Section B

SK/1/17	Spillway study 1994
SK/1/18	Spillway study (Stevens) 1:500
SK/1/19	Spillway Survey (BGA Lund) 1:500

2.4 RECENT POST-CONSTRUCTION ANALYSES

2.4.1 Flood Estimation

See Appendix C for flood estimation analysis.

2.4.2 Dam Break Flood Analysis

See Appendix C for dam break flood analysis.

2.4.3 Spillway Retaining Wall Stability

See Appendix D for spillway retaining wall stability analysis.

2.4.4 Seepage Analysis

See Appendix H for seepage analysis.

2.4.5 Slope Stability Analysis

See Appendix I for slope stability analysis.

3 BACKGROUND INFORMATION

3.1 LOCALITY

Ngodwana Dam is located on the Ngodwana River, a tributary of the Elands River, Mpumalanga Province, directly upstream from the N4 highway and the Ngodwana Paper Mill, 40 km from Nelspruit (See Locality Map under Appendix A – Drawings.

3.2 PURPOSE – COST – YIELD

The dam was completed in 1983 for water supply to the then SAPPI Kraft Ngodwana Paper Mill. At a project cost of approximately R 1.2 billion, the paper mill was one of the largest single private sector undertakings in South Africa at the time. The maximum water demand to be met for the paper mill, is 40 000 cubic meters per day. The Ngodwana River was chosen in preference to the Elands River, as it provides high quality water with a TDS of only 40 mg/l – a very desirable factor in papermaking.

4 DESCRIPTION OF THE DAM

Wall Type	Zoned embankment dam with a sloping clay core and a sloping, un-zoned chimney filter, connected to a blanket drain.
Wall Height (max)	41,20 meters
Crest Length	440 meters
Crest Width	6 meters
Crest Elevation	965,20 MASL
Toe Elevation (lowest)	924.00 MASL
Storage Capacity	10,4 million cubic meters
Dead Storage	0,52 million cubic meters
Surface Area at FSL	87 Ha
Historical Firm Yield	WR90 = 21,6 million m^3/a JIBS = 26,3 million m^3/a
Annual Abstraction	14,6 million m^3/a
Outlet Works	The outlet works consist of a dry intake tower, with intakes at three levels (two service and one emergency bottom outlet). The intake tower is connected to a dry reinforced concrete conduit through which access is gained to the intake tower.
Completion Date	The dam wall construction took place between 1981-1983. The dam cost R 10 million to complete in 1983.
Designer	The late Mr BGA Lund PrEng was both the designer of and Approved Professional Person [in terms of the dam safety regulations of Chapter 12 of the National Water Act 36 of 1998] for the dam, until his death in 2006.

Contractor	The contractor was Peter Faber (Pty) Ltd and the grouting sub-contractor was Ground Engineering (Pty) Ltd.
Betterment Works after Completion	Installation of toe drain Construction of concrete slab at culvert outlet Installation of 7 new standpipe piezometers
Problems which occurred previously	Large vertical and horizontal settlements and deflections of the embankment "Wet Spot" on downstream embankment near left flank Concerns over the origin of high water flows immediately below the embankment Concerns over high phreatic surfaces and slope stability, in particular on the left flank Failed joint sealers between spillway chute concrete slabs and concerns for undercutting erosion and "lifting" of slabs Failed water stops in construction joints between outlet conduit sections, causing leaching of adjacent embankment material into the conduits Inadequate slope stability of the downstream slope of the embankment – as determined by studies and analyses conducted during the period 2017-2018

5 GEOLOGY OF DAM SITE

5.1 GENERAL GEOLOGY

5.1.1 Embankment and Foundations

The site geology is extremely complex due to the occurrence of a variety of sedimentary, volcanic and metamorphic rocks, unconformities in the sedimentary succession, faulting, variable weathering, thick alluvium in the river section, colluvium on the left flank and large dislodged quartzite blocks on the right flank. The distribution of the material types along the dam centre line and the cut-off trench is illustrated on the attached sections in Drawing:- SAPPI 2018-04 LONGITUDINAL SECTIONS, attached hereto under Appendix A – Drawings. A summary of geological conditions as compiled for the 2009 dam safety report is included under Appendix B.

5.2 GEOLOGICAL CONDITIONS - ACTUAL OR POTENTIAL PROBLEMS

5.2.1 Left Flank

The upper part of the left flank is underlain by a 5m – 20m thick cover of partly recemented (calcified) talus blocks in a matrix of soil, resting upon a layer of completely weathered very weak tuff and agglomerate. Although the embankment in founded on these low strength and potentially permeable materials, the cut-off wall was re-aligned in an upstream direction and is largely founded on a downstream dipping layer of strong Godwan Formation quartzites. Only the upper part (mostly above FSL) of the cut-off is founded on weak tuff and partly cemented colluvium. This part of the flank had been covered by an impervious blanket, and based on seepage records, it can be concluded that a reasonably watertight cut-off had been achieved.

5.2.2 Right Flank

Along the major part of the right flank, the embankment and spillway structures are founded on a layer of large dislocated (slumped) blocks of Black Reef Formation quartzite, resting upon a thin layer of weak tuff, tuffaceous shale and agglomerate of the Godwan Formation that is underlain by strong quartzite of the same formation. The cut-off is founded partly on the strong quartzite and partly on the weak rocks overlying it. The engineering properties (deformability and permeability) of these rocks are unknown.

5.2.3 River Section

The river section is underlain by a 5m – 10m thick layer of river alluvium that is underlain by medium strong tuff, lava, hornfels and quartzite of the Godwan Formation. These rocks are closely jointed in some areas. The cut-off is founded on bedrock.

5.2.4 Spillway & Return Channel

The OGEE overspill structure and chute are founded on dislodged quartzite blocks and colluvium, while the stilling basin is located in weathered, weak tuffaceous shale. The breaching section is partly founded on the dislocated quartzite blocks and partly on colluvial soil comprising gravely sand and clay.

5.2.5 Intake Tower and Outlet Conduit

The intake tower and outlet conduit is underlain by quartzite and possibly alluvium. The presence of alluvium under the conduit needs to be verified.

5.2.6 Slope Stability Around the Dam Basin

The slopes around the dam basin are stable, i.e. no sliding failures observed.

6 DESCRIPTION OF DAM WALL MATERIAL

6.1 UPSTREAM ZONE

Clayey sand and gravel.

6.2 DOWNSTREAM ZONE

The table below contains information on soil samples taken from the 7 drill holes for the installation of the new standpipe piezometers in 2017. From these samples, most of the material was classified as stiff clayey sand with some silt and gravel.

SAMPLE DETAILS			TEST RESULTS													
BH No	Depth (m-m)	Sample No.	Clay (%)	Silt (%)	Sand (%)	Grav. (%)	PI (whole sample) (%)	GM	UCSC	Double H. Dispersion (%)	MDD (kg/m³)	OMC (%)	k (m/s)	c' (kPa)	Φ' (degrees)
SP1	14.04 – 21.00	A7/1720	29.2	15.1	48.3	7.4	13	0.81	SC	62					
SP1	21.00 – 30.04	A7/1721	20.4	11.7	60.0	7.8	10	1.01	SC		1877	12.4	$6.5x10^{-7}$	32.8	19.5
SP2	4.05 – 7. 53	A7/1722	24.0	11.3	63.6	1.2	10	0.83	SC	23.2					
SP2	8.08 – 14.14	A7/1723	24.0	11.5	63.5	1.1	9	0.86	SC		1827	10	$1.4x10^{-7}$	32.0	18.3
SP3	1.90 - 5.92	A7/1724	24.9	22.0	50.5	2.6	13	0.68	CL	26.4					
SP3	11.80 – 17.06	A7/1725	34.1	47.6	16.5	1.8	24	0.24	OH		1478	22.9	$1.1x10^{-8}$	0.4	29.0
SP4	13.00 – 19.15	A7/1726	28.9	13.9	47.7	9.6	14	0.88	SC	13.0					
SP4	19.15 – 26.15	A7/1727	23.7	9.8	65.0	1.5	9	0.91	SC		1864	11.0	$2.2x10^{-7}$	39.6	16.8
SP5	9.70 – 15.36	A7/1728	26.3	11.9	58.6	3.2	1	0.90	SC	13.7					
SP5	17.46 – 25.52	A7/1729	26.5	15.9	55.4	2.2	13	0.80	SC		1834	15.1	$3.1x10^{-9}$	15.9	25.7
SP6	1.84 – 6.55	A7/1730	27.8	15.1	54.1	2.9	13	0.81	SC	0					
SP6	7.55 – 16.00	A7/1731	26.5	12.2	55.9	5.4	11	0.89	SC		1609	15.	$5.6x10^{-7}$	13.4	20.3
SP7	2.20 – 6, 54	A7/1732	30.0	17.9	49.4	2.8	13	0.74	SC	10.5					
SP7	7.00 – 12.00	A7/1733	26.2	11.9	58.9	2.9	12	0.83	SC		1857	13.7	$4.7x10^{-9}$	33.0	24.4
AVERAGES			**26.61**	**16.27**	**53.39**	**3.74**	**11.79**	**0.8**		**21.3**	**1764**	**14.3**	$\mathbf{2.3x10^{-7}}$	**23.9**	**22.0**

6.3 CLAY CORE

Clay and silty clay.

6.4 FILTER

Un-zoned sand and gravel filter.

7 EVALUATION OF THE HAZARD POTENTIAL

Downstream development since last report?	This is difficult to estimate, but it must be presumed that the size of Mataffin Village had increased since the last compulsory inspection in 2016.
Estimate of potential loss of life	This is also difficult to estimate, but it is estimated to be significantly higher than 10.
Estimate of potential economic loss	This too is difficult to estimate, but with certainty in the hundreds of millions of Rand direct damage and possibly billions of Rand consequential damages.
Hazard potential rating as classified	Category III High Risk
Agreement with classification?	A "dam break event" of Ngodwana Dam would result in economic damage well in excess of R20 million and the probability that more than 10 lives could be lost. In compliance with SANCOLD recommendations, Ngodwana Dam is therefore classified with the Dam Safety Office of the Department of Water and Sanitation [DWS] as a Category III High Risk dam. The APP thus agree with this classification.
Checking of Registration Information	The registration information as it appears on the computer printout from the DWS Dam Safety Office corresponds with the above assessment.

8 FLOOD ESTIMATES

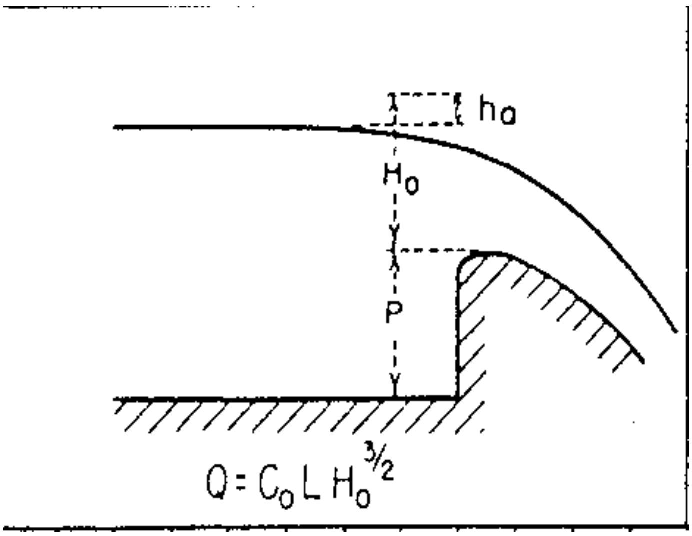
See Appendix C – Flood Hydrology Report for more detailed information.

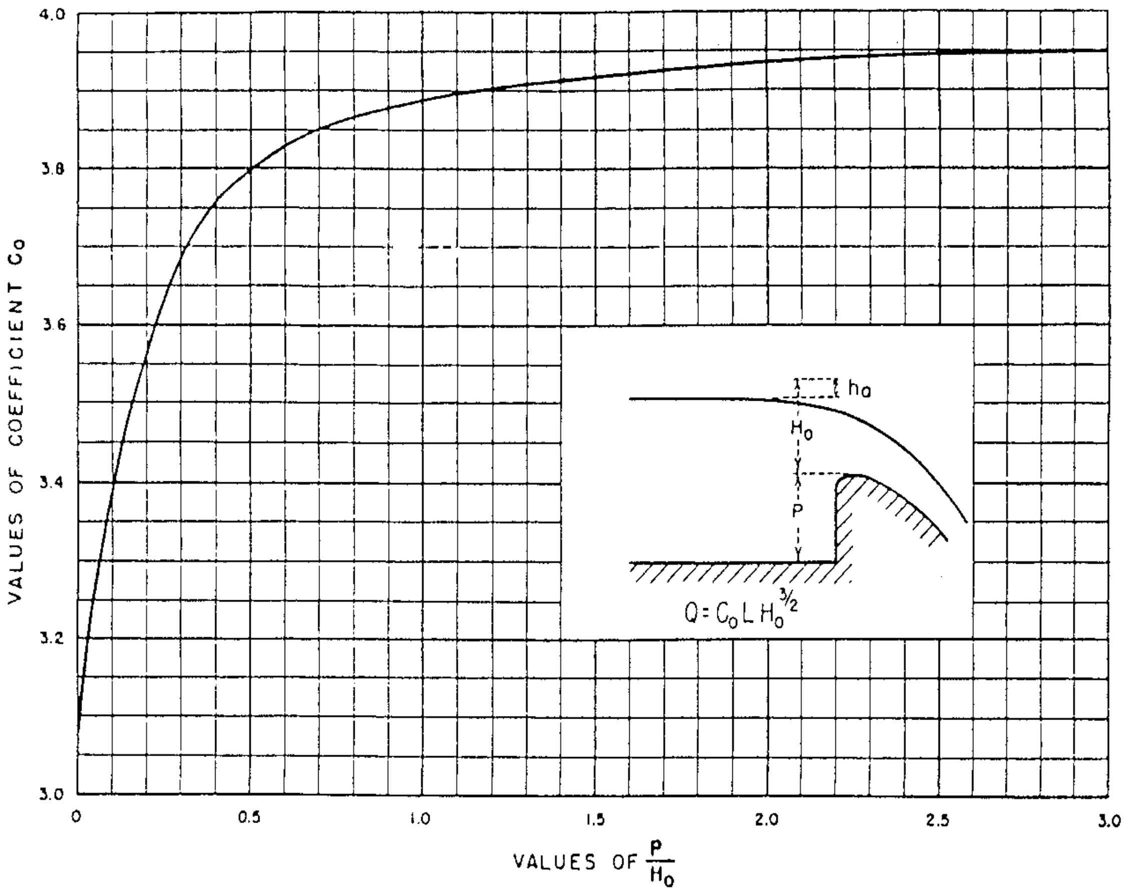
Catchment Area	229 km^2
MAP	1068 mm
Methods Used for Flood Estimation	HRU Region HRU Formula Rational Method TR137 SDF
1:20 (m^3/s)	338
1:50 (m^3/s)	496
1:100 (m^3/s)	650
1:200 (m^3/s)	832
RMF (m^3/s)	1,330
PMF (m^3/s)	2,563
RDF (m^3/s)	800
SEF (m^3/s)	1,330
Motivation for choice of RDF & SEF	Weighted Average of HRU Region, HRU Formula, Rational Method, TR137, SDF

9 EVALUATION OF SPILLWAY CAPACITY

Spillway Type	Mass concrete uncontrolled OGEE spillway on right bank. Control structure discharges onto a reinforced concrete return channel consisting of reinforced concrete slabs and reinforced concrete sidewalls. The spillway chute terminates in a flip bucket and plunge pool. An emergency spillway with a break section is located adjacent to the service spillway, further along the right bank.
Spillway Length	90 m
Non-overspill Level	965,20 MASL (corrected datum 2017 survey)
Breach section Level	963,80 MASL (corrected datum 2017 survey)
Full Supply Level	959,58 MASL (corrected datum 2017 survey)
Freeboard = Lowest NOC – FSL	4,22 m below emergency spillway (breach section) crest and 5.62 m below main wall non-overspill crest (was >6m before vertical settlement of 438 mm @ beacon SB7)
Max Capacity (no freeboard to crest of breach section)	2,830 m^3/s
Flood Attenuation	Negligible
RDF Freeboard	1,48 meters
Will the dam fail if overtopped?	Yes, but only with sustained, significant overtopping flow depth
Erosion assessment during RDF	Erosion of the plunge pool and of the narrow exit from the plunge pool could be expected
Erosion assessment during SEF	Same as above, but emergency breach section could also be washed away
Evaluation of Spillway Capacity	The spillway capacity is adequate – see below

Calculation of spillway capacity:- See below





$P/H_0 \approx 1{,}0$

Thus $C \approx 3{,}9$

Thus $Q = 3{,}9 \times L \times H_0^{3/2}$

The spillway of 90 m length has a central lower section of L=30m and corresponding H_0=5,62m and two x 300mm higher sections with combined L=60m and corresponding H_0=5,32m.

Thus, ignoring the contribution of the breaching section to flood discharge, then:-

$Q_{max} = 3,9 \times [(30 \times 5,62^{3/2}) + (60 \times 5,32^{3/2})] = 4,430\ m^3/s$, before overtopping of the main embankment.

The freeboard from FSL to the crest of the breaching section = 4,22 m.

Thus the maximum discharge before overtopping of the breaching section is:-

$Q_{breach-noc} = 3,9 \times [(30 \times 4,22^{3/2}) + (60 \times 3,92^{3/2})] = 2,830\ m^3/s$, which is significantly greater than either the RMF = 1,330 m^3/s and the PMF = 2,563 m^3/s.

The Flood Hydrology Report included under Appendix C contains valuable information on the flood hydrology.

However the recent (2017) survey of the dam wall and appurtenant works have now rendered the flood discharge capacity calculations in that report obsolete, due to significant adjustments to freeboard heights. It is for this reason that the updated flood discharge calculations are included above.

10 INSPECTION OF THE DAM

10.1 PREVIOUS COMPULSORY INSPECTIONS

Previous 5-yearly compulsory dam safety inspections and reports were undertaken during:

1987 June

1993 August

1998 July

2003 August

2009 August

2016 September

The next 7-yearly compulsory dam safety inspection is due in 2023.

10.2 INSPECTION DATE, CONDITIONS AND TEAM

Inspection Date	13 September 2019
Water Level (MASL & m below or above FSL)	±958.58 MASL – 1,000 mm below FSL
Recent Rain? (Wet or Dry)	43 mm rain one week prior to inspection. Conditions mostly hot and dry during inspection.
Persons present at inspection	C van der Merwe (Owner's Representative) A de Beer PrEng (APP)

10.3 CREST OF EARTH WALL

Crest Width – Any change?	6 meters, no change.
Settlement – Vertical & Horizontal (m)	Ngodwana Dam wall displayed large initial vertical and horizontal settlement, but settlements have now stabilised. Maximum vertical settlement (Beacon 7) of 200 mm occurred within the first year (1983-1984) after completion, then another 200 mm settlement during the next 15 years (1984-1999) and only 37 mm during the past 18 years (1999-2019). Total current vertical settlement is 439 mm. Horizontal movement of the dam crest (downstream) under the water load, tracked the vertical settlement pattern. Total maximum horizontal movement (Beacon 6) is currently 246 mm, having stabilised to 20 mm horizontal movement over the past 15 years (2004-2019). The portion of the embankment between the riverbed and the left flank foundation contact showed significant erratic movements over the past 5 years, e.g. beacon SB15 on the crest near the foundation contact, underwent vertical settlement of 22 mm and horizontal (downstream) movement of 12 mm. During 2017 there was a vertical rebound (upwards) of 11 mm. The precision survey of 5'th-9'th August 2019 revealed that the entire extreme left portion of the embankment settled vertically by 7 mm and horizontally (downstream) by between 7-14 mm. See Appendix G – Monitoring Records, for graphs of dam wall deflections over time and plotted against water level in the dam.
Erosion? Describe	None
Cracks? Describe	None

Burrow Animal Holes? Describe	None

10.4 UPSTREAM FACE OF EARTH WALL

Slope (Vert:Hor)	1:2.5
Slope Protection	900 φ Rip-rap over natural gravel filter
Erosion? Describe	None
Cracks? Describe	None
Settlement? Describe	Some mild local subsidences in the rock rip-rap above and at FSL.

10.5 DOWNSTREAM FACE OF EARTH WALL

Slope (Vert:Hor)	1:2.0
Slope Protection	Grass
Erosion? Describe	None
Cracks? Describe	None
Settlement? Describe	None
Bulging / Sliding?	None
Wet Patches	The ”wet spot” near left flank on lower part of embankment is again quite visible. See Appendix F - Photographic Record: Plate 11
Seepage / Leaks?	Small amount of seepage in open drain below toe of embankment near left flank.
Turbidity of Seepage Water	Clear
Burrow Animal Holes? Describe	Large termite nest on lower berm near previous “wet spot” – now effectively poisoned. See Appendix F – Photographic Record, Plate 10.

10.6 VEGETATION

Trees or Shrubs on the Wall?	Trees along upstream face of embankment near crest was removed. Now only shallow-rooted Aloes and Ferns.

10.7 DRAINAGE SYSTEM

Description of Drainage System	All drains are un-zoned "natural gravel and sand" filters. Drains consist of a sloping chimney filter, connected to horisontal strip drains, culminating in internal toe drains. A separate drain envelops the outlet conduit. Most internal drains are discharged through V-notch flow gauges.
"Culvert" Drain North (l/s)	0.004 l/s (2017 aver.) 0.004 l/s (2018 aver.) 0.007 l/s (2019 aver.)
"Culvert" Drain South (l/s)	0.328 l/s (2017 aver.) 0.328 l/s (2018 aver.) 0.448 l/s (2019 aver.)
Toe Drain North (l/s)	Zero (never registered seepage)
Toe Drain South (l/s)	0.004 l/s (2017 aver.) 0.004 l/s (2018 aver.) 0.015 l/s (2019 aver.)
Total Seepage (l/s)	0.470 l/s average over 2019 There is an additional small amount of unmeasured seepage in the open drain below the dam wall on the left flank.
Turbidity	No turbidity – clear seepage water

Seepage from the dam is measured as follows:

- V-notches at "Culvert Drains North (formerly East) and South (formerly incorrectly referred to as West)" that measure seepage from the drain surrounding the culvert as well as the strip drains to the right side of the culvert.
- A V-notch that measures flow from "Toe Drains North (formerly incorrectly referred to as East) and South (formerly incorrectly referred to as West)". "Toe Drain North" never had any flow.

A V-notch in a weir in the river is supposed to measure "Seepage 1" from the Internal Drains. However, this gauge seems to include mainly the flow from the small stream originating downstream of the dam on the left (South) bank of the river.

It was concluded that the results of "Seepage 1" can therefore not be relied upon for calculating the total seepage from the dam and that the sum of the flow from the other V-notches should rather be used.

Flow in the above mentioned small stream seems to be fairly constant throughout the year. Earlier water tests (chemical and isotopes) have shown that this water does not originate from the dam basin.

10.8 CONCRETE WALLS

Cracks	None
Leakage	"Clay seepage" through some construction joints of the outlet conduit. The "clay seepage" is estimated at less than 150 ml per month or about 1,5 litre per annum. See Appendix F – Photographic Record, Plates 13-18.
Joints	Suspected torn water stops in construction joints of the outlet conduit.
Settlement	Suspected settlement of the outlet conduit responsible for torn water stops in construction joints.
Relative movement	A small amount of relative movement was observed of the spillway left side flank wall. The stability of this structure was back-analysed and found to be in order. See Appendix D for structural analysis.
Pressure relief holes	Pressure relief holes in spillway discharge chute slabs are sealed.

10.9 DOWNSTREAM TOE AND FLANKS

Wet Patches	A small stream runs parallel to and approximately 10m below the left side of the embankment toe. Origin of small stream is NOT from water in dam basin.
Seepage / Leaks	Small amount of seepage in open drain below the toe of the dam wall near the left flank.
Turbidity of Seepage Water	Clear
Trees within 5m of toe?	None

10.10 SPILLWAY, RETURN CHANNEL & FLANK WALLS

OGEE Condition	Good – some grass growing in Ogee construction joints. See Appendix F - Photographic Record - Plates 8 & 9.
Return Channel Condition	Good - joint sealants between cill slabs of spillway discharge chute were repaired. See Appendix F – Photographic Record, Plates 6-8.
Flank Walls Condition	Retaining wall on left flank displayed movements as evidenced by opening joints – probable cause is foundation settlement, but no cause for stability concern. See Appendix D – Structural Report.
Emergency Spillway Condition	Good

10.11 STILLING BASIN / PLUNGE POOL

Erosion & Scouring?	Some scouring but nothing abnormal to be concerned about.

10.12 OUTLET WORKS

Number of Outlet Pipes	2
Service Outlet 1 (Type & Dia)	500 mm dia. steel
Service Outlet 2 (Type & Dia)	500 mm dia. steel
Bottom Outlet (Type & Dia)	1,000 mm dia. steel
Intake Tower	Dry intake tower
Intake Levels (MASL) New MASL according to 2017 survey datum	Service Intake 1 – 956.133 MASL Service Intake 2 – 949.633 MASL Service Intake 3 – 943.133 MASL

Valve Control Positions (Upstream & Downstream) & Valve Type	Bottom Outlet – Flange bolted to inside of intake tower Service Outlets – Fulton Gates external to intake tower; Butterfly Valves inside intake tower; Needle control valves at downstream outlet.
Outlet System	Two 600 mm dia. steel oulet pipes located in two side-by-side 165 meter long reinforced concrete access conduits that connects the dry intake tower with the outlet works at the toe of the embankment. The two outlet conduits are actually a single structure with a vertical separation wall.
Foundations	The intake tower is founded on competent quartzite rock and the outlet conduits are founded on quartzite and alluvium. There is evidence of movement of the outlet conduits insofar as there is evidence of ingress of clay core material into the conduits through suspected torn water stops at two construction joints. See Appendix F – Photographic Record, Plates 13-18.
Operation and Maintenance	The valves are regularly operated, particularly the downstream control valves through which water supply to the water treatment works is regulated. Rust protection is undertaken on a regular basis.
Other observations	See Appendix E – Mechanical Report for more comprehensive details on the outlet works system.

10.13 PUBLIC SAFETY

Precautions are taken to safeguard members of the public by preventing uncontrolled access to the dam wall area and dam basin, through the provision of fencing, locked gates and a manned security boom.

10.14 INSTRUMENTATION AND MONITORING

Most of the dam safety monitoring instrumentation and observations are geared at detecting any possible problems associated with the performance of filters and drains and with slope stability.

Ngodwana Dam is fitted with V-notch gauges for monitoring seepage and with standpipe and pneumatic piezometers for monitoring the phreatic surface and pore pressures in the embankment. The pneumatic piezometers no longer function.

Standpipe piezometers however provide vital information on the phreatic surface in the downstream part of the embankment.

Ngodwana Dam is fitted with settlement beacons on the crest of the wall as well as at some strategic locations on the downstream face of the embankment, with which to monitor movements of the dam wall, i.e. slope stability.

10.14.1 Seepage Monitoring

Seepage from the dam is measured as follows:

- V-notches at "Culvert Drains North and South" that measure flow from parts of the chimney drain and from the drain enveloping the culvert.
- A V-notch that measures flow from "Toe Drains North and South". "Toe Drain North" never had any flow.

10.14.2 Phreatic Surface Monitoring

The phreatic surface in the embankment is monitored with standpipe piezometers. 7 new standpipe piezometers were installed during 2017 along two critical sections on the downstream slope of the embankment, along the so-called "culvert section" and the "riverbed section" respectively. The variability in the phreatic surface as measured in these standpipe piezometers, are shown in Appendix G. Generally, the phreatic surface showed insignificant movement, except in SP1 where the phreatic surface was lowered by an amount of 1 meter during 2018 and by 5 meters total (from 2017 baseline) in 2019. SP1 is located on the dam crest on the "culvert section". There is no plausible explanation for this lowering of the phreatic surface.

10.14.3 Slope Stability Monitoring

The slope stability of Ngodwana Dam is being monitored through regular and accurate surveys of the settlement beacons on the dam wall. Additional settlement monitoring beacons have been installed to monitor specific parts of the dam wall, such as the "wet spot" on the left bank toe of the embankment.

The water surface levels detected in the standpipe piezometers will be an additional indication of slope stability.

10.14.4 Instrumentation Plan

The position and type of safety monitoring equipment is shown on drawing "SAPPI 2018-01 and SAPPI 2018-02, attached under Annexure A – Maps & Drawings.

11 EVALUATION OF STABILITY OF DAM WALL

During 2017 and 2018, post-construction analyses of seepage through the dam wall as well as slope stability of the downstream zone, was undertaken, for 2 critical sections, i.e. near the culvert and at the highest point of the wall near the river section.

The results of these studies are contained in the report "*NGODWANA DAM – POST-CONSTRUCTION ANALYSES OF SEEPAGE AND SLOPE STABILITY - 20 JANUARY 2018 – Altus de Beer Consulting Engineers*".

The post-construction analyses found that the lowest factor of safety was 1.207 for the downstream zone of the embankment.

Table 11.1 – USBR Factors of Safety for Downstream Embankments

Loading Condition	Shear Strength Parameters	Pore Pressure Characteristics	Minimum Factor of Safety
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 and no field monitoring during construction and no laboratory 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
Steady-state Seepage	Effective	Steady-state seepage under active conservation pool	1,5
Operational Conditions	Effective or Undrained	Steady-state seepage under maximum reservoir level (during a probable maximum flood)	1,2
	Effective or Undrained	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
Other	Effective or Undrained	Drawdown at maximum outlet capacity (Inoperable internal drainage; unusual drawdown)	1,2
	Effective or Undrained	Construction modifications (applies only to temporary excavation slopes and the resulting overall embankment stability during construction)	1,3

This factor of safety does not meet the safety factor design criteria as set out in "*USBR Design Guideline # 13 for Embankment Dams:*

Chapter 4: Static Stability Analysis:– Table 4.2.4-1. Minimum factors of safety based on two-dimensional limit equilibrium method using Spencer's procedure" – which recommends a design factor of safety of 1.5 for "steady state seepage" conditions. See Table 11.1 above (*Table 4.2.4-1* reproduced from "*USBR Design Guideline* ..") for recommended factors of safety.

In that report the APP concluded that the margin of safety was adequate enough to pronounce the downstream slope "stable", but in light of the tight margins of safety, it was further recommended that additional permutations of soil properties be analysed.

11.1 SOIL PROPERTIES & ANALYSIS SCENARIOS

Table 11.2 below show the additional soil properties and permutations.

Table 11.2 – Soil Properties to be used in additional Seepage and Slope Stability Analyses

NGODWANA DAM SEEPAGE & SLOPE STABILITY ANALYSIS SOIL PROPERTIES AND ANALYSIS CASES - OCTOBER 2018								
SOIL PROPERTY	**LAYER 1 - FOUNDATION**	**LAYER 2 - DOWNSTREAM SHELL**			**LAYER 3 - SLOPING FILTER**	**LAYER 4 - IMPERVIOUS UPSTREAM ZONE**	**LAYER 5 - HORISONTAL STRIP FILTERS**	**Filter Case**
k	6.5×10^{-7}	6.5×10^{-7}	Maximum Seepage Coefficient		1.0×10^{-4}	1.0×10^{-9}	1.0×10^{-5}	Filters fully functional
		3.1×10^{-9}	Minimum Seepage Coefficient		6.5×10^{-7}		6.5×10^{-7}	Filters clogged
		2.6×10^{-7}	Average Seepage Coefficient					
γ	1700 kg/m3	2044 kg/m3	γ	Material properties for slope stability analysis case 1 - Average γ and c' and high Φ'	1650 kg/m3	1700 kg/m3	1650 kg/m3	
c'	50 kPa	27.8 kPa	c'		0 kPa	35 kPa	0 kPa	
Φ'	35 degrees	25.7 degrees	Φ'		35 degrees	15 degrees	35 degrees	
γ	1700 kg/m3	2044 kg/m3	γ	Material properties for slope stability analysis case 2 - Average γ and c' and low Φ'	1650 kg/m3	1700 kg/m3	1650 kg/m3	
c'	50 kPa	27.8 kPa	c'		0 kPa	35 kPa	0 kPa	
Φ'	35 degrees	18.3 degrees	Φ'		35 degrees	15 degrees	35 degrees	
γ	1700 kg/m3	2044 kg/m3	γ	Material properties for slope stability analysis case 3 - Average γ and Φ' and high c'	1650 kg/m3	1700 kg/m3	1650 kg/m3	
c'	50 kPa	39.6 kPa	c'		0 kPa	35 kPa	0 kPa	
Φ'	35 degrees	20.8 degrees	Φ'		35 degrees	15 degrees	35 degrees	
γ	1700 kg/m3	2044 kg/m3	γ	Material properties for slope stability analysis case 4 - Average γ and Φ' and low c'	1650 kg/m3	1700 kg/m3	1650 kg/m3	
c'	50 kPa	13.4 kPa	c'		0 kPa	35 kPa	0 kPa	
Φ'	35 degrees	20.8 degrees	Φ'		35 degrees	15 degrees	35 degrees	

11.2 SEEPAGE ANALYSIS SCENARIOS

There are 4 scenarios for seepage analysis, for each of the two sections, as follows:

Case 1a - Filters fully functional, maximum k for downstream zone

Case 1b - Filters fully functional, minimum k for downstream zone

Case 2a - Filters clogged, maximum k for downstream zone

Case 2b - Filters clogged, minimum k for downstream zone

11.3 SLOPE STABILITY ANALYSIS SCENARIOS

For slope stabality analysis, there are also 4 scenarios for each of the two sections.

These scenarios are set out in Table 11.2 above, as follows:

Case 1 – Average soil density & cohesion with high friction coefficient

Case 2 - Average soil density & cohesion with low friction coefficient

Case 3 – Average soil density & friction coefficient with high cohesion

Case 4 - Average soil density & friction coefficient with low cohesion

11.4 SEEPAGE & SLOPE STABILITY ANALYSIS RESULTS

The graphical output results of the seepage and slope stability analyses, using the soil parameters in Table 11.2, are included under Appendices H & I respectively.

11.5 SUMMARY OF SEEPAGE ANALYSIS RESULTS

Since both the soil properties as well as the pore pressures are affected below the phreatic surface, it is important to determine the likely position and shape of the phreatic surface.

The results of the seepage analysis will determine the range of likely phreatic surface positions and shapes that can be expected in the downstream portion of the embankment.

With reference to Appendix H, the following can be concluded:

1. For the "culvert section", the modelled phreatic surface is generally at a higher elevation than the actual measured phreatic surface. This can be ascribed to the fact that the enveloping filter around the conduit structure has a draw-down effect on the actual measured phreatic surface, that is not reflected in the modelling analysis.
2. For the above mentioned reason, the relevance of the "culvert section" analysis will be ignored for now.
3. For the "riverbed section", the four analyses cases produce significantly different results, where in some cases some or

most of the actual measured phreatic surface is at a higher elevation than the modelled phreatic surface.

4. This holds especially true for the cases where the maximum permeability is assumed for the downstream zone.

5. For the cases where the minimum permeability is assumed for the downstream zone, there is generally a close correspondence between the actual measured and modelled phreatic surfaces respectively.

6. The relative position of the modelled phreatic surface appears to be little affected by whether the filter is clogged or not, thereby indicating that the position or orientation of the sloped filter degrades its efficacy.

7. In fact, the closest correlation between the modelled and actual measured phreatic surface occurs for the case where the filter is assumed to be clogged and the downstream zone has the lowest permeability (Case 2b in Appendix H).

11.6 SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS

Table 11.3 below summarises the results of the slope stability analysis for the 4 material property permutations as set out in Section 11.3 above for the two sections "riverbed" and "culvert" respectively. See Annexure I for full analysis results.

Table 11.3 Calculated minimum factors of safety for slope stability

Scenario	Downstream Slope Stability		Upstream Slope Stability	
	Riverbed Section	Culvert Section	Riverbed Section	Culvert Section
1	1.672	1.606	2.228	2.248
2	1.326	1.289	2.228	2.248
3	1.629	1.539	2.228	2.248
4	1.175	1.193	2.228	2.248

The slope stability factors of safety for the upstream embankment are in excess of 2 in all cases.

The downstream slope stability factors are however as low as 1.175 for the "riverbed section" and 1.193 for the "culvert section".

For the "riverbed section" the critical failure slip circle starts at the upstream berm and exits at the toe of the dam wall.

For the "culvert section" the critical failure slip circle starts at the upstream side of the crest and exists at the toe of the dam wall.

11.7 CONCLUSIONS DRAWN FROM SEEPAGE & SLOPE STABILITY ANALYSES

The seepage analyses indicate that:

1. The sloping filter is not very effective in forcing the shape and position of the phreatic surface.
2. The modelled phreatic surface most closely resembles the actual measured phreatic surface for the case where the sloping filter is blocked and the downstream slope material has the lowest measured permeability coefficient.
3. At this stage, both the actual measured as well as the modelled phreatic surfaces are at acceptable levels and shapes.
4. The lowest downstream slope stability factors of 1.175 for the "riverbed section" and 1.193 for the "culvert section" indicate that the downstream slope is only just in a state of equilibrium.
5. Both of these factors of safety are far below the recommended factor of safety of 1.5 (as per USBR recommendation – Table 11.1 above), in fact there is hardly any factor of safety to speak of.
6. It is important to note that a slightly rising water table in the downstream zone could be enough to completely nullify these almost non-existent factors of safety.
7. For the reasons mentioned above, new significance should be given to the "wet" spot and to observed movements on the left flank of the dam wall.

12 EVALUATION OF DRAINAGE SYSTEM

The drainage system is evaluated to be in good working order.

All seepage water is clear (no turbidity) and seepage rates are well within acceptable limits, indicating that the clay core and grout curtain are providing an effective seal.

Seepage is monitored through four V-notches, these being:

1. Toe-drain North (formerly East)
2. Toe-drain South (formerly West)
3. Conduit left
4. Conduit right

Total current seepage is estimated at less than 0,5 l/s, significantly down from the initial seepages of 4,5 l/s.

The Owner measures seepage through the V-notches on a weekly regime. The results of measured seepage over the years are summarised in Appendix G.

13 EVALUATION OF OPERATION & MAINTENANCE

The Ngodwana Dam is well maintained and operated. The Owner has always assigned a dedicated "responsible person" to look after the dam. The Owner sets aside a budget to operate and maintain the dam in good order.

The Owner maintains in good order all relevant documentation with respect to the dam, including emergency procedures and contact details.

The Owner implements an annual inspection by the APP.

All settlement survey beacons are precision-surveyed by a professional land surveyor on an annual basis.

All standpipe piezometers are read by the owner on a monthly basis.

All seepage is measured through V-notches on a weekly basis by the Owner's personnel, as well as water levels in the dam and rainfall.

Downstream control valves are operated daily and other valves tested at least quarterly. A lubrication and stroking maintenance schedule is in place for the valves.

All trees and shrubs within 5 meters of the toe of the dam wall are removed. Grass on the downstream embankment is regularly mown.

Any damage to the spillway joint sealants are promptly repaired.

Rust-proofing paint is regularly applied to the mechanical installations.

14 RECOMMENDATIONS OF PREVIOUS EVALUATIONS BY APP

Recommendations following from previous evaluations by the APP, were implemented as follows:

1. Settlement monitoring beacons are surveyed once per year and the results are included in the annual and compulsory 7-yearly dam safety inspection reports.
2. Standpipe Piezometers:- 7 new standpipe piezometers were installed at two critical sections of the dam wall.
3. Core Sampling:- Soil samples were retrieved during the drilling of holes for the standpipe piezometers.
4. Soil Testing:- Soil samples from the drill cores were tested in a soil laboratory and the relevant soil parameters determined.
5. Seepage Analysis and Slope Stability Analysis:- Seepage & Slope Stability Analyses were conducted, using the soil properties determined under 4 above.

4. Left Bank:- The "wet spot" and embankment settlements at the left bank is inspected and monitored by the APP during his annual dam safety inspections. In addition, the SAPPI staff responsible for taking seepage readings once per week, also monitors the "wet spot" for any anomalous behaviour.
5. Spillway:- All spillway joints were cleaned of grass and other matter and the joint "bandages" replaced / repaired.
6. Mechanical Equipment:- Some mechanical equipment in the outlet works were corrosion protected or replaced in accordance with the recommendations of the mechanical engineer.
7. Warning and Evacuation Plan:- SAPPI is in the process of updating the contact details of all relevant parties in the Warning & Evacuation Plan. The plan is otherwise still relevant and in good order.

15 KEY DAM SAFETY ISSUES

15.1 KEY DAM SAFETY RISKS

The difficult foundation conditions, combined with the sloping un-zoned chimney filter, pose most of the safety risks for Ngodwana Dam. The key safety risks could be summarised as being:

(i) the precarious stability of the downstream slope of the embankment, as demonstrated by the slope stability analysis conducted as part of the 2018 dam safety report (see Sections 11.6, 11.7 and Appendix I);

(ii) further impairment of the downstream slope stability, caused by a rising downstream phreatic surface;

(iii) sliding failure through the foundations on the left bank. Of particular concern in this regard, was the development of a "wet spot" on the embankment (See Appendix X Plate 11), near the downstream toe of the embankment, on the left flank and the generally poor foundation conditions at the left flank foundation interface, as well as the recent significant measured movements of the embankment at the left bank interface;

(iv) the potential for uncontrolled piping (through either the dam wall or foundations), caused or initiated by the combined effect of the sloping, un-zoned filter, high phreatic surface, erodible material in the downstream embankment and unsatisfactory foundation conditions on the left bank foundation contact;

(v) leaching of embankment material into the outlet conduit, possibly caused by movement of the conduit that lead to tearing of water stops at construction joints;

(vi) undercutting and / or lifting of the reinforced concrete slabs forming the spillway return channel bottom, caused by water ingress through failed joint sealers; and

(vii) loss of life caused by dam break flood and inadequate advanced warning and poorly executed evacuation plan and procedures. During 1987 a dam break flood analysis was undertaken by the firm Stewart, Sviridov & Oliver (subsequently Stewart Scott Incorporated [SSI] which was recently acquired by, and trading as, Royal Haskoning DHV) that indicated a dam break flood peak of 11,000 cubic meters per second, which would, although much dissipated, travel well past Nelspruit. The dam break flood analysis showed that, in the event of a dam break failure of Ngodwana Dam, the following damage is likely to be caused:

1. Washing away of a section of the N4 highway to Nelspruit
2. Washing away of a large portion of the Ngodwana Paper Mill
3. Inundation of the Mataffin village on the banks of the Crocodile River

15.2 KEY DAM SAFETY OBSERVATIONS

The most significant **visual** observations made during the 2016 dam safety inspection is that:

i. The leakage of clay core material through the construction joint in the culvert at chainage 62-63 m downstream from the intake tower;
ii. Further (negligible) leakage of clay core material in the culvert is now taking place at a construction joint located at chainage 106 m downstream from the intake tower.
iii. Remains of termite nest near the lower berm on the left (South) flank downstream embankment near the "wet spot".

The most significant observations that can be made from the dam safety **instrumentation monitoring**, is that:

i. The portion of the embankment between the riverbed and the left flank foundation contact showed significant erratic movements over the past 5 years, e.g. beacon SB15 on the crest near the foundation contact, underwent vertical settlement of 22 mm and horizontal (downstream) movement of 12 mm.

However, during 2017 there was a vertical rebound (upwards) of 11 mm. The precision survey of 5'th-9'th August 2019 revealed that the entire extreme left portion of the embankment settled vertically by 7 mm and horizontally (downstream) by between 7-14 mm.

ii. Ngodwana Dam wall displayed large initial vertical and horizontal settlement, but settlements have now stabilised. Maximum vertical settlement (Beacon 7) of 200 mm occurred within the first year (1983-1984) after completion, then another 200 mm settlement during the next 15 years (1984-1999) and only 38 mm during the past 20 years (1999-2019). Total current vertical settlement is 439 mm.

iii. Horizontal movement of the dam crest (downstream) under the water load, followed the vertical settlement pattern. Total maximum horizontal movement (Beacon 6) is currently 246 mm, having stabilised to only 20 mm horizontal movement over the past 15 years (2004-2019).

iv. Seepage declined to less than 0,5 l/s. Measured seepage has steadily decreased from 3 l/s in 1997/1998 to less than 1 l/s in 2008/2009. Over the period 2008/2009-2012/2013 the measured seepage has again increased from under 1 l/s to about 1.5 l/s in 2015, but these "increases" could have been due to the measurement of "non-seepage" runoff at monitoring weir "Seepage 1 – internal drains", which is no longer used in the calculation of total seepage.

15.3 KEY DAM SAFETY CONCLUSIONS

The key conclusions to be drawn from the above observations and the analyses conducted as part of this report, are as follows:

i. From both the dam safety instrumentation monitoring and visual observations by dam safety specialists, it can generally be concluded that, allowance being made for the large early deflections and settlements of the dam wall, Ngodwana Dam has behaved well within expected parameters over the past 36 years.

ii. However, the slope stability calculations that were performed as part and parcel of the 2018 safety report (Sections 11.6 and 11.7 plus Appendix I), clearly indicate that the downstream slope is only just in a state of equilibrium but that it is far out of range of acceptable safety factors (See Table 11.1) that are prescribed for a dam of this safety risk classification.

iii. the relatively sudden and significant movement of the embankment on the left bank foundation interface could have been caused by a number of factors, alone or in combination, i.e.:- (i) settlement of the relatively poor foundation conditions underlying the embankment in that area; (ii) washing out of foundation material; (iii) losing of the "bridging effect" of the embankment through settlement over the years, causing "slippage" of the left edge of the embankment; etc.

iv. the leakage of embankment material into the outlet conduit is likely due to torn water stops, caused by movement of the conduit on its foundations;

v. the large initial settlements of the dam wall (near its highest point) is likely due to the consolidation of the thick unconsolidated foundation material that underlies the dam wall foundation;

vi. the unconsolidated foundation materials are now almost fully consolidated from the dam wall loads;

vii. the steadily reducing seepage might be ascribed to the reducing permeability of the consolidating foundation materials underlying the dam wall; and

viii. Seepage water is clear (no turbidity) and is very low for an earth embankment dam of this height and size.

16 SUMMARY AND RECOMMENDATIONS

The principal safety risk for Ngodwana Dam is the precarious stability conditions of the downstream slope, as was determined as part and parcel of the 2018 dam safety report. The most critical condition arises for the permutation when the lowest measured cohesion values are used in combination with the average values for the other soil properties, i.e. density and internal friction angle. These factors of safety for the "riverbed" and "culvert" sections are only 1,175 and 1,193 respectively.

Even the more benign permutations of material properties deliver factors of safety that are lower or just above the prescribed factor of safety of 1,5.

Furthermore, the precarious slope stability can be further impaired by a rising phreatic surface. It was already demonstrated in the seepage analysis (Appendix H) that the sloping, un-zoned filter is ineffective at controlling the phreatic surface and that it is mainly the permeability of the downstream zone materials that affect the phreatic surface.

For the above mentioned reasons, it is recommended that the Owner promptly undertake measures to improve the stability of the downstream slope of the embankment.

These measures should commence with a study to determine the most feasible and cost-effective way to achieve the desired slope stability. The desired slope stability can be achieved by adding a stabilizing berm at the toe of the dam with a filter between the berm and the existing embankment. The study would be required to optimize the configuration of the berm.

With regards to other dam safety issues, it should be mentioned that the Owner generally maintains Ngodwana Dam in a good condition. The responsible person, Mr Carel van der Merwe, diligently kept records and ensured the proper maintenance of the dam.

However, Ngodwana Dam was built on complex foundations and that, together with other factors, increase the inherent risks that the dam poses to life and property. In particular, the dam safety risks associated with the difficult foundation conditions and the filter design should be closely monitored.

This would be best achieved by having:

i. monthly reading of the water table in the standpipe piezometers;

ii. weekly gauging of seepage flow through V-notches;

iii. annual monitoring of dam wall deflections through precision survey of deflection beacons;

iv. the embankment material leakage as well as the water seepage through the construction joints of the outlet conduit should be carefully monitored;

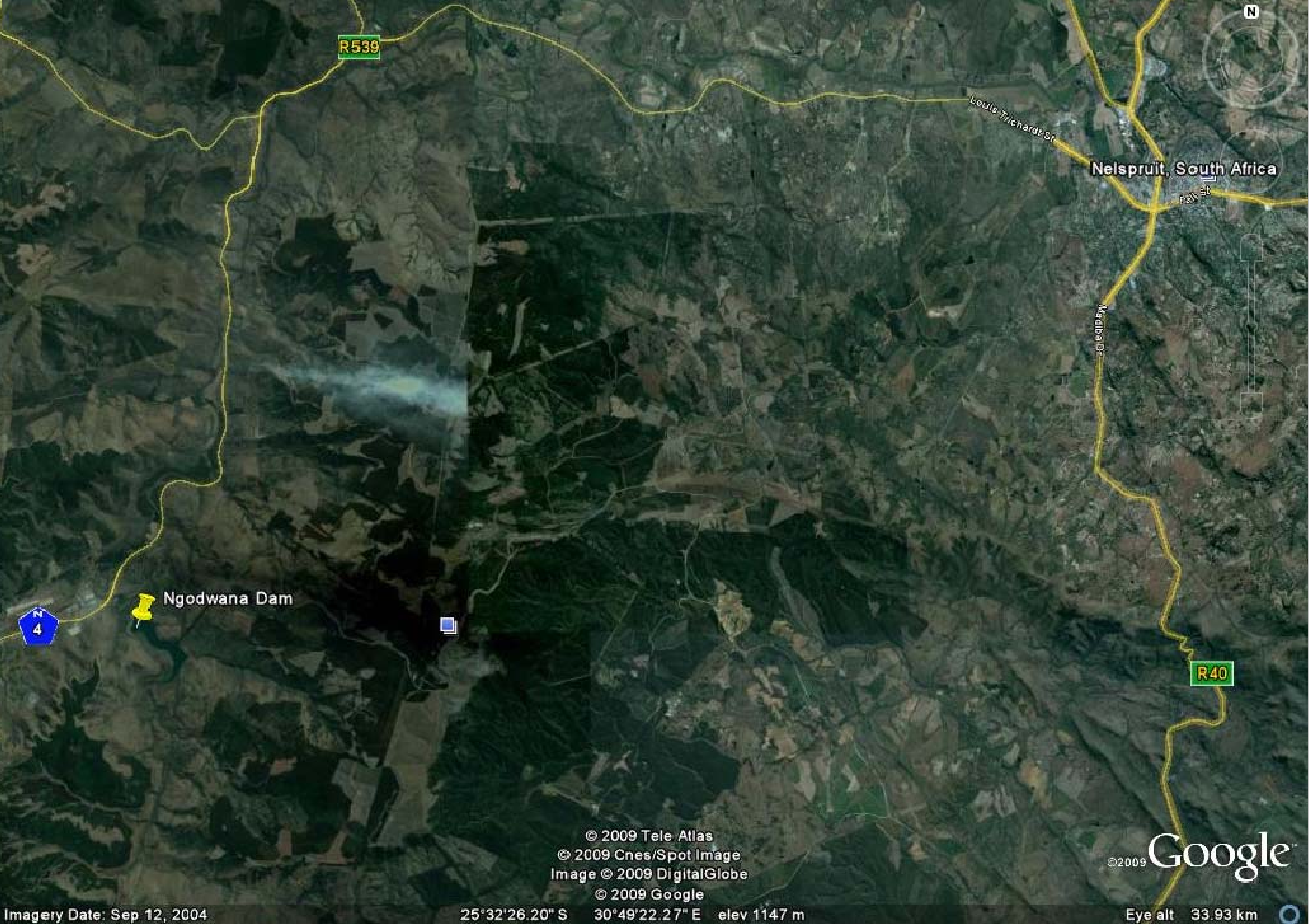
v. termite nests must be poisoned and monitored for any further activity;

vi. the Warning & Evacuation Plan should contain all the necessary warning and evacuation procedures as well as updated contact numbers of all affected people and organisations. The Plan is to be kept in an accessible location, which should be known to all key safety operational staff at SAPPI;

vii. the SAPPI safety operational staff should, at least once per year, check whether they are able to contact all relevant affected individuals in a timely manner to take appropriate action in the event of a "dam break flood" event; and

viii. the early warning and evacuation plan should be maintained and practised, in the event that a dam failure risk is detected or experienced.

APPENDIX A: MAPS AND DRAWINGS


R539
Louis Trichardt St
Nelspruit, South Africa
N
Ngodwana Dam
N 4
R40
© 2009 Tele Atlas
© 2009 Cnes/Spot Image
Image © 2009 DigitalGlobe
© 2009 Google
©2009 Google
Imagery Date: Sep 12, 2004
25°32'26.20" S 30°49'22.27" E elev 1147 m
Eye alt 33.93 km


SAPPI Ngodwana Paper Mill
Site Inspection Meeting Place
Ngodwana Dam Access Road from N4
Ngodwana Dam Embankment
© 2016 Google
© 2016 AfriGIS (Pty) Ltd.
Image © 2016 CNES / Astrium
463 m
2003
Imagery Date: 3/23/2014 25°34'51.65" S 30°40'06.07" E elev 952 m eye alt 2.92 km
Google earth

DAM WALL AND SPILLWAY PLAN WITH AERIAL PHOTOGRAPHIC FEATURES

SCALE 1 : 1000

AMENDMENTS

NR.	DATE	APPROVED	DESCRIPTION	PAR.
A	SEPT 2017		FOR APPROVAL	TEL

DESIGNED J STANDER	DRAWN J STANDER
DESIGN CHECKED BY A. DE BEER SIGNATURE:........... DATE:.........	INFRASTRUCTURE TECHNICAL INFORMATION MANAGEMENT NAME: SIGNATURE:........... DATE:...........

PROJECT STATUS

CONCEPT DRAWING	TENDER DRAWING	APPROVED FOR CONSTRUCTION	AS BUILT DRAWING
○	○	○	○

PROJECT ENGINEER (CONSULTANT)

INITIALS AND SURNAME SIGNATURE AND Pr. No. DATE

INSPECTOR OF WORKS (EKURHULENI):

INITIALS AND SURNAME SIGNATURE AND Pr. No. DATE

CLIENT

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Reg No 1951/003180/06
Ngodwana

APPROVED PROFESSIONAL PERSON

A de Beer Pr Eng
Reg No. 820410

CONSULTING ENGINEER

ALTUS DE BEER
CONSULTING ENGINEERS

LOCATION OF PROJECT:

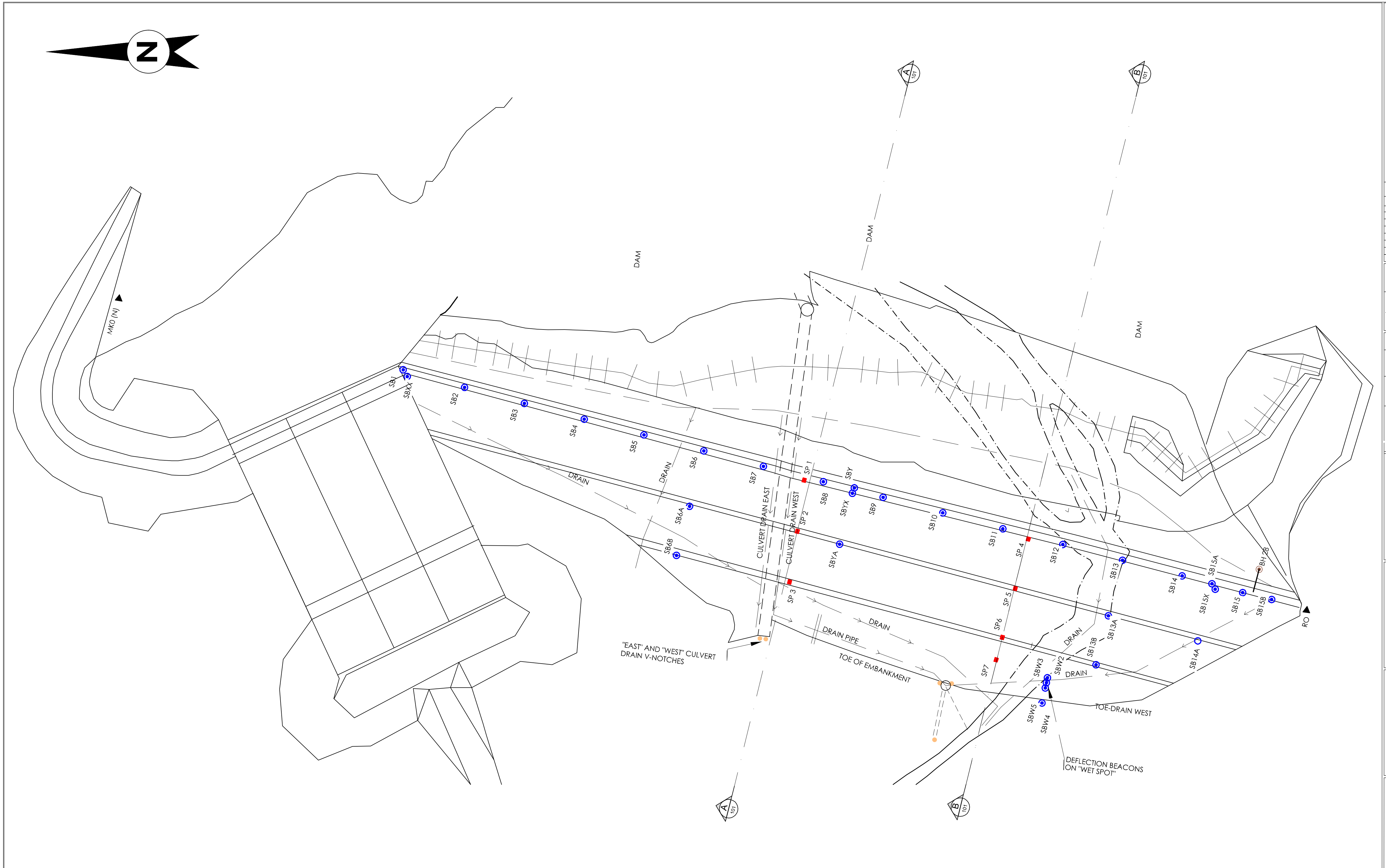
NGODWANA DAM

DESCRIPTION OF PROJECT

DAM WALL AND SPILLWAY PLAN

WITH AERIAL PHOTOGRAPYHIC FEATURES

CONTRACT No. : 1734	PROJECT No. : 1734	
DATE : SEPT 2018	SCALE : AS SHOWN	ORIGINAL PAPER SIZE: A1
DRAWING NO. SAPPI 2018 - 01	SHEET NO: 1 OF 4	A REVISION



SETTLEMENT MONITORING BEACON SURVEY DATA - August 2018						
Station	Horisontal (Y) Deflection (mm)	ORIGINAL BEACON MASL	CURRENT BEACON MASL	Vertical Deflection (mm)	ORIGINAL GROUND MASL	CURRENT GROUND MASL
SP2	-28	965,933	965,869	-64	965,635	965,571
SP3	-88	965,973	965,741	-232	965,665	965,433
SP4	-186	965,915	965,628	-287	965,603	965,316
SP5	-216	965,944	965,612	-332	965,681	965,349
SP6	-245	965,948	965,546	-402	965,634	965,232
SP7	-177	965,900	965,462	-438	965,636	965,198
SP8	-129	965,863	965,545	-318	965,553	965,235
SP9	-90	965,861	965,579	-282	965,560	965,278
SP10	-75	965,861	965,651	-210	965,582	965,372
SP11	-71	965,880	965,674	-206	965,589	965,383
SP12	-57	965,865	965,665	-200	965,578	965,378
SP13	-66	965,747	965,527	-220	965,434	965,214
SP14	-75	965,835	965,582	-253	965,568	965,315
SP15A #	-34	965,675	965,636	-39	965,425	965,386
SP15B #	-10	965,651	965,594	-57	965,400	965,343

DAM WALL AND SPILLWAY PLAN

SCALE 1 : 1000

AMENDMENTS

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DESIGN CHECKED BY: A. DE BEER
INFRASTRUCTURE TECHNICAL INFORMATION MANAGEMENT
NAME:
SIGNATURE:........... DATE:.......... SIGNATURE:........... DATE:............

PROJECT STATUS

PROJECT ENGINEER (CONSULTANT) DATE
INITIALS AND SURNAME SIGNATURE AND Pr. No.

INSPECTOR OF WORKS (EKURHULENI):
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Ngodwana

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Reg No. 820410

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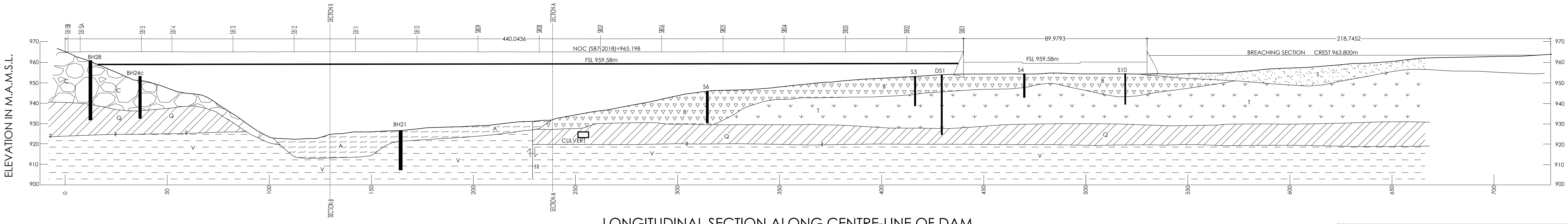
ALTUS DE BEER
CONSULTING ENGINEERS

LOCATION OF PROJECT:
NGODWANA DAM

DESCRIPTION OF PROJECT
DAM WALL AND SPILLWAY PLAN

CONTRACT No. : 1734
PROJECT No. : 1734
DATE : SEPT 2018
SCALE : AS SHOWN
ORIGINAL PAPER SIZE: A1
DRAWING NO. SAPPI 2018 - 02
SHEET NO: 2 OF 4
REVISION A



LONGITUDINAL SECTION ALONG CENTRE-LINE OF DAM
Scale 1:1000

BH1 — DIAMOND DRILL HOLE

BH1 — PERCUSSION DRILL HOLE

S6 — PERCUSSION DRILL HOLE (NO LOG AVAILABLE)

FAULT (UNCERTAIN) f?

GEOLOGICAL CONTACT (INFERRED)

EXTENT OF GROUT CURTAIN

S — COLUVIAL SOIL

C — COLLUVIAL BOULDERS AND BRECCIA

A — ALLUVIUM

(S, C, A: RECENT UNCONSOLIDATED DEPOSITS)

B — QUARTZITE REMNANT BLOCKS — BLACK REEF FORMATION

T — VOLCANIC TUFF, TUFFACEOUS SHALE, AGGLOMERATE

Q — QUARTZITE

V — LAVA, HORNFELS, QUARTZITE

(T, Q, V: GODWAN FORMATION)

AMENDMENTS

NR	DATE	APPROVED	DESCRIPTION	PAR.
A	SEPT 2018		FOR APPROVAL	TEL

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DRAWN: J STANDER
DESIGN CHECKED BY: A DE BEER
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NAME:
SIGNATURE:............ DATE:.......... SIGNATURE:............DATE:............

PROJECT STATUS
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PROJECT ENGINEER (CONSULTANT)
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Reg No. 820410

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ALTUS DE BEER
CONSULTING ENGINEERS

LOCATION OF PROJECT:
NGODWANA DAM

DESCRIPTION OF PROJECT
LONG SECTION

CONTRACT No. : 1734	PROJECT No. : 1734	
DATE : SEPT 2018	SCALE : AS SHOWN	ORIGINAL PAPER SIZE: A1
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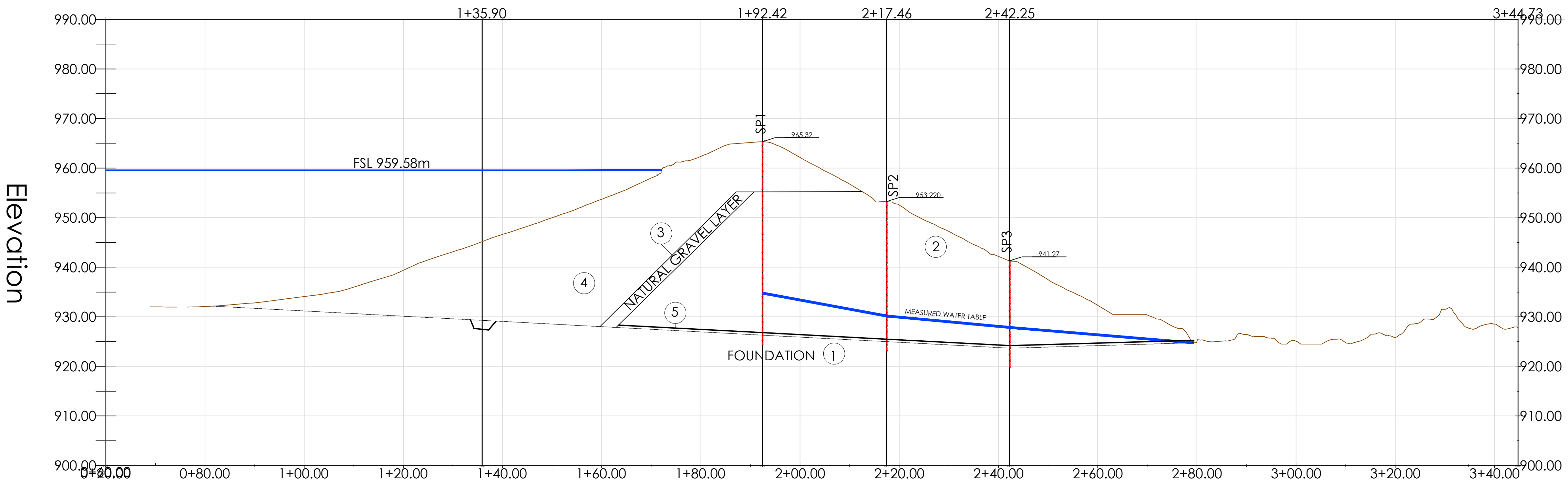
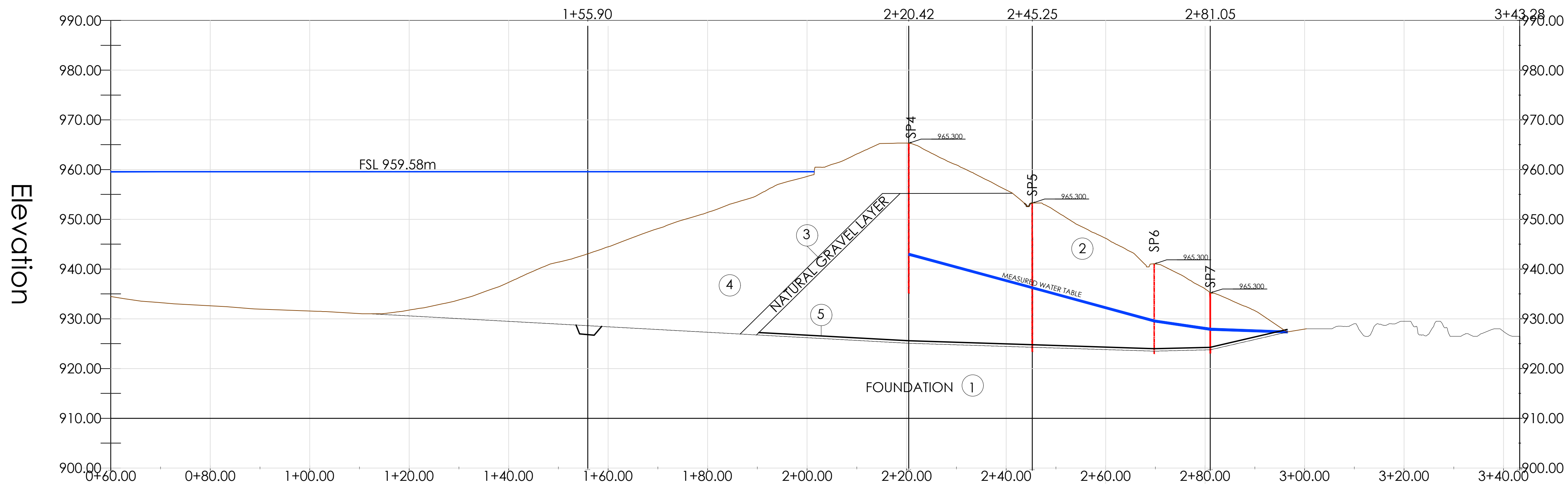


SECTION A-A "CULVERT"
SCALE 1:500



SECTION B-B 'RIVERBED"
Scale 1:500

LEGEND:

1 FOUNDATION
2 DOWNSTREAM SHELL
3 NATURAL GRAVEL
4 IMPERVIOUS ZONE
5 GRAVEL STRIP

AMENDMENTS

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DRAWN: J STANDER
DESIGN CHECKED BY: A DE BEER
INFRASTRUCTURE TECHNICAL INFORMATION MANAGEMENT
SIGNATURE:........... DATE:........ SIGNATURE:............DATE:...........

PROJECT STATUS

○ CONCEPT DRAWING ○ TENDER DRAWING ○ APPROVED FOR CONSTRUCTION ○ AS BUILT DRAWING

PROJECT ENGINEER (CONSULTANT)
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A de Beer Pr Eng
Reg No. 820410

CONSULTING ENGINEER
ALTUS DE BEER
CONSULTING ENGINEERS

LOCATION OF PROJECT:
NGODWANA DAM

DESCRIPTION OF PROJECT
CROSS SECTIONS

CONTRACT No.: 1734
PROJECT No.: 1734
DATE: SEPT 2018
SCALE: AS SHOWN
ORIGINAL PAPER SIZE: A1
DRAWING NO. SAPPI 2018 - 04
SHEET NO: 4 OF 4
REVISION A

APPENDIX B: GEOLOGICAL REPORT

NGODWANA DAM

SAFETY INSPECTION: AUGUST 2009
ENGINEERING GEOLOGY REPORT

TEAM MEMBER / AUTHOR: A VAN SCHALKWYK

1. INTRODUCTION

The Engineering Geology Report deals with the site geology and its influence on the performance of the embankment and spillway of the Ngodwana Dam.

Reference is made to previous reports on engineering geological safety inspections by A van Schalkwyk dated 7 August 1998 and 23 September 2003. The first report contains a review of all available geological data and a detailed assessment of geological conditions.

2. SUMMARY OF GEOLOGICAL CONDITIONS

2.1 General

The site geology is extremely complex due to the occurrence of a variety of sedimentary, volcanic and metamorphic rocks, unconformities in the sedimentary succession, faulting, variable weathering, thick alluvium in the river section, colluvium on the left flank and large dislodged quartzite blocks on the right flank. The distribution of the material types along the dam centre line and the cut-off trench is illustrated on the attached sections (see **Annexure H Drawings – Dwg No Ngodwana Dam (3) – Longitudinal Sections**)

2.2 Left flank

The upper part of the left flank is underlain by a 5m – 20m thick cover of partly recemented (calcified) talus blocks in a matrix of soil, resting upon a layer of completely weathered very weak tuff and agglomerate. Although the embankment in founded on these low strength and potentially permeable materials, the cut-off wall was re-aligned in an upstream direction and is largely founded on a downstream dipping layer of strong Godwan Formation quartzites. Only the upper part (mostly above FSL) of the cut-off is founded on weak tuff and partly cemented colluvium. This part of the flank had been covered by an impervious blanket, and based on seepage records, it can be concluded that a reasonably watertight cut-off had been achieved.

2.3 River section

The river section is underlain by a 5m – 10m thick layer of river alluvium that is underlain by medium strong tuff, lava, hornfels and quartzite of the Godwan Formation. These rocks are closely jointed in some areas. The cut-off is founded on bedrock.

2.4 Right Flank

Along the major part of the right flank, the embankment and spillway structures are founded on a layer of large dislocated (slumped) blocks of Black Reef Formation quartzite, resting upon a thin layer of weak tuff, tuffaceous shale and agglomerate of the Godwan Formation that is underlain by strong quartzite of the same formation. The cut-off is founded partly on the strong quartzite and partly on the weak rocks overlying it. The engineering properties (deformability and permeability) of these rocks are unknown.

2.5 Spillway Structure

The overspill and chute are founded on dislodged quartzite blocks and colluvium, while the stilling basin is located in weathered, weak tuffaceous shale.

2.6 Breaching Section

The breaching section is partly founded on the dislocated quartzite blocks and partly on colluvial soil comprising gravely sand and clay.

3. PRESENT INSPECTION AND EVALUATION

The site visit of 27 August 2009 included an inspection of the left flank, the dam crest, the upstream and downstream slopes (including the “wet area” on the downstream slope), the seepage measuring points, the outlet culvert, the spillway chute, the return channel and the breaching section. Available records of seepage and deflection measurements were reviewed. Inspection of the structure was greatly facilitated by the high standard of maintenance (see **Plate G1**).

Plate G1. View of dam wall from left abutment.

3.1 Displacement Monitoring

Since 1983, 13 Monitoring Beacons (MK2 – MK14) and 2 Floating Reference Beacons (MK1 and MK15) on the dam crest were monitored for vertical and horizontal displacements (see **Annexure H Drawings – Dwg No Ngodwana Dam (1) – Instrumentation Plan**). The maximum movements occurred in the vicinity of the culvert (MK6 - MK 8) and have been a source of concern, both on account of the magnitude of the movements and the large horizontal (downstream) components. The downstream components could be explained by the orientation of the resultant load of a full dam, but the total recorded movements of about 400mm vertical and 200mm downstream at Beacons MK 6 and MK7, are more than what could be expected of a well-compacted embankment of this height. It is therefore considered likely that some of the movements have taken place in the foundation material which, in the area of the culvert, comprises 8m – 10m of alluvium (upstream part of the footprint) and similar thicknesses of dislodged quartzite blocks with unconsolidated soil between them (near the centre-line and downstream of it). Both the alluvium and the quartzite blocks are prone to some degree of consolidation settlement.

Plots of the maximum movements (e.g. MK6 and MK7 on **Annexure I – Monitoring Records and Results – Beacon 7 Vertical Movement vs Time vs Water Level** and **Beacon 6 Horizontal Movement vs Time vs Water Level**), show decreases with time that are consistent with consolidation settlement in poorly compacted soil. Over the last five years, movements have generally been less than 1mm per year, and it can be concluded that the dam wall and foundations are now almost fully consolidated.

During the years 2000 and 2001, a number of additional Monitoring Beacons (6A, 6B, 13A, 13B, 15A and 15B) as well as a few additional Floating Reference Beacons were installed on the crest and downstream berms, while in 2008 a row of Monitoring Beacons (W1 – W6) were installed across the "wet area" on the downstream slope. The new beacons on the crest and berms also show vertical and downstream movements, but the accumulated magnitudes are less than 20mm. After about one year of installation, movements of between 1mm and 2mm were recorded at Beacons W1 – W6.

In the first safety inspection report, it was noted that the Fixed Reference Beacon B0 above crest level on the left abutment was located on a colluvial boulder that might be subject to movement (see **Plate G2**). Sappi reported that an additional beacon was installed higher up along the flank, but from surface indications, it appears that the new beacon may also be situated on colluvium.

Plate G2. Beacon on left abutment.

Six standpipe piezometers located above crest level on the left flank have also been used as beacons to monitor possible movement of the steep slope that was formed during excavation for the cut-off trench. Maximum settlement since 1998 is about 30mm, but movements have decreased, and since 2003, the maximum recorded movement is 6mm amounting to less than 1mm per year.

3.2 Rip-rap

Rip-rap on the upstream slope is generally in a good condition and the rock blocks show no signs of weathering. In one area near the dam crest, there is a shallow trench in the rip-rap where blocks had been removed (see **Plate G3**). This area must be restored.

Plate G3. Trench on upstream face.

3.3 "Wet" Area on Downstream Slope

The "wet" area that was reported on in previous reports is still visible (see **Plate G4**), but is has not increased in size and appears to be much drier than before. This is reassuring since the dam was at FSL. No significant movements have been recorded by the Monitoring Beacons installed in 2009 (see **Plate G5**).

Plate G4. "Wet" area on downstream face also showing left flank (west) surface drain.

Plate G5. Beacons WD2 – WD5 across "wet area".

3.4 Seepage from Dam

Seepage from the dam is measured as follows:

- V-notches at Culvert Drains East and West that measure flow from parts of the chimney drain and from along the culvert.
- A V-notch that measures flow from Toe Drains East and West (see **Plate G6**). Toe Drain East never had any flow.
- A V-notch in a weir in the river that measures Seepage 1 from the Internal Drains (see **Plate G7**).

Plate G6. V-notch for Toe Drain East and West.

Plate G7. Weir for measuring “Seepage 1”.

The total seepage from the dam is calculated as the flow from all the above measuring points. Records since 1997 show that the total seepage is sometimes (but not always) affected by the water level in the dam and sometimes (but not always) by the rainfall. Total seepage has generally decreased from a maximum of about 4,5l/sec in 1997 to a maximum of about 1,6l/sec in 2009. Most of the seepage originates from the internal drains as measured as Seepage 1 at the weir, while the remainder comes from the Toe Drain West. In October 2001, the record for Seepage 1 is exceptionally high (about 6l/sec) and this can only be attributed to a mistake.

A possible problem with the record from Seepage 1 at the weir is that some of the flow may bypass the V-notch by seepage below or around the weir. The decrease in measured flow could be the result of an increase in sub-surface flow due to washing out of fines below the weir.

Flow in a small stream originating from downstream of the dam on the left (west) flank is not being measured, but seems to be fairly constant throughout the year (see **Plate G8**). Earlier water tests have shown that this water does not originate from the dam.

Plate G8. Seepage in stream on left flank

3.5 Piezometers

The pneumatic piezometers that had originally been installed along two rows between the core and the dam crest are no longer working, and in the previous safety inspection report, it was recommended that two rows of stand-pipe piezometers must be installed to replace the defunct ones. This had not been done, and the only available water levels readings are from 6 standpipe piezometers located on the left flank upstream of the centre line, and 2 piezometers located on the middle downstream berm (see **Annexure H Drawings – Dwg No Ngodwana Dam (1) – Instrumentation Plan**). Six of these points are also being used as settlement beacons (see paragraph 3.1 above).

The collar heights of the piezometers on the left flank vary between 965,3 masl and 979, 6 masl (5,3m – 19,6m above FSL) and the water levels vary between 6m and 14m below FSL. Between 1998 and 2009, the water levels fluctuated generally less than 1m, except for the readings taken in November 2003, when the water levels were generally 1m to 2m deeper than in the years before and after. Groundwater levels decrease in a downstream direction but do not drop much from east to west. Their levels with respect to the FSL indicate an effective cut-off below the dam wall, but a high permeability of the left flank.

3.6 Spillway Chute and Plunge Pool

The spillway chute appears to be in good condition with no major signs of displacement or cracking (see **Plate G9**). During February 2009, one of the largest recorded floods of about 500m^3/s passed over the spillway. However, based on a comparison of photographs taken in 2003 and during the present visit, the plunge pool and the narrow exit channel in weak rock had not been eroded (see **Plate G10**). However, due to the limited capacity of the discharge channel, major flooding of the area downstream of the dam will occur when larger floods are passed.

Plate G9. View of spillway. Note that curved appearance is due to "photo stitching".

Plate G10. Plunge pool. Note erosion between rock blocks and side of channel.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 The dam has been maintained very well, and only a small part of the rip-rap needs to be repaired.

4.2 Vertical and horizontal (downstream) movements of Monitoring Beacons on the dam are larger than what could be expected of a well-compacted embankment. It is considered likely that part of the movements took place within the foundation materials that comprise of alluvium and dislodged quartzite blocks. Vertical and horizontal movements are decreasing with time and have amounted to less than 1mm per year over the last five years. It can be concluded that the dam wall and foundations have now been almost fully consolidated and that there are no cause for concern regarding the stability of the embankment

4.3 The “wet” area that was reported on in previous reports is still visible, but is has not increased in size and appears to be much drier than before. This is reassuring since the dam was at FSL. No significant movements have been recorded by the Monitoring Beacons installed across the “wet” area in 2009.

4.4 Total seepage has generally decreased from a maximum of about 4,5l/sec in 1997 to a maximum of about 1,6l/sec in 2009. Most of the seepage originates from the internal drains as measured at Seepage 1 at the weir, while the remainder comes from the Toe Drain West. A possible problem with the record from Seepage 1 is that some of the flow may bypass the V-notch by seepage below or around the weir. It is recommended that test pits be dug to investigate the founding conditions of the weir and to take steps to ensure that all seepage can be measured.

4.5 The only available water levels readings are from 6 standpipe piezometers located on the left flank upstream of the centre line, and 2 piezometers located on the middle downstream berm. Water levels are well below the FSL and fluctuate very little with time. Groundwater levels decrease in a downstream direction but do not drop much from east to west. Their levels with respect to the FSL indicate an effective cut-off below the dam wall, but a high permeability of the left flank. It is recommended that new standpipe piezometers be installed between the core and the middle berm along two lines across the embankment as shown on **Annexure H Drawings – Dwg No Ngodwana Dam (1) – Instrumentation Plan**.

4.6 No significant erosion had taken place in the stilling basin and along the narrow discharge channel. However, it appears that in the case of a major flood, the limited capacity of the discharge channel could result in flooding of the area downstream of the dam.

APPENDIX C: FLOOD HYDROLOGY REPORT

NGODWANE DAM

THIRD DAM SAFETY INSPECTION : 27 August 2009

Report for the Department of Water Affairs & Forestry

REVIEW OF FLOOD HYDROLOGY AND HAZARD POTENTIAL

By Dr W V Pitman

1. INTRODUCTION

This is the second review of the flood hydrology for Ngodwane Dam, which was first undertaken by the author for the dam safety inspection of June 1998. As it is more than 10 years since the first inspection, it is considered prudent at this stage to re-assess the flood hydrology in the light of new methodologies and calculation procedures that have come into usage since that time. Apart from new methodology, some of the original methods that relied on graphical interpretation have been computerized to ensure consistent results.

2. FLOOD HYDROLOGY

2.1 Methods for design flood determination

A total of five methods have been used to derive flood peaks for various return periods in addition to the RMF and PMF. A brief description of each method is given in Table 1.

Table 1 Description of methods for design flood determination

Method	Brief description
HRU 1/72 Regional formula	Appendix B of HRU Report No. 1/72 contains a regional map and a co-axial diagram for a quick estimation of floods for return periods ranging from 5 to 200 years. The estimates can not be considered as accurate since they rely only on catchment area and location.
HRU 1/71 Flood formula	This method takes into account veld cover, mean rainfall, catchment shape and catchment slope. It is meant to give similar – but quicker – results to that of the HRU 1/72 unit hydrograph method.
Rational method	This method is designed primarily for small catchments of the order of 10 km^2 or less. Nevertheless many hydrologists apply this method to relatively large catchments.
DWAF TR137 method	The report contains factors which, when applied to the RMF, yields quick estimates of the 50-, 100- and 200-year flood peaks.
Standard design flood (SDF)	This method is a relatively simple, but robust, method developed by Alexander (SAICE Journal, 44(1), 2002). Model parameters are provided for 29 major drainage basins/regions.

2.2 Catchment characteristics related to flood hydrology

The methods outlined in Table 1 rely on some of the following physical characteristics of Ngodwane Dam catchment, as listed in Table 2 below.

Table 2 Catchment characteristics

Physical characteristic	Value	Units
Catchment area	229	km^2
Average channel slope	0.0173	-
Length of longest watercourse	26	km
Length to catchment centre	13	km
Mean annual precipitation	1068	mm
HRU veld zone	3	-
HRU extreme rainfall zone	1	-
Francou-Rodier "k"	5	-
Francou-Rodier "k+Δ"	5.2	-
Francou-Rodier "k-Δ"	4.6	-
HRU flood zone (Appendix B)	4	-
Percentage of catchment under afforestation	28	%
Percentage of catchment with dolomitic exposure	56	%

2.3 Comparison of results obtained with different methods

Table 3 lists the results obtained with the different methods. It should be noted that the SDF method was not used in the original dam safety assessment of flood hydrology but was used in the previous (2003) assessment.

Table 3 Summary of results (flood peaks in m^3/s)

Flood event	HRU Region	HRU Formula	Rational method	TR137	SDF	Mean estimate
10-year	173	188	338	*312*	234	249
20-year	243	259	406	*427*	354	338
50-year	351	356	628	609	536	496
100-year	446	453	893	771	690	650
200-year	551	*564*	*1230*	948	*868*	832
RMF-Δ				901		901
RMF				1513		1513
RMF+Δ				1968		1968
PMF		2695	3130			2913

Note – extrapolated peaks shown in *italics*.

One does not necessarily accept the simple mean of the different methods, as it usually necessary to give greater weight to the methods deemed to be the more reliable. Before accepting the final values, it is also appropriate in this case to compare these results with the values obtained in the first dam safety report and any revisions emanating from the second review. This comparison is set out in Table 4.

Table 4 Comparison of results with previous flood estimates

Flood event	Means from Table 3	1998 Dam Safety Report	2003 Dam Safety Report
10-year	249	Not done	Not done
20-year	338	380	No change
50-year	496	560	No change
100-year	650	720	No change
200-year	832	910	No change
RMF-Δ	901	Not done	Not done
RMF	1513	1510	No change
RMF+Δ	1968	1970	No change
PMF	2913	2700	No change

Before any changes to the flood peaks can be considered, it is necessary to re-visit the adjustments that were considered necessary to account for the large areas of afforestation and dolomite in the catchment of Ngodwane Dam, as explained below.

2.4 Allowance for Afforestation and Dolomitic Exposure

In report TR137 it is stated (on page 17) that " ………K may be reduced if (i) more than half of the area is very permeable, dolomitic or covered by plantations……….." It also states that "The reduced K may not be lower, however, than the K of the next lower number region….."

The Ngodwane Dam catchment is predominantly dolomitic (56%) and, in addition, has plantations of exotic forest covering 28% of the catchment. There is thus some justification in reducing K, provided it remains above the next lower number region – in this case 4.6. However, as the catchment is steep it would be prudent not to reduce K substantially. Accordingly, a K of 4.9 was adopted, giving a revised RMF of 1330(m^3/s) or 88% of the unadjusted estimate. This factor was used to adjust all flood peaks, but has not been applied to the mean estimates of the present analysis (for floods in the range 20- to 200-year) as listed in the second column of Table 4. The flood peaks are compared in Table 5, where it can be seen that the two sets of figures are quite close.

Table 5 Comparison of adjusted flood peaks

Return period (years)	Flood peak (m^3/s)	
	Current analysis	Previous analysis (adjusted)
20	338	330
50	496	490
100	650	630
200	832	800
RMF	1330*	1330
PMF	2563*	2400

NB * These values also adjusted as for previous analysis

2.5 Standard Design Discharge (SDD) and Safety Evaluation Discharge (SED)

As Ngodwane is a Category III dam the SDD must be the 1 in 200 year event, hence the value of 800 m^3/s was adopted for the 1998 analysis. This value was retained for the subsequent 2003 analysis. The current analysis yielded an average 200-year (unadjusted) peak of 832 m^3/s but, as can be seen from inspection of Table 3, there is a wide range of estimates derived from the various methods. Accordingly, it is recommended that the original value of 800 m^3/s be retained for the SDD.

Ngodwane Dam has a low hazard rating, hence the SED can be set equal to the (adjusted) RMF, which is 1330 m^3/s for both the original and current analysis.

The full supply area of Ngodwane Dam is 87ha, which is equivalent to 0.38% of the catchment area. This suggests that the attenuation of incoming floods is likely to be minimal and can be ignored.

3. Appraisal of Spillway

The discharge formula for the spillway remains acceptable. The spillway rating shows that a discharge of 1766 m^3/s can be passed before the breaching section is overtopped. This discharge is well in excess of the SEF of 1330 m^3/s and is about 70% of the PMF. The dam wall itself is well above the SEF level and RDD level plus freeboard, as shown below.

RDD (800 m^3/s): elevation = 964.07m (incl. freeboard allowance of 1.35m)
SED (1330 m^3/s): elevation = 963.76m

Elevation of breaching section: 964.50m
Elevation of main dam wall: 966.00m

Signed…………………………….. Date…27 August 2009…
(Dr W V Pitman)

APPENDIX D: STRUCTURAL REPORT

NGODWANA DAM

SAFETY INSPECTION REPORT –AUGUST 2016

STRUCTURAL SUB-REPORT

TEAM MEMBER / AUTHOR: ALTUS DE BEER PrEng

1 GENERAL

The structural sub-report deals with all the concrete elements of the Ngodwana Dam. There are three main concrete elements on the Ngodwana Dam, i.e.:

- Spillway structure, including retaining walls, slabs and OGEE control.
- Intake tower.
- Outlet conduit.

These elements were inspected by the author on 10 May 2016. Photographs of these structural elements are included in the main report.

2 SPILLWAY STRUCTURE

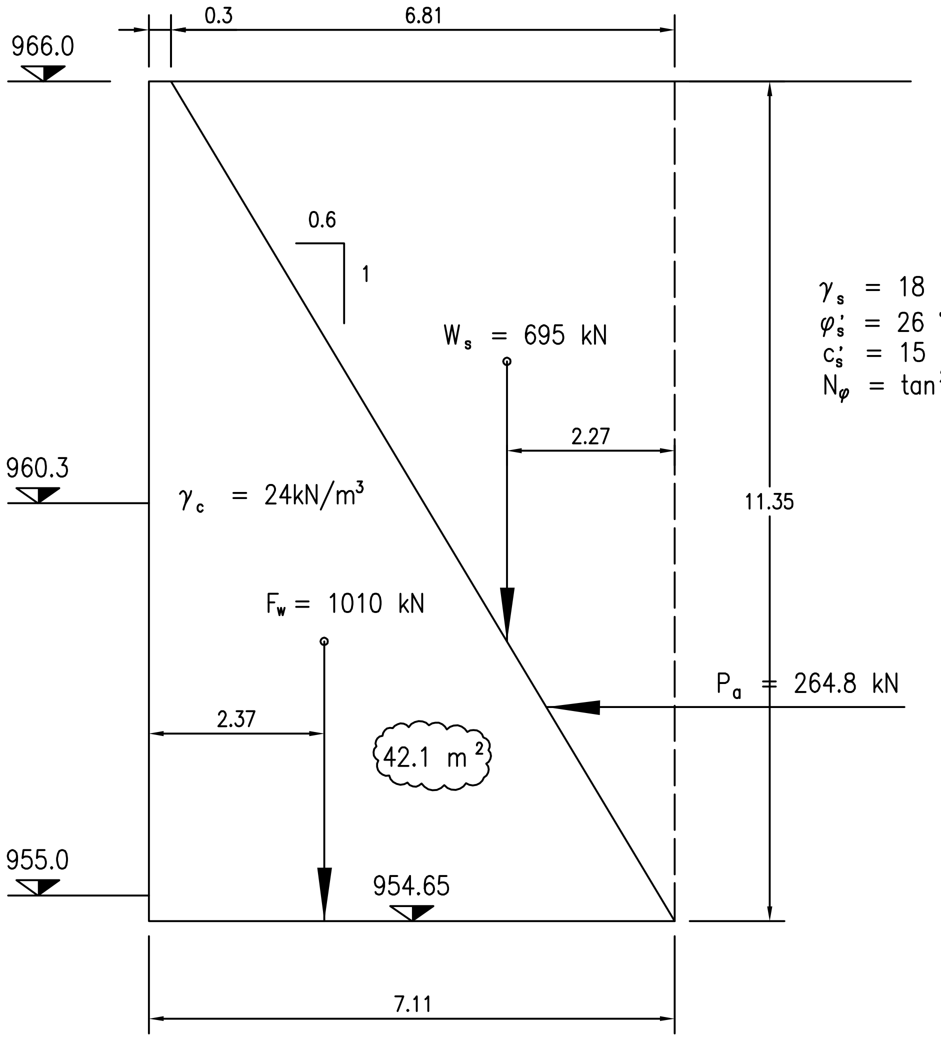
2.1 SPILLWAY FLANK WALLS

The retaining walls flanking the OGEE structure were analysed for stability during the previous inspection and again for this inspection. The critical section was taken as immediately downstream of the OGEE structure. The restraining effect of the OGEE were ignored in both cases. For this inspection, more realistic assumptions were made for the soil action on the wall, resulting in higher overall factors of safety. The assumptions and results of the stability analyses are listed in table 1 below. The Coulomb theory was used for active and passive pressures.

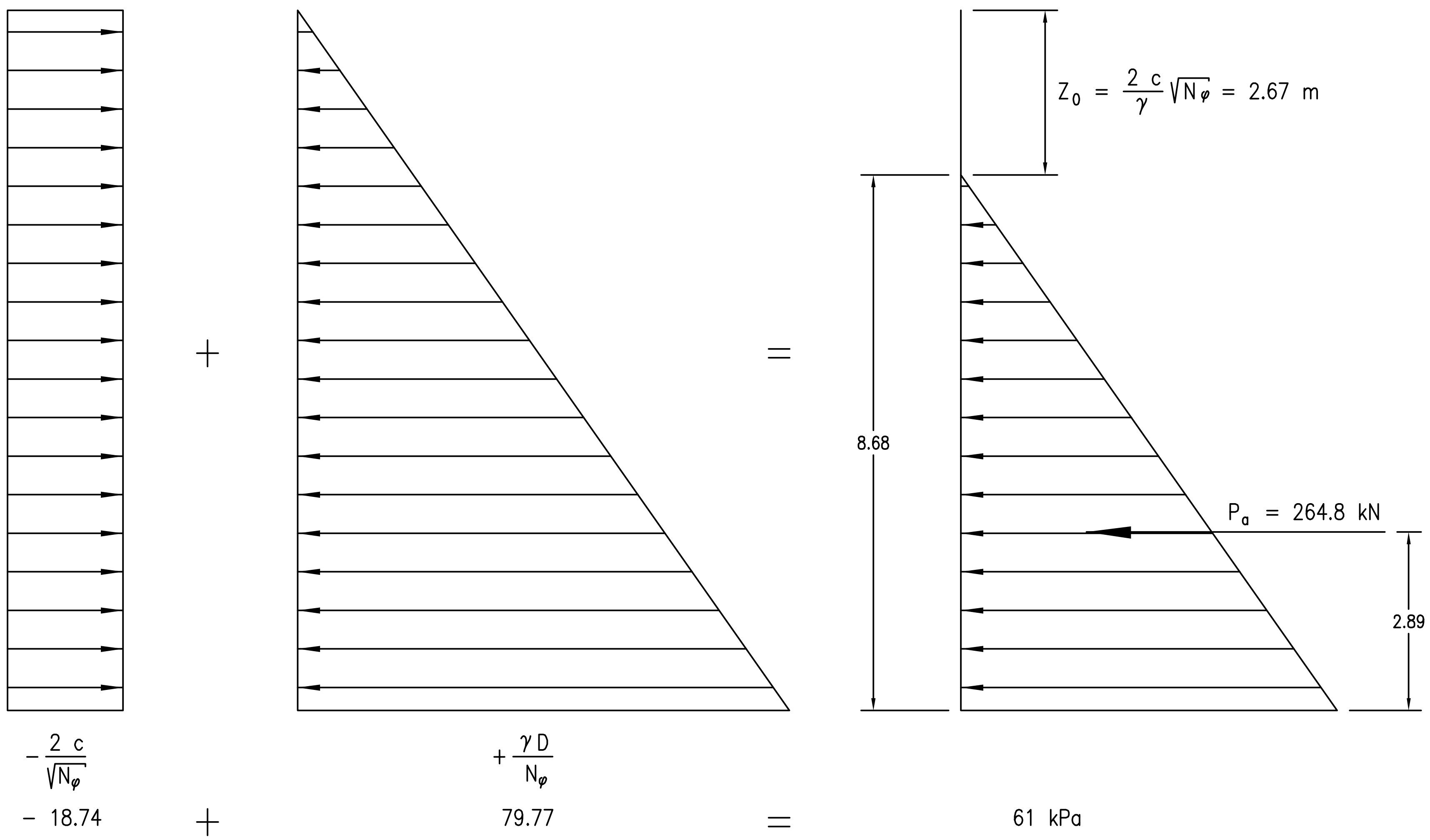
Table 1:- Stability Analysis Results for Spillway Retaining Walls.

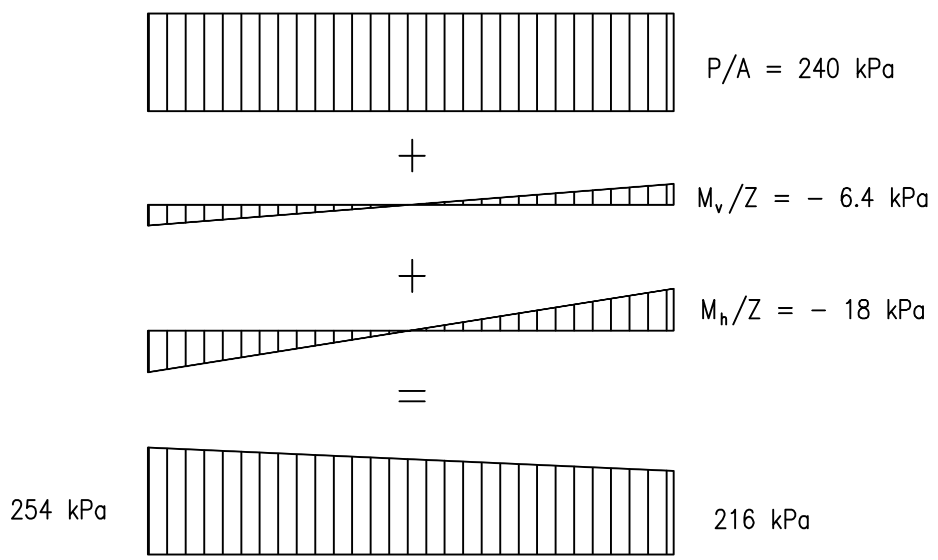
WALL	γ (kN/m^3)	Φ(degrees)	C (kPa)	FOS $_{overt.}$	FOS $_{slide}$
LEFT	18	26	15	2.6	2.3
RIGHT	20	35	0	1.8	18.3

The other structural elements and all recommendations associated therewith, are dealt with in the main report.



Soil active pressure :





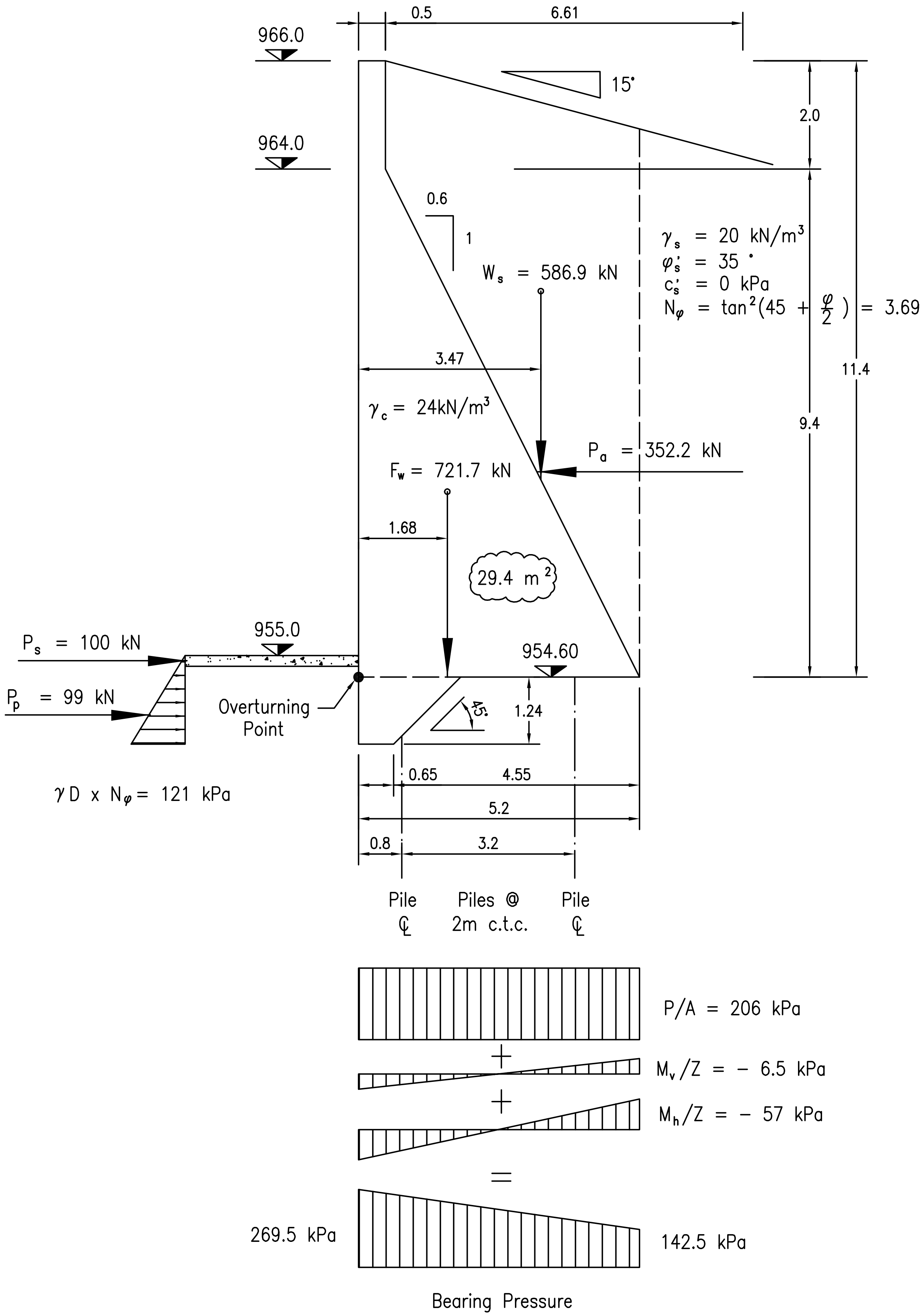
Bearing Pressure

$$FOS_{ovt} = \frac{(F_w \times 2.37) + (W_s \times 4.84)}{(P_a \times 2.89)} = 7.52$$

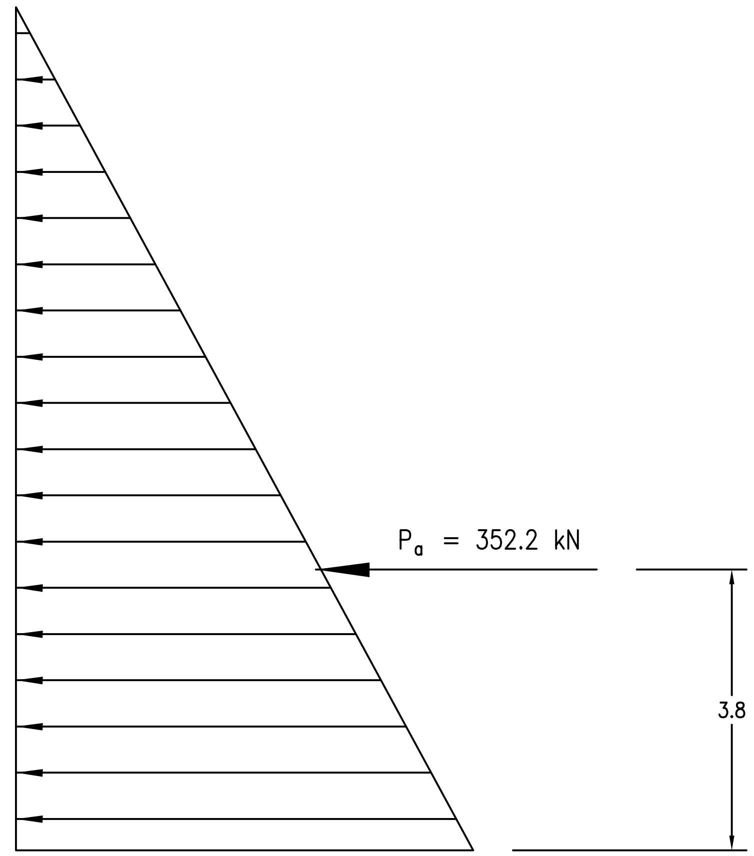
$$FOS_{slide} = \frac{(F_w + W_s)\tan \varphi/2}{P_a} = 1.48$$

0 1 2 3 4 5 6 7 8 9 10

REVISIONS						
DATE: / No.: / DESCRIPTION	NGODWANE DAM SOUTH RETAINING WALL STABILITY CALCULATION	SAPPI NGODWANE PAPER MILL	NGODWANE DAM NORTH RETAINING WALL STABILITY CALCULATION	**Altus de Beer PrEng** **Consulting Engineer** Email: debeeraltus@gmail.com Cell: 083 700 8733 15A Shrublands Drive, Hurlpark, Sandton 2196, South Africa	DRAWING TITLE: NGODWANE DAM SOUTH RETAINING WALL STABILITY CALCULATION	PRINT ISSUED: SCALE: AS SHOWN DATE: 12 SEPT 2016 PLAN No.: ADA9802-01 DESIGNED: A. de Beer DRAWN: R. Petrov CHECKED: REVISION:



Soil active pressure :



$$\frac{\gamma D}{N_\varphi} = 61.78 \text{ kPa}$$

$$FOS_{ovt} = \frac{(F_w \times 1.68) + (W_s \times 3.47)}{(P_a \times 3.8)} = 2.43$$

$$FOS_{slide} = \frac{(F_w + W_s)\tan \varphi/2}{P_a} = 1.17$$

Take into account Passive Pressure and Slab Constraint, then:

$$FOS_{slide} = \frac{(F_w + W_s)\tan \varphi/2 + P_p + P_s}{P_a} = 1.73$$

Pile cap forces

Left pile: − P_L = 558 x 2 = 1116 kN

Right pile: − P_R = 513 x 2 = 1026 kN

0 1 2 3 4 5 6 7 8 9 10

REVISIONS		
DATE:	No.:	DESCRIPTION

NGODWANE DAM
NORTH RETAINING WALL
STABILITY CALCULATION

SAPPI
NGODWANE PAPER MILL

NGODWANE DAM
NORTH RETAINING WALL
STABILITY CALCULATION

Altus de Beer PrEng Consulting Engineer
Email: debeeraltus@gmail.com
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15A Shrublands Drive, Hurlpark, Sandton 2196, South Africa

DRAWING TITLE:
NGODWANE DAM
SOUTH RETAINING WALL
STABILITY CALCULATION

PRINT ISSUED:	
SCALE: AS SHOWN	DESIGNED: A. de Beer
	DRAWN: R. Petrov
DATE: 12 SEPT 2016	CHECKED:
PLAN No.: ADA9802−02	REVISION:

APPENDIX E: MECHANICAL REPORT

DCE

A.R. Du Plessis Consulting Engineers
(A.R. Du Plessis and Associates t/a)

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SONPARK
1207

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FAX : (013) 734 6075
CELL : 082 850 4417
E-MAIL: andre.duplessis@sappi.com

NGODWANA DAM MECHANICAL INSPECTION REPORT

Complied by: André du Plessis
Date: 6 June 2016

SUMMARY

DCE conducted a visual inspection on the mechanical aspects of the Ngodwana dam on 23 May 2016.

In summary the following main items should be noted:

- Pipe supports are severely corroded and needs to be replaced/repaired.
- Water damming below the 1000mm line needs to be resolved, pipe external corrosion is occurring.

It is important to resolve the corroded pipe supports, this could have a negative impact on the pipe integrity at the point of attachment to the pipe support.

The conclusion of the inspection is that the mechanical equipment is safe to operate, but that the corrosion related issues should be resolved as a matter of urgency.

TERMS OF REFERENCE

DCE-1400-15

DOCUMENT DISTRIBUTION, REVISION AND APPROVAL HISTORY

Rev	Date	Distribution
1	6 June 2016	Issue to Client

CONTENTS

1. BACKGROUND

DCE was appointed to do the 5 yearly statutory mechanical inspection of the Ngodwana dam.

A visual inspection was done on 23 May 2016.

Sappi submitted wall thickness reports on piping.

2. METHOD STATEMENT

A visual inspection was done on valves, piping, pipe supports and structures.

Results of wall thickness test reports were evaluated.

The scour valve was opened and mechanical functionality witnessed.

Recommendations were made.

3. REFERENCE DRAWINGS & DOCUMENTATION

Description	**Document Number**	**Rev**	**Revision Date**
NDT Report	NG15-007	-	28Aug15

4. ASSUMPTIONS

Assumptions:

- Wall thickness measurements in 2003 was incorrect.

5. FINDINGS

<u>Piping & Pipe Supports</u>

Wall thickness testing results, of external piping, show an average wall thickness of 9.57 mm with no significant average material loss since the last measurements in 2009 with an average of 9.66.

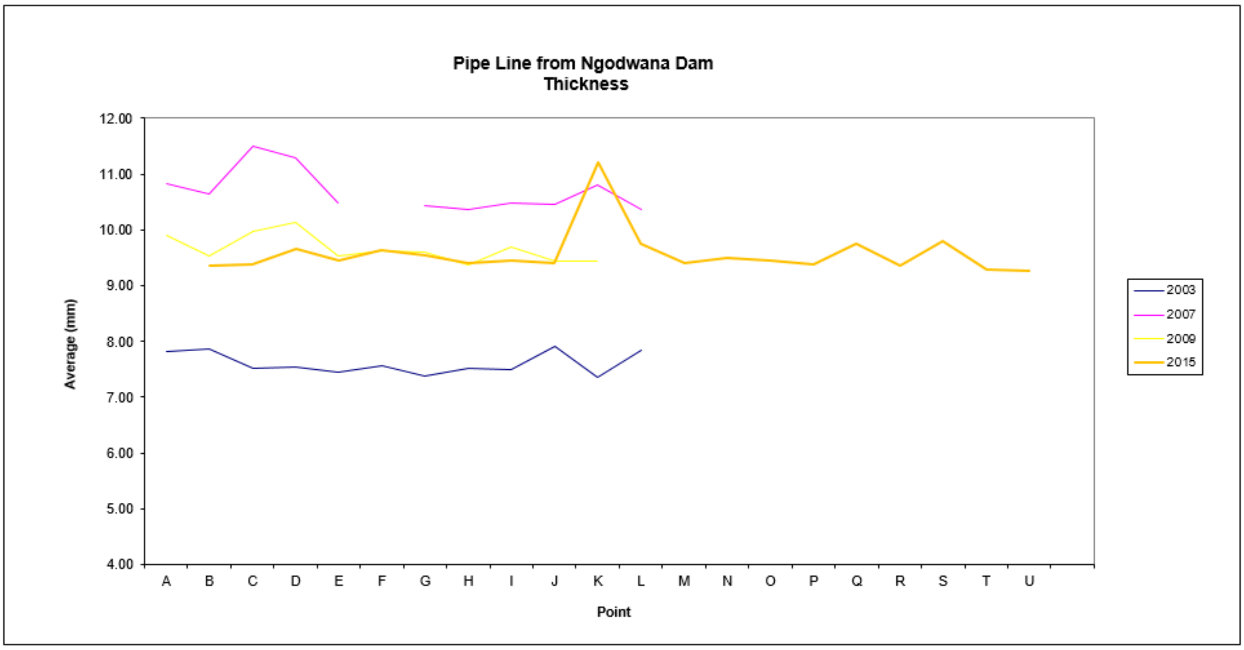
Additional measuring points where introduced not measured during previous years.

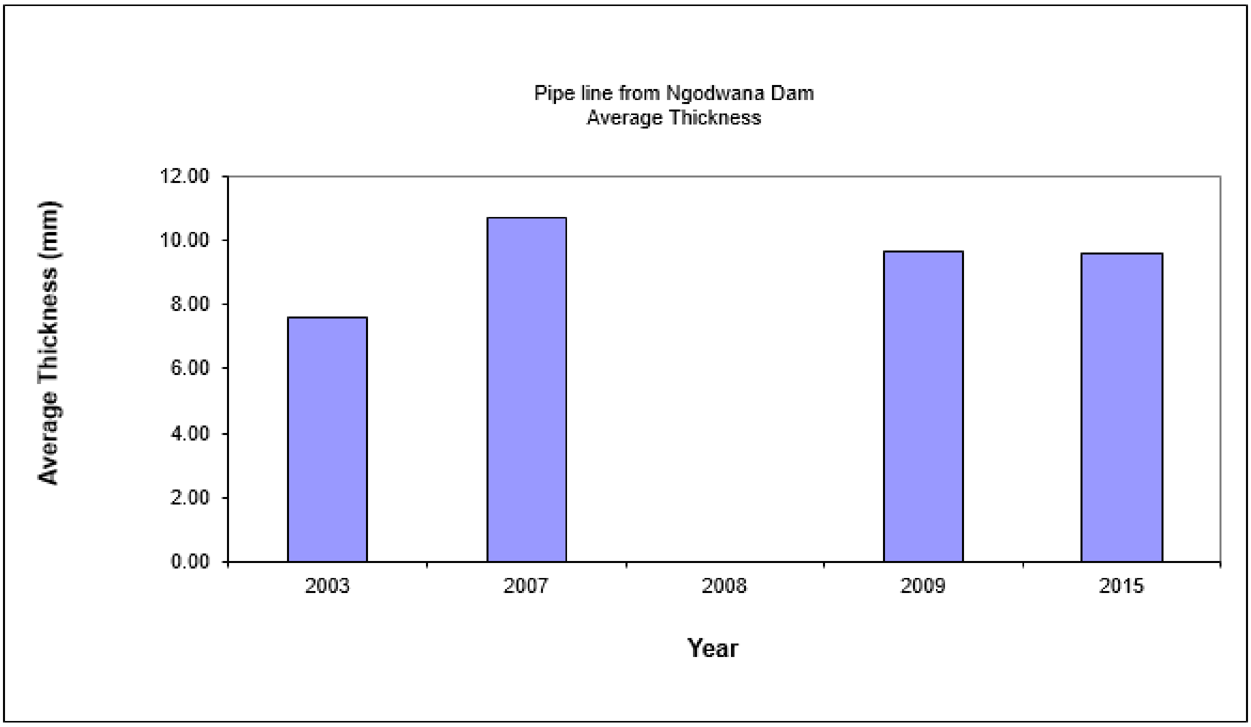
Thickness measurements at point K has increased by +- 1.75mm since measurements in 2009 which doesn’t make sense.

Thicknesses in area C & D decreased by 0.5mm.

Pipe wall thickness is still sufficient.

No wall thickness measurements are available of the pipe sections below the dam wall.





Pipe supports are severely corroded.

Most of the piping corrosion protection system is in a good condition requiring spot repairs in some areas *[Photo-A1/B3]*, with the exception of a section of the 1000mm pipe where exposed to water*[Photo-A13/B13/C12]*.

Water is damming up below the 1000mm pipe, this is leading to severe corrosion of the pipe supports and bottom of the pipe in this area.

Valves

The scour valve was opened and operated satisfactory. A lubrication schedule is in place and is being followed.

While operating the scour valve it was noted that water build-up downstream of the valve occurs and then floods the tunnel under the dam wall – plant material is then carried into the tunnel. This could be contributing to the damming of water inside the tunnel.

The other valves where visually inspected externally only.

Bolts and nuts are badly corroded on drain valves at the bottom of the tower *[Photo-B1/A2//B2/A10/B10*].

Structures

Access structures where visually inspected. Some corrosion was observed *[Photo – C10/B16/A4]*.
Grating hold down clamps are corroded.

6. RECOMMENDATIONS

Piping & Pipe Supports

- Repair and replace steel pipe supports
- Corrosion protection system on all pipe supports to be re-applied
- Improve drainage on area where water is damming up below the 1000mm pipe to ensure the pipe is not submerged in water.
- Annual schedule be put in place to ensure no water damming which exposes pipe and pipe supports to corrosion.
- Included piping below the dam when thickness tests are done.

Valves

- Corrosion protection system on drain valves to be re-applied
- Scour valve drainage area be improved to ensure water does not flow back into the tunnel.

Structures

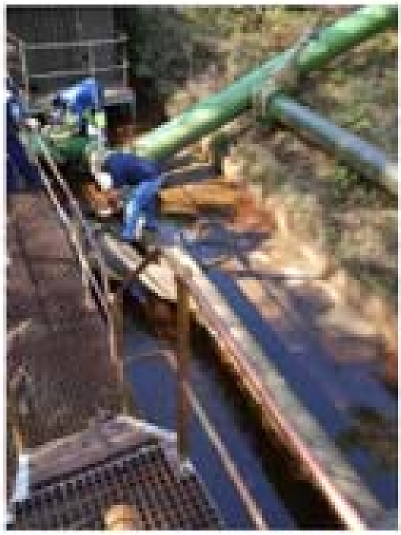
- Corrosion protection system to be touched-up.
- Grating hold-down clamp to be replaced.
- The installation of additional hand railing should be considered to improve safety on access structures.

7. ATTACHMENTS

7.1 PHOTO REFERENCE

A B C

1
2
3

	A	B	C
4			
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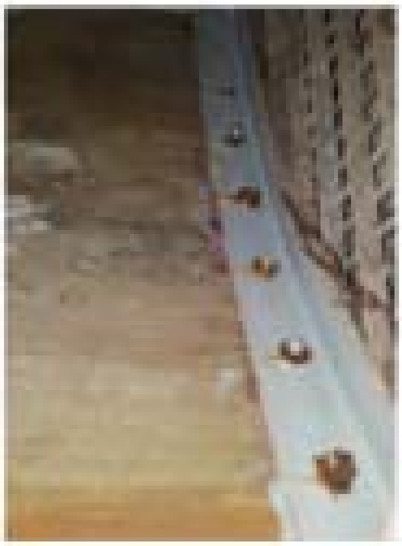

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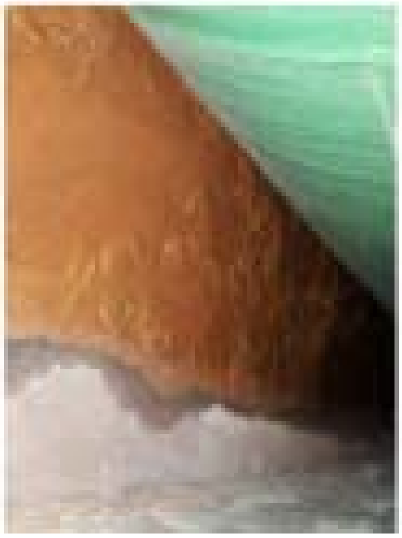

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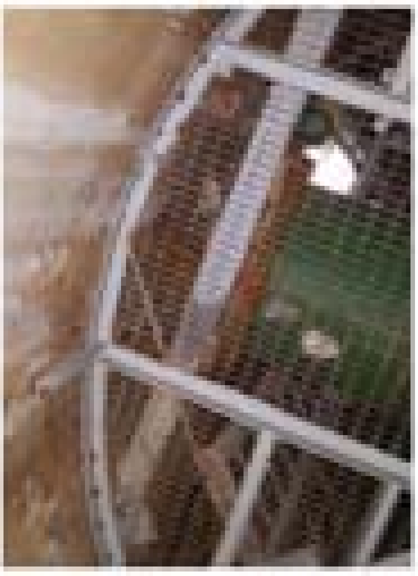
16			
17			
18			

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7.2 NDT REPORT

NDT INSPECTION

12 Mitco Industrial Park
Houtkop road
Vereeniging

VAT NO. 4170231312
REG NO. 2005/147907/23

Telephone: (016) 428 5735
Cell phone: 082 854 8055
083 280 4488
Fax: 086 588 4664/5/6
Email: ndtinspection@telkomsa.net
Web: ndtinspection.co.za

133 203 733

WT:	NG15-007
DATE:	28 August 2015

WALL THICKNESS TEST REPORT

DETAILS OF COMPONENT	*Client:*	Sappi Southern Africa	*Contact Person:*	J.Bischoff
	Description:	Mill Reservoir Main Line	*Plant:*	Outside Service
	Manufacturer:	In Service	*Item No.:*	KN/UTI -FWT - FER -SUPPL - 1130530
	Material:	Stainles steel	*Drawing No.:*	1140002
TECHNICAL DATA	*Apparatus:*	SIUI	*Type and Number:*	CTS-30A
	Calibration Date:	21 April 2015	*Surface Cond.:*	Wire Brushed / Paint removed
	Test block Type:	4mm Test block	*Calibration Rangs:*	0 mm - 30 mm
	Couplant:	GEL	*Procedure No.:*	UT-001-REV.! 3

TEST RESULTS

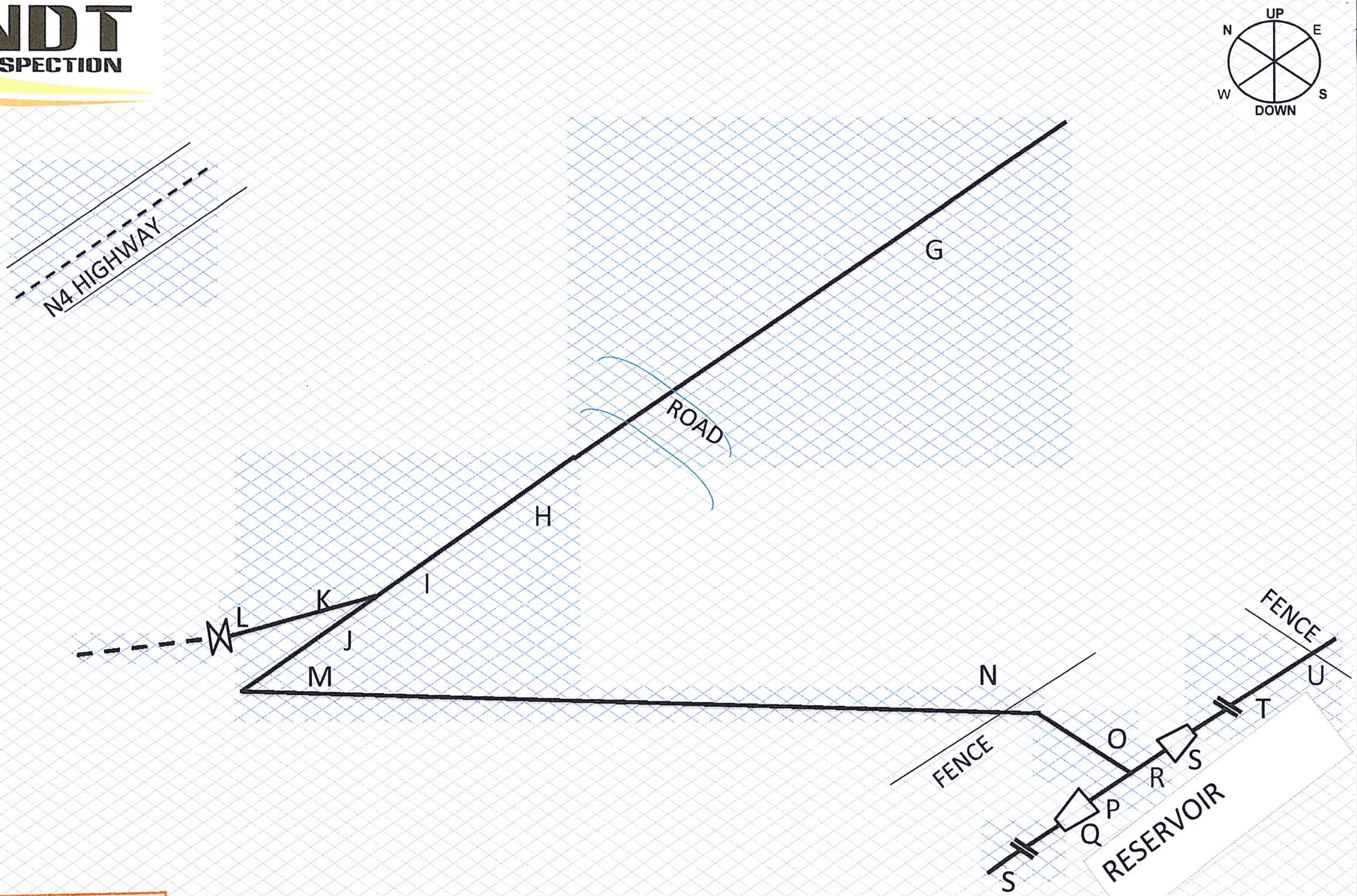
Position	A	B	C	D	E	F	G	H	I
1	N/A	9.3	9.1	9.5	9.2	9.3	9.3	9.1	9.2
2	N/A	9.7	9.3	9.4	9.4	9.7	9.5	9.5	9.2
3	N/A	9.1	9.3	9.8	9.6	9.9	9.6	9.4	9.5
4	N/A	9.3	9.8	9.9	9.6	9.6	9.8	9.6	9.9

Position	J	K	L	M	N	O	P	Q	R
1	9.0	11.6	9.6	9.1	9.2	9.5	9.6	9.7	9.2
2	9.3	10.7	10.1	9.7	9.5	9.7	9.4	10.0	9.4
3	10.2	11.5	10.0	N/A	9.9	9.4	9.1	9.6	9.2
4	9.1	11.0	9.3	9.4	9.4	9.2	9.4	9.7	9.6

Position	S	T	U						
1	9.9	9.3	9.4						
2	9.6	9.4	9.4						
3	9.9	9.3	9.5						
4	9.8	9.2	9.2						

Performed by:	E.Benade	*Inspection Authority:*	
Qualification:	SNT-TC-1A II		
Date:	28 August 2015	*Date:*	

NDT INSPECTION
J.S. BENADE
SNT TC 1A L1 RT/MT/PT
SAQCC WELDING INSPECTOR L1



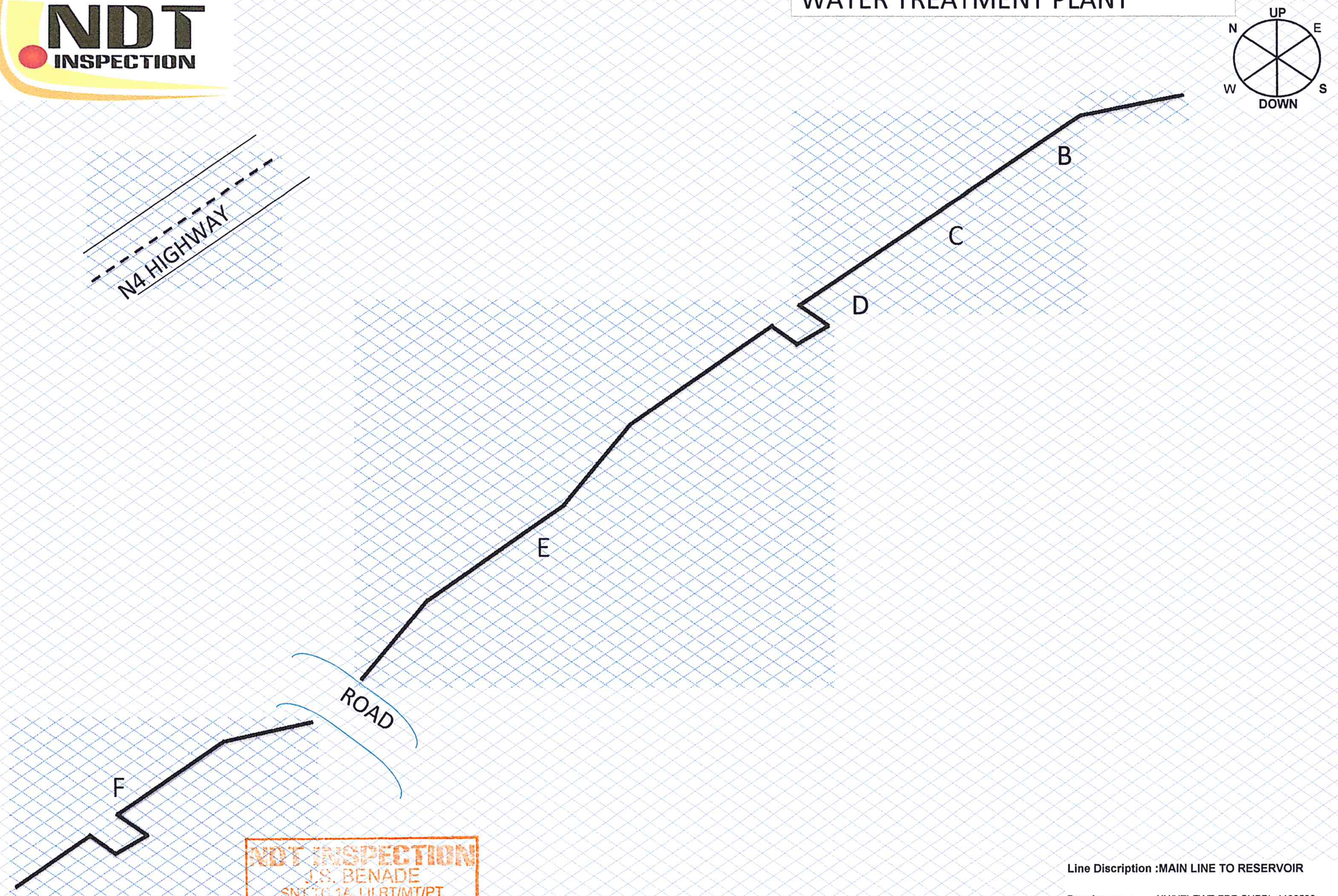
NDT INSPECTION
J.S. BENADE
SNT TC 1A LII RT/MT/PT
SAQCC WELDING INSPECTOR L1

Line Discription :MAIN LINE TO RESERVOIR

Drawing no : KN/UTI-FWT-FRE-SUPPL-1130530
Raport no : : NG15-007 1 OF 2
DRAWN BY : E.Benade

Sappi Outside services

WATER TREATMENT PLANT



NDT INSPECTION
J.S. BENADE
SNT TC 1A LII RT/MT/PT
SAQCC WELDING INSPECTOR L1

Line Discription :MAIN LINE TO RESERVOIR

Drawing no : KN/UTI-FWT-FRE-SUPPL-1130530
Raport no : : NG15-007 1 OF 2
DRAWN BY : E.Benade

Sappi Outside services

APPENDIX F: PHOTOGRAPHIC RECORD

PLATE 1:- Ngodwana Dam spillway and dam wall viewed from downstream

PLATE 2:- Standpipe piezometer SP4 on dam crest at “riverbed” section.

PLATE 3:- Standpipe piezometers SP4-7 along “riverbed” section

PLATE 4:- Standpipe piezometer SP1 on dam wall crest at “culvert” section

PLATE 5:- Standpipe piezometers SP1-3 along “culvert” section

PLATE 6:- Spillway return channel and “bandaging” of joints

PLATE 7:- Spillway return channel

PLATE 8:- Spillway OGEE control structure and return channel

PLATE 9:- Spillway OGEE control structure - grass growing in construction joints

PLATE 10:- Termite nest (now dormant) on riverbed section SP4-7

PLATE 11:- Wet spot (green grass) near bottom of embankment at left flank

PLATE 12:- Dam basin and intake tower

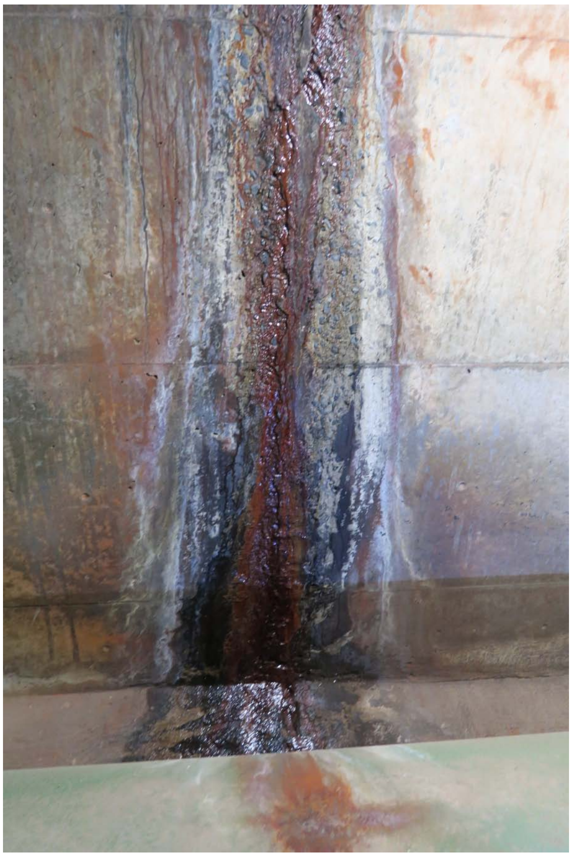


PLATE 13:- Embankment material seeping into outlet conduit at CH 106 (as measured from inside face of outlet tower in downstream direction).

PLATE 14:- material seeping into outlet conduit at CH 106 (as measured from inside face of outlet tower in downstream direction).

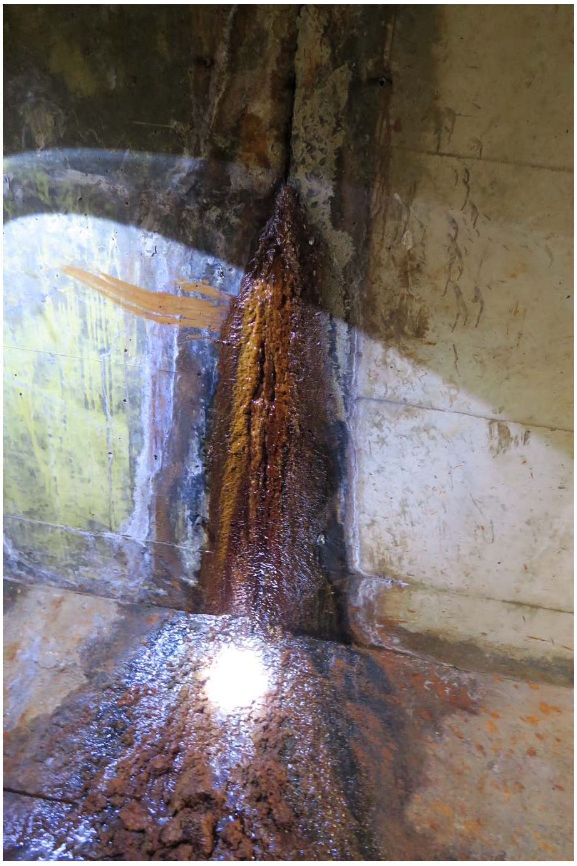


PLATE 15:- material seeping into outlet conduit at CH 63 (as measured from inside face of outlet tower in downstream direction).

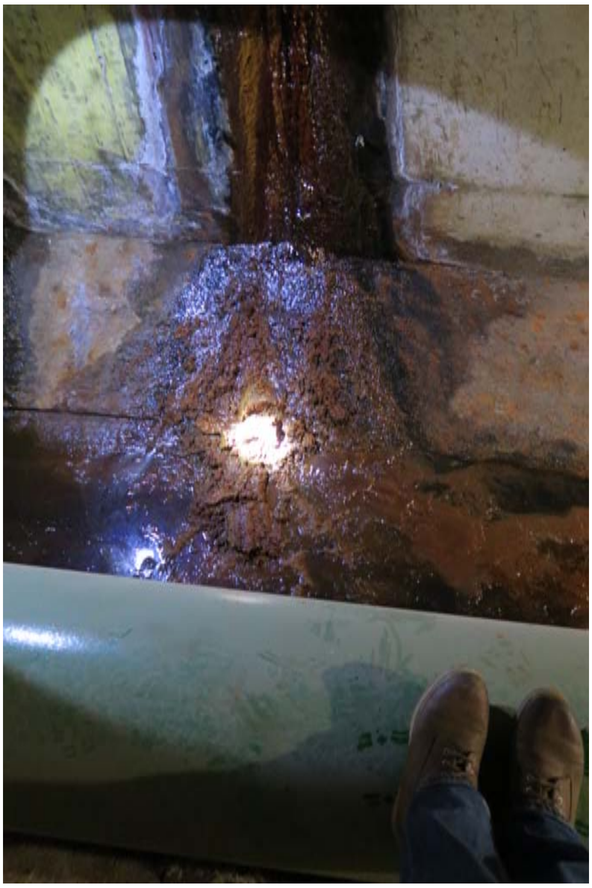


PLATE 16:- material seeping into outlet conduit at CH 63 (as measured from inside face of outlet tower in downstream direction).

PLATE 17:- material seeping into outlet conduit at CH 62 (as measured from inside face of outlet tower in downstream direction).

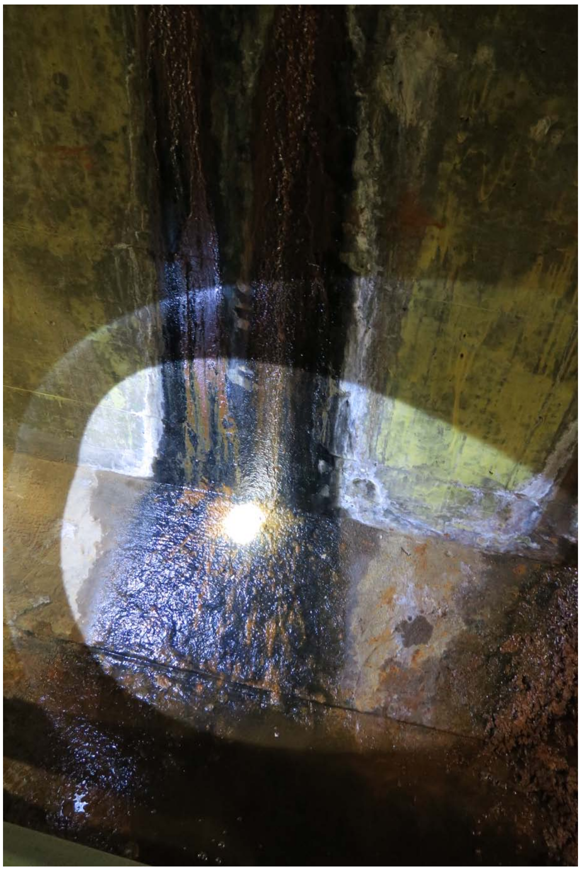
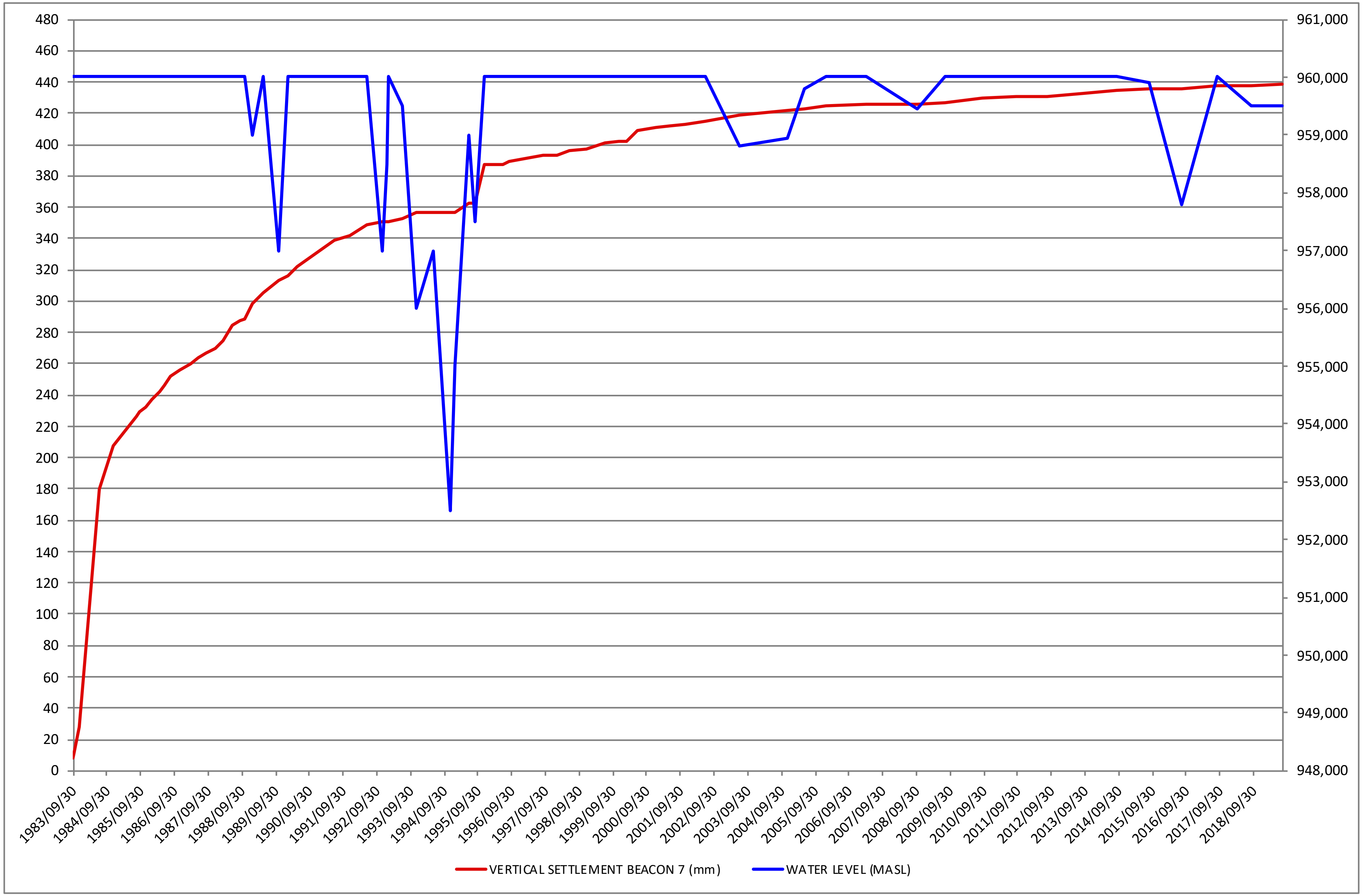


PLATE 18:- material seeping into outlet conduit at CH 62 (as measured from inside face of outlet tower in downstream direction).

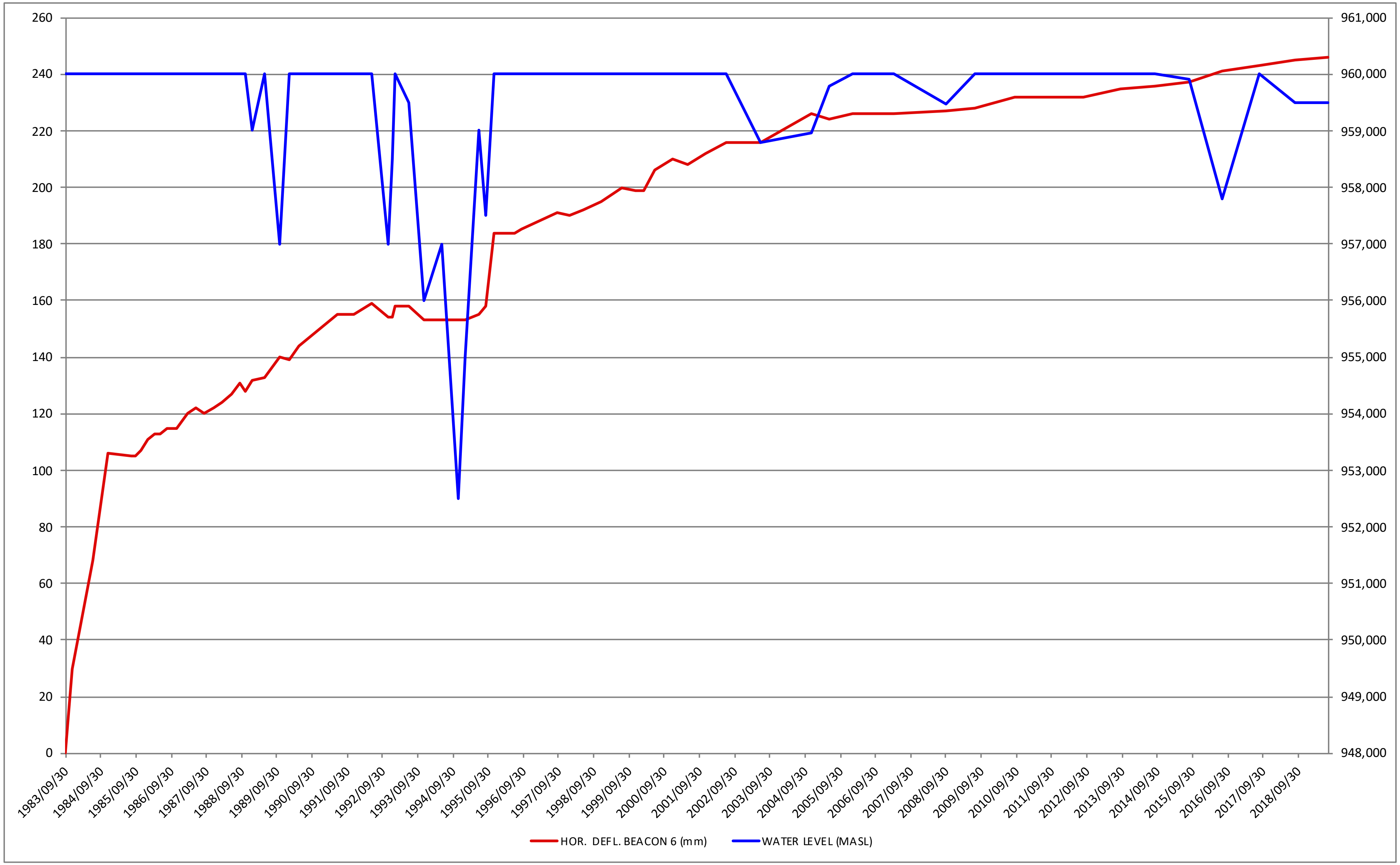
APPENDIX G: MONITORING RECORDS

The monitoring records and summarised results for the Ngodwana Dam rainfall, water level, seepage, settlement and piezometer readings are attached overleaf, in both tabular and graphical form, for almost the entire record since completion of the dam wall.

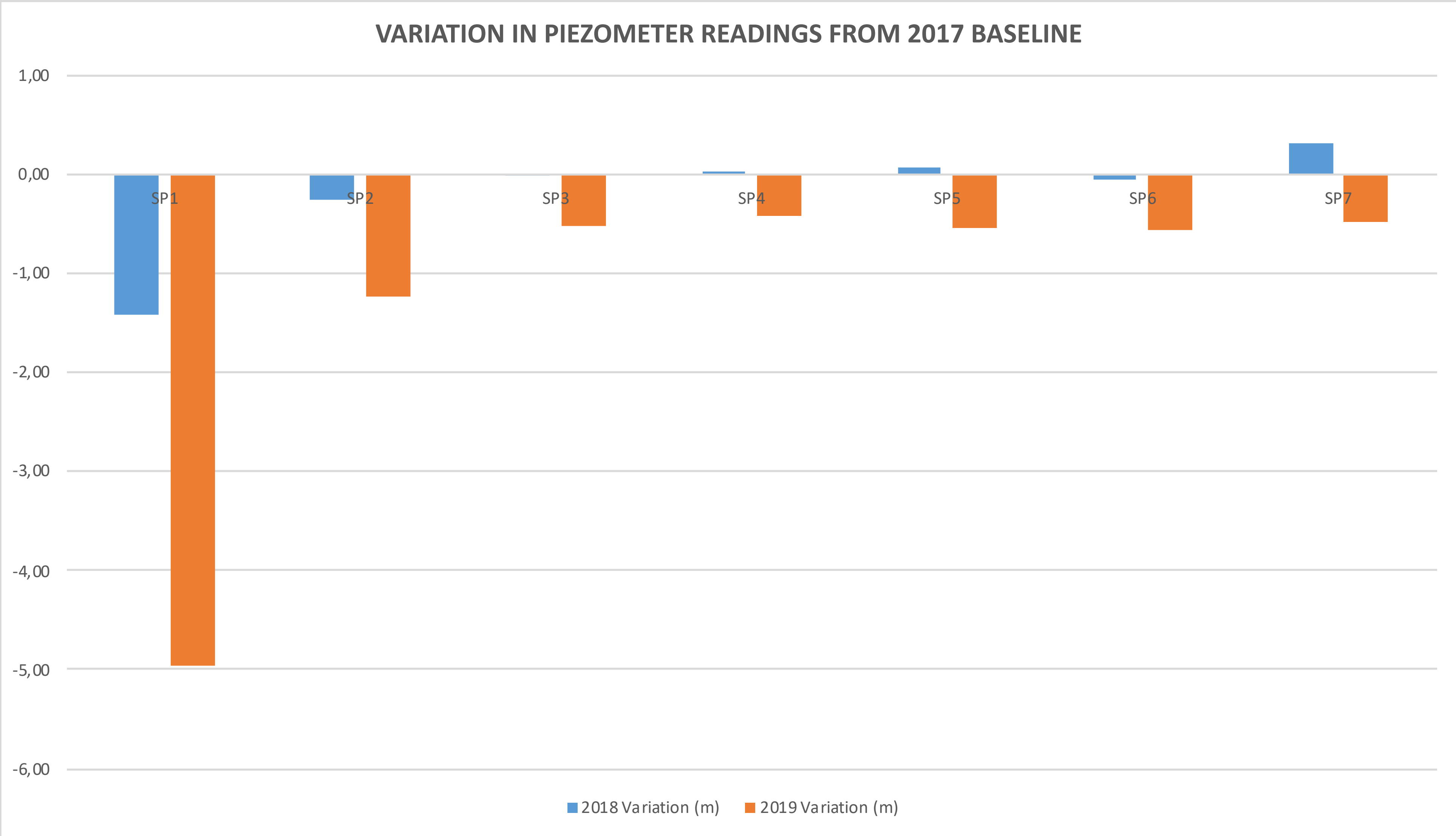
The interpretation and implications of the monitoring records are more fully discussed in the body of the report.



Ngodwana Dam Vertical Settlement at Beacon 7 (maximum) vs Water Level in Dam Basin



Ngodwana Dam Horisontal Displacement (Downstream) at Beacon 6 (maximum) vs Water Level in Dam Basin



VARIATION IN PIEZOMETER READINGS FROM 2017 BASELINE
1,00
0,00
-1,00
-2,00
-3,00
-4,00
-5,00
-6,00
SP1
SP2
SP3
SP4
SP5
SP6
SP7
2018 Variation (m)
2019 Variation (m)

APPENDIX H – SEEPAGE ANALYSIS

SECTION A-A "CULVERT"

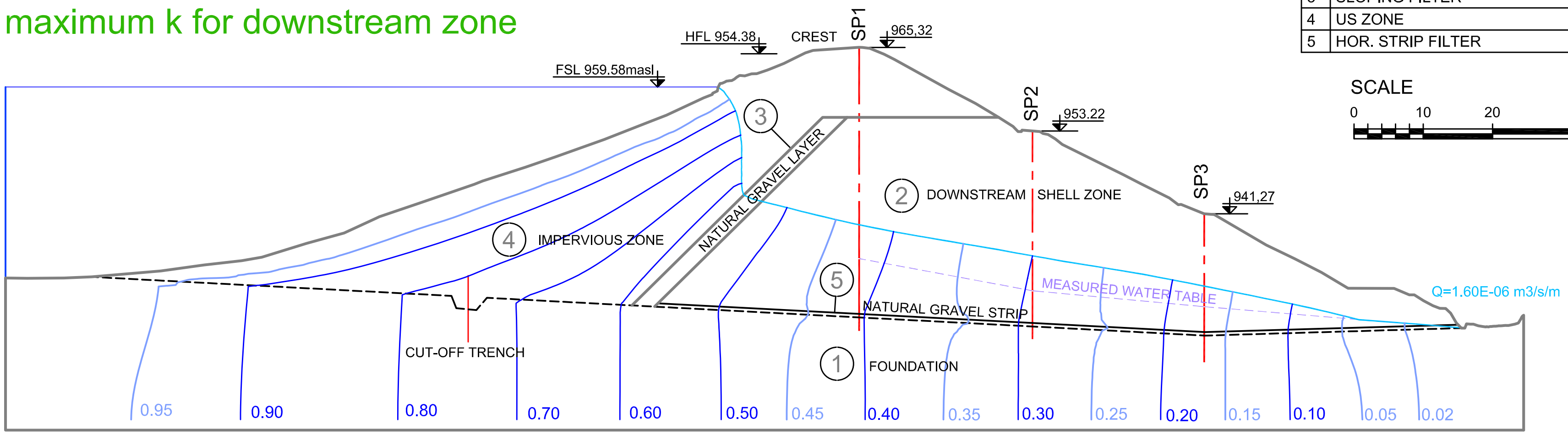
Case 1a - Filters fully functional, maximum k for downstream zone

X cels	Y cels	Accuracy	Iterations
150	40	5.000E-05 / 1.214E-03	1.500E+05 / 1.500E+05

No	Seep Layer Name	Kh [m/s]	Kv [m/s]
1	FOUNDATION	6.500E-07	6.500E-07
2	DS SHELL	6.500E-09	6.500E-09
3	SLOPING FILTER	1.000E-04	1.000E-04
4	US ZONE	1.000E-09	1.000E-09
5	HOR. STRIP FILTER	1.000E-05	1.000E-05

SCALE

0 10 20 40 60



SECTION A-A "CULVERT"

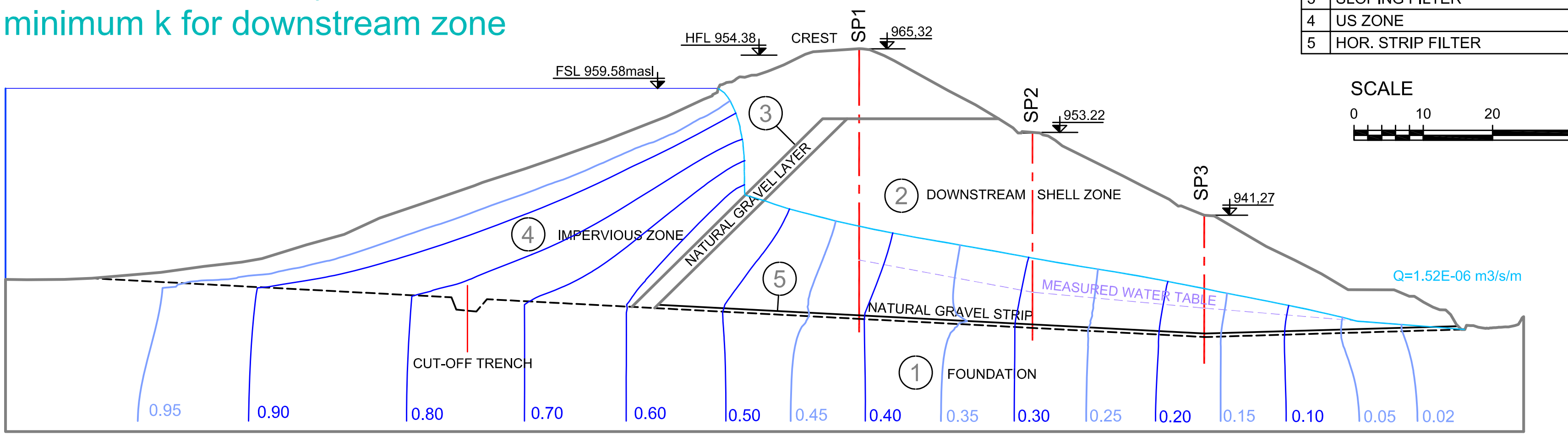
Case 1b - Filters fully functional, minimum k for downstream zone

X cels	Y cels	Accuracy	Iterations
150	40	5.000E-05 / 1.372E-03	1.500E+05 / 1.500E+05

No	Seep Layer Name	Kh [m/s]	Kv [m/s]
1	FOUNDATION	6.500E-07	6.500E-07
2	DS SHELL	3.100E-09	3.100E-09
3	SLOPING FILTER	1.000E-04	1.000E-04
4	US ZONE	1.000E-09	1.000E-09
5	HOR. STRIP FILTER	1.000E-05	1.000E-05

SCALE

0 10 20 40 60



SECTION A-A "CULVERT"

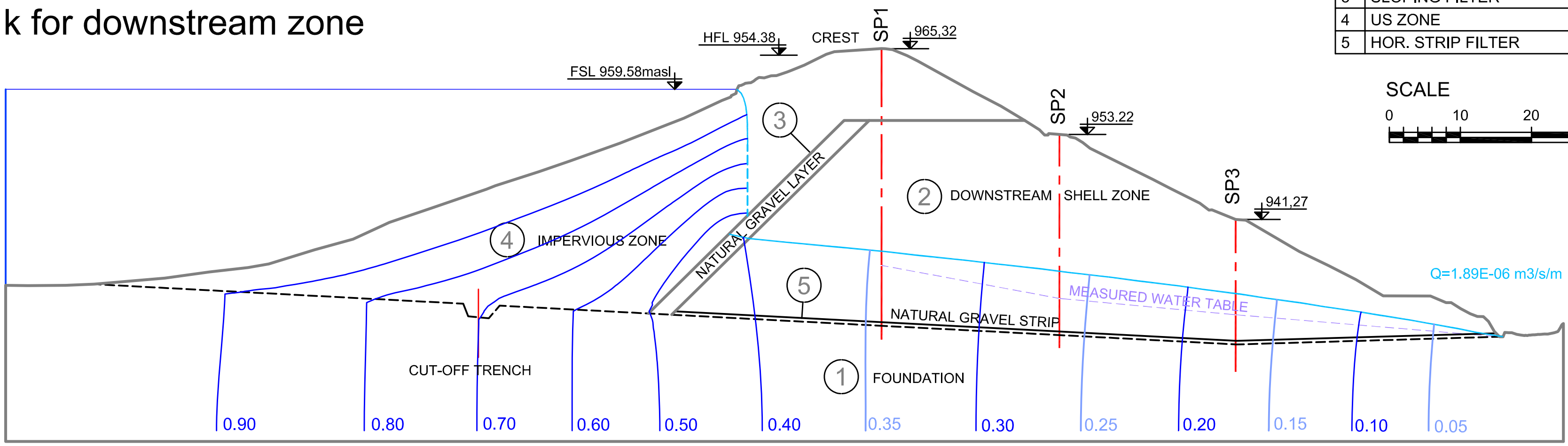
X cels	Y cels	Accuracy	Iterations
150	40	5.000E-05 / 1.255E-03	1.500E+05 / 1.500E+05

No	Seep Layer Name	Kh [m/s]	Kv [m/s]
1	FOUNDATION	6.500E-07	6.500E-07
2	DS SHELL	6.500E-07	6.500E-07
3	SLOPING FILTER	6.500E-07	6.500E-07
4	US ZONE	1.000E-09	1.000E-09
5	HOR. STRIP FILTER	6.500E-07	6.500E-07

SCALE

0 10 20 40 60

Case 2a - Filters clogged, maximum k for downstream zone



SECTION A-A "CULVERT"

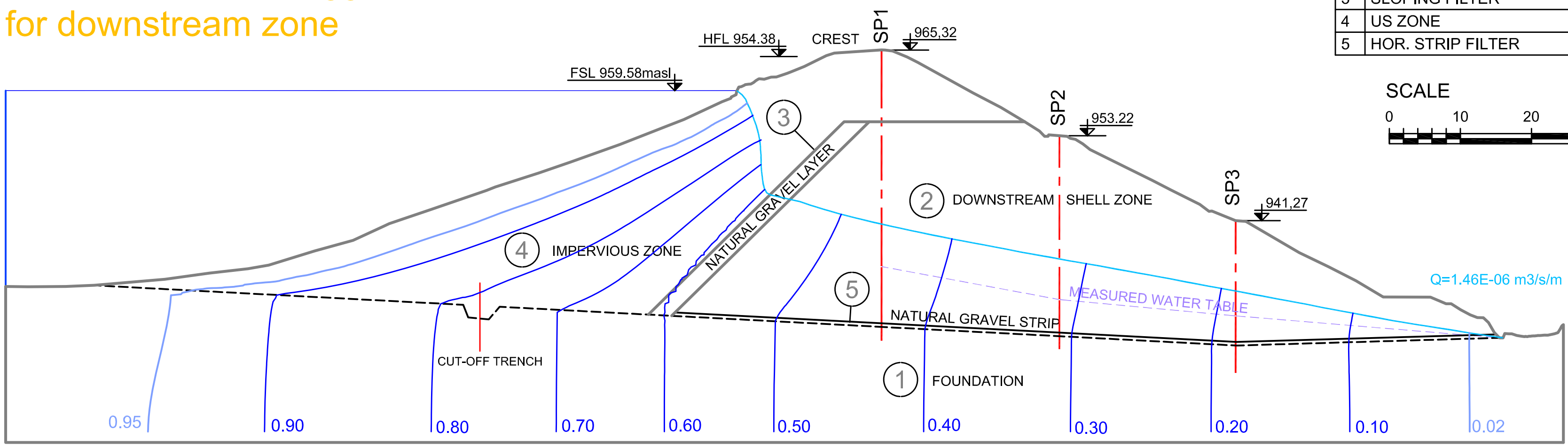
X cels	Y cels	Accuracy	Iterations
150	40	5.000E-05 / 1.330E-03	1.500E+05 / 1.500E+05

No	Seep Layer Name	Kh [m/s]	Kv [m/s]
1	FOUNDATION	6.500E-07	6.500E-07
2	DS SHELL	3.100E-09	3.100E-09
3	SLOPING FILTER	6.500E-07	6.500E-07
4	US ZONE	1.000E-09	1.000E-09
5	HOR. STRIP FILTER	6.500E-07	6.500E-07

SCALE

0 10 20 40 60

Case 2b - Filters clogged, minimum k for downstream zone



SECTION B-B "RIVERBED"

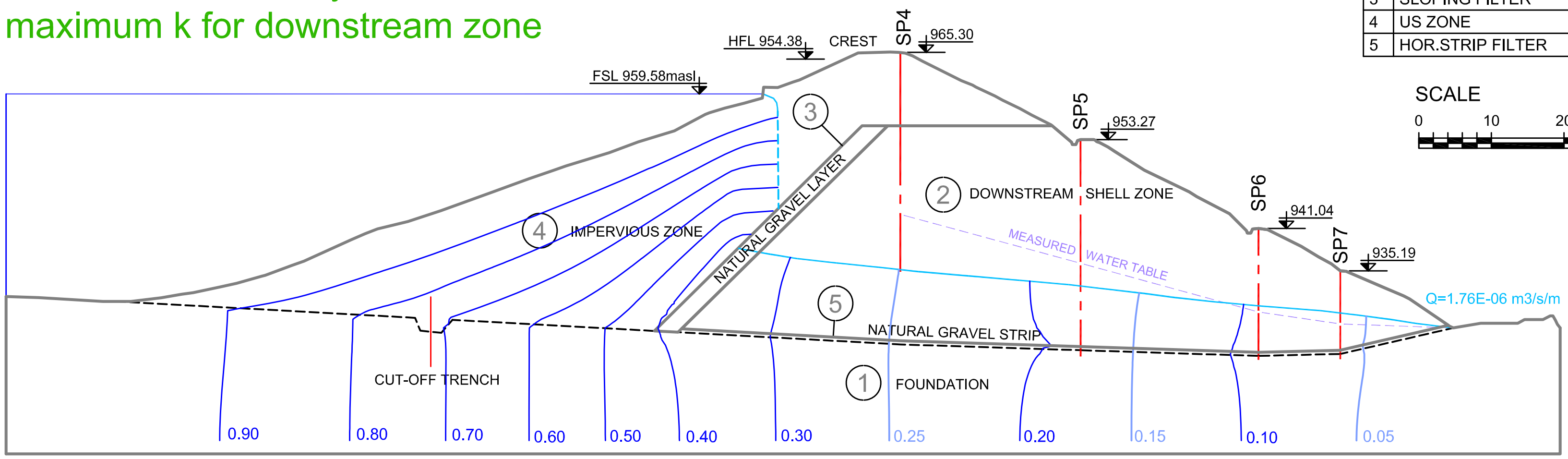
X cels	Y cels	Accuracy	Iterations
160	40	5.000E-05 / 1.612E-03	1.500E+05 / 1.500E+05

No	Seep Layer Name	Kh [m/s]	Kv [m/s]
1	FOUNDATION	6.500E-07	6.500E-07
2	DS SHELL	6.500E-07	6.500E-07
3	SLOPING FILTER	1.000E-04	1.000E-04
4	US ZONE	1.000E-09	1.000E-09
5	HOR.STRIP FILTER	1.000E-05	1.000E-05

SCALE

0 10 20 40 60

Case 1a - Filters fully functional, maximum k for downstream zone



SECTION B-B "RIVERBED"

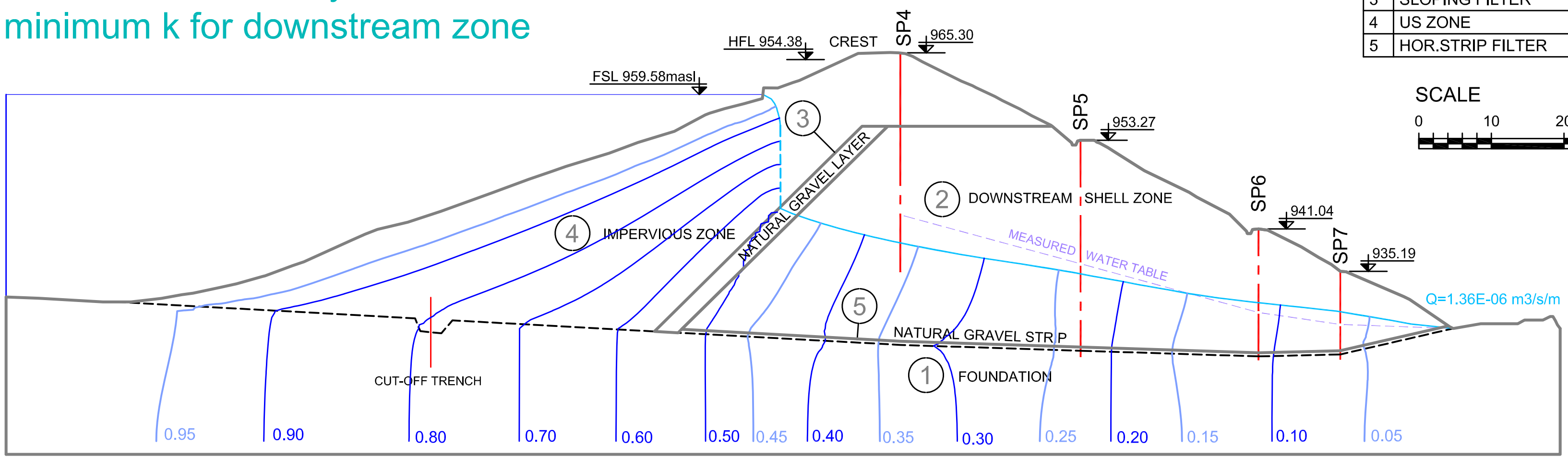
X cels	Y cels	Accuracy	Iterations
160	40	5.000E-05 / 1.623E-03	1.500E+05 / 1.500E+05

No	Seep Layer Name	Keh [m/s]	Kv [m/s]
1	FOUNDATION	6.500E-07	6.500E-07
2	DS SHELL	3.100E-09	3.100E-09
3	SLOPING FILTER	1.000E-04	1.000E-04
4	US ZONE	1.000E-09	1.000E-09
5	HOR.STRIP FILTER	1.000E-05	1.000E-05

SCALE

0 10 20 40 60

Case 1b - Filters fully functional, minimum k for downstream zone



SECTION B-B "RIVERBED"

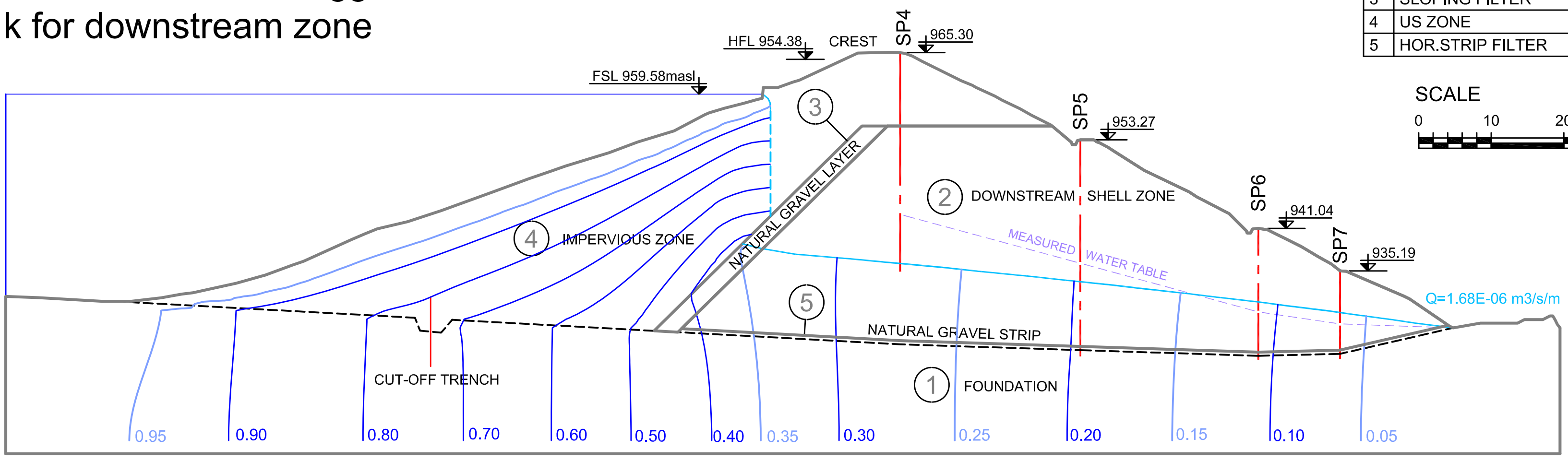
X cels	Y cels	Accuracy	Iterations
160	40	5.000E-05 / 1.519E-03	1.500E+05 / 1.500E+05

No	Seep Layer Name	Kh [m/s]	Kv [m/s]
1	FOUNDATION	6.500E-07	6.500E-07
2	DS SHELL	6.500E-07	6.500E-07
3	SLOPING FILTER	6.500E-07	6.500E-07
4	US ZONE	1.000E-09	1.000E-09
5	HOR.STRIP FILTER	6.500E-07	6.500E-07

SCALE

0 10 20 40 60

Case 2a - Filters clogged, maximum k for downstream zone



SECTION B-B "RIVERBED"

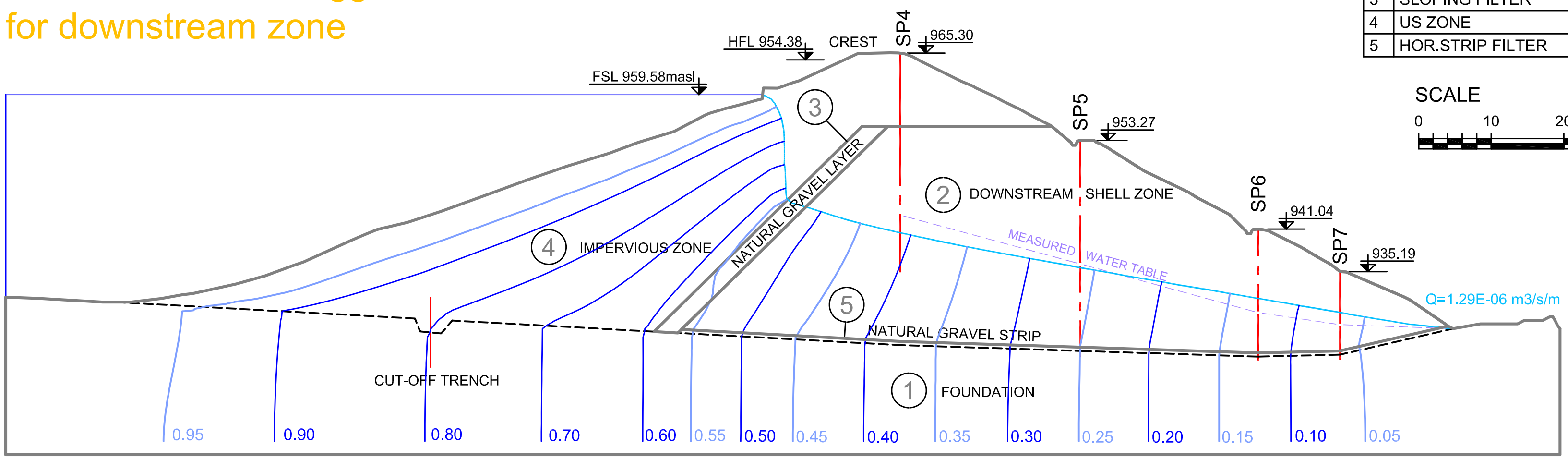
X cels	Y cels	Accuracy	Iterations
160	40	5.000E-05 / 1.528E-03	1.500E+05 / 1.500E+05

No	Seep Layer Name	Kh [m/s]	Kv [m/s]
1	FOUNDATION	6.500E-07	6.500E-07
2	DS SHELL	3.100E-09	3.100E-09
3	SLOPING FILTER	6.500E-07	6.500E-07
4	US ZONE	1.000E-09	1.000E-09
5	HOR.STRIP FILTER	6.500E-07	6.500E-07

SCALE

0 10 20 40 60

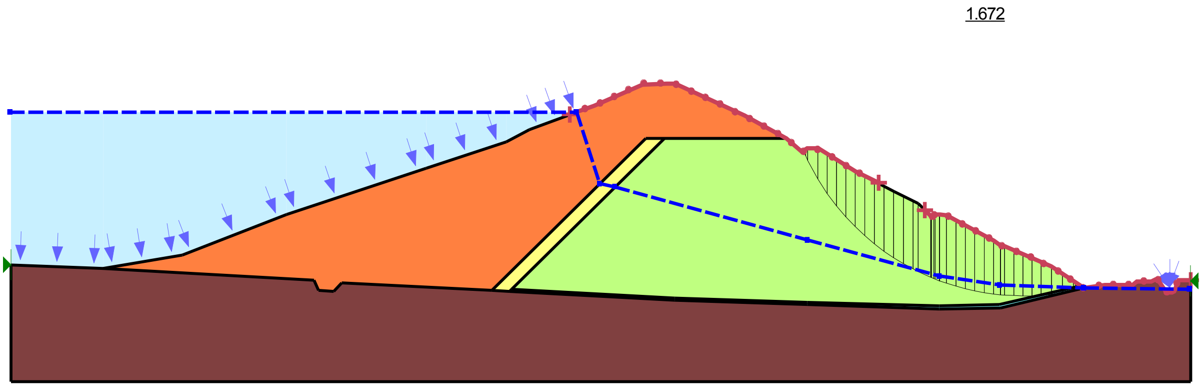
Case 2b - Filters clogged, minimum k for downstream zone



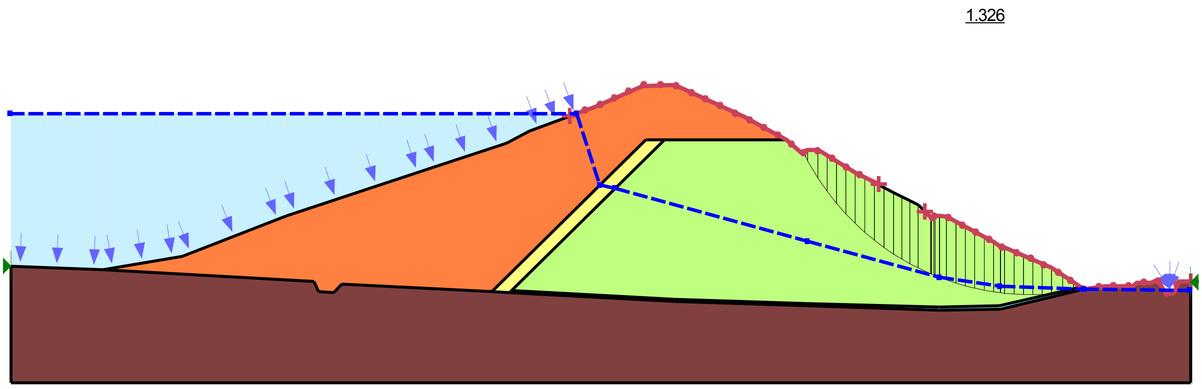
APPENDIX I – SLOPE STABILITY ANALYSIS

Table 2: Calculated minimum factors of safety for slope stability

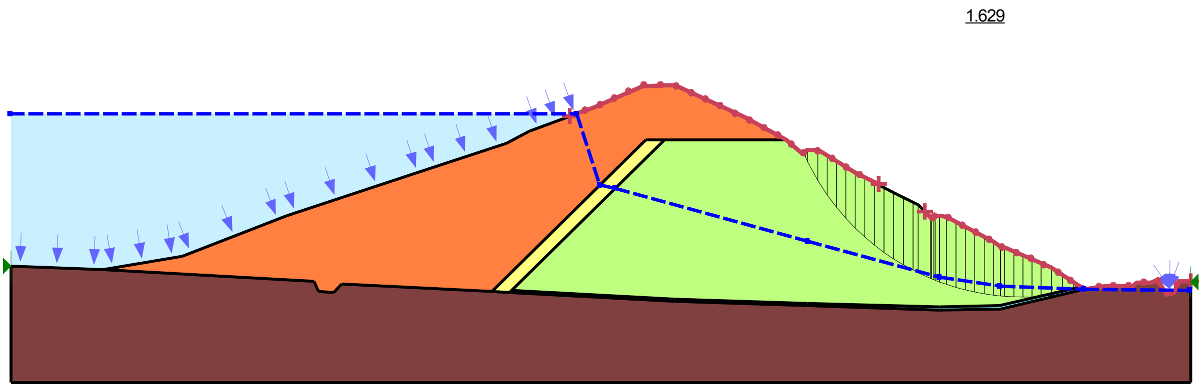
Scenario	Downstream Slope Stability		Upstream Slope Stability	
	Riverbed Section	Culvert Section	Riverbed Section	Culvert Section
1	1.672	1.606	2.228	2.248
2	1.326	1.289	2.228	2.248
3	1.629	1.539	2.228	2.248
4	1.175	1.193	2.228	2.248



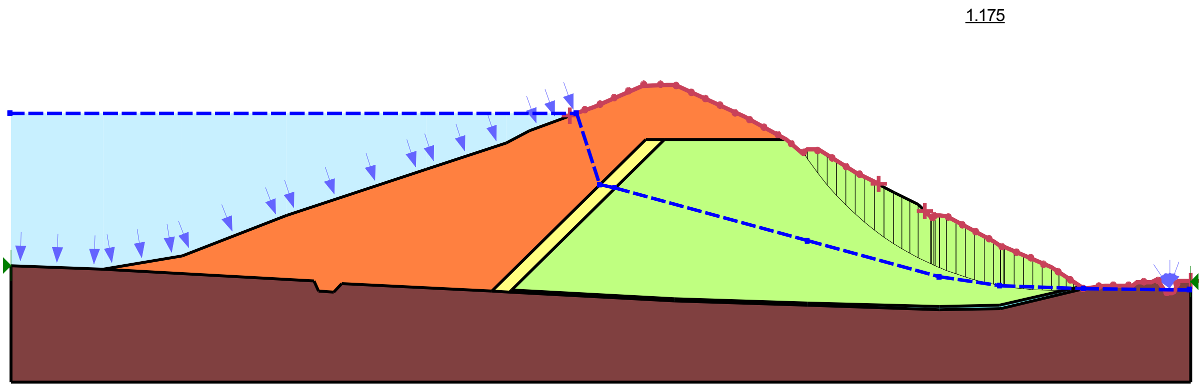
Riverbed Section Downstream Slope Stability: Scenario 1



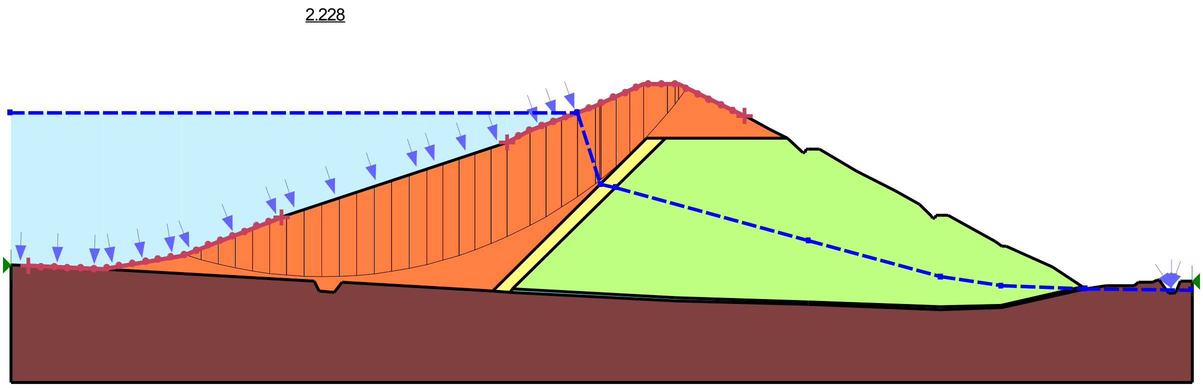
Riverbed Section Downstream Slope Stability: Scenario 2



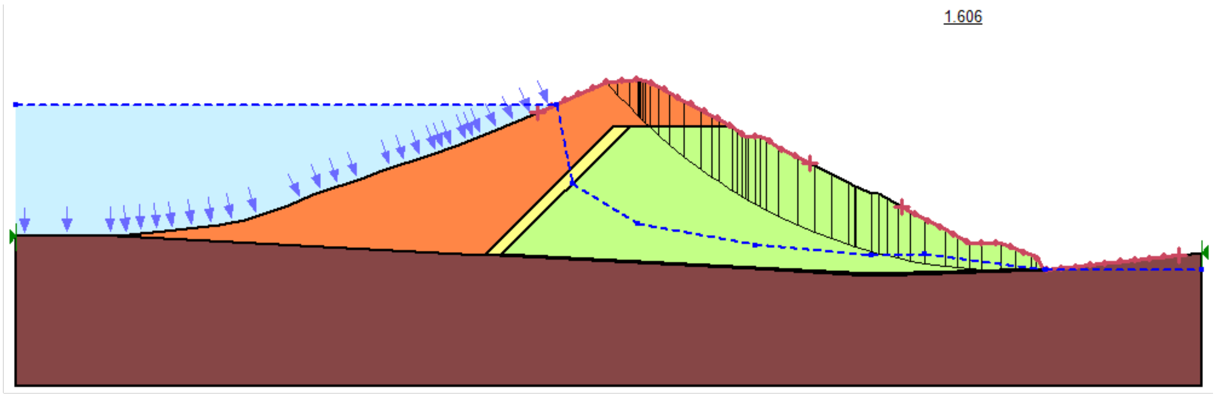
Riverbed Section Downstream Slope Stability: Scenario 3



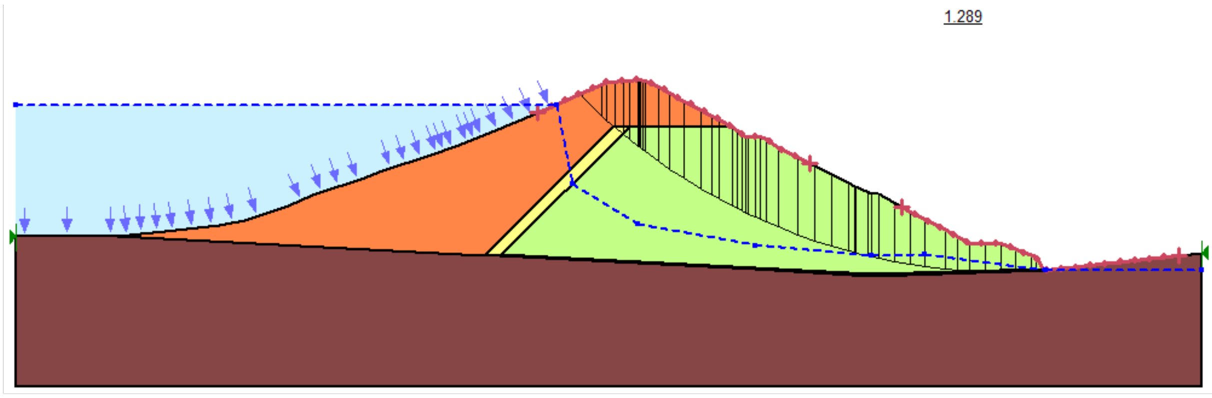
Riverbed Section Downstream Slope Stability: Scenario 4



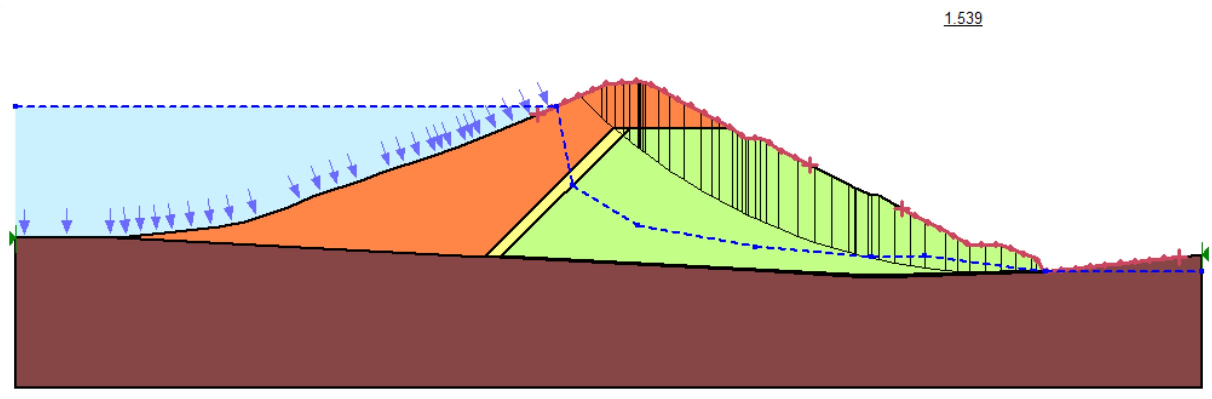
Riverbed Section Upstream Slope Stability



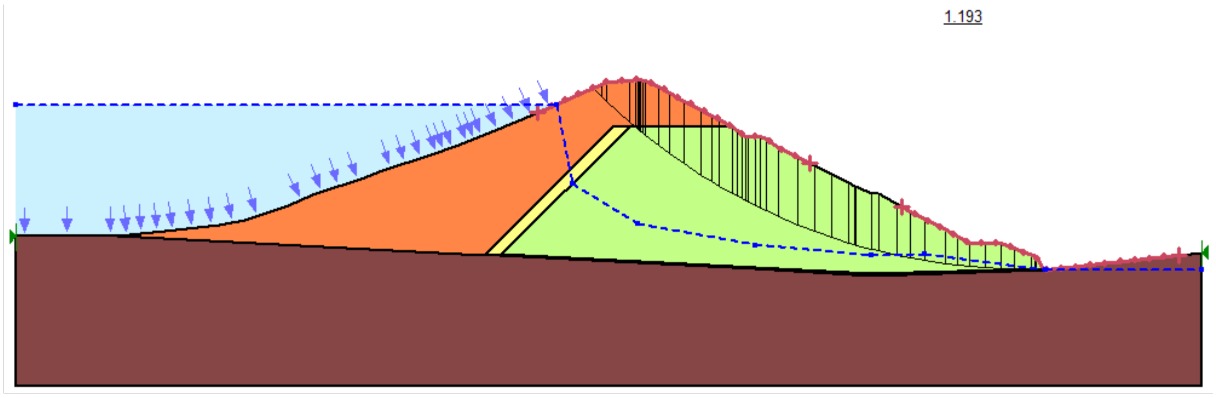
Culver Section Downstream Slope Stability: Scenario 1



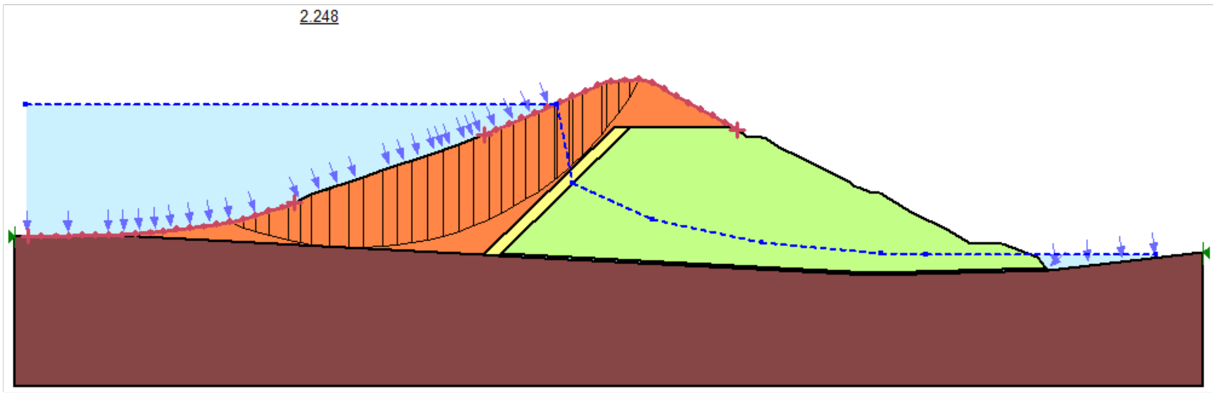
Culvert Section Downstream Slope Stability: Scenario 2



Culvert Section Downstream Slope Stability: Scenario 3



Culvert Section Downstream Slope Stability: Scenario 4



Culvert Section Upstream Slope Stability

APPENDIX J – DWA FORMS

water & sanitation

Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

185 Francis Baard (previously Schoeman) Street, Pretoria, 0002 Private Bag X313, Pretoria, 0001
Tel: (012) 336-7500 , Fax: (012) 336-8674, www.dwa.gov.za

Ms A Engelbrecht
EngebreghtA@dwa.gov.za
(012) 336-8421
12/2/X201/68 vol 2

SAPPI Manufacturing (Pty) Ltd
SAPPI Kraft
Ngodwana Mill
Private Bag X1001
NGODWANA
1209

ATTENTION: MS M ACKERMANN (email: Mia.Ackermann@sappi.com)

Madam

SIXTH DAM SAFETY EVALUATION REPORT OF A DAM WITH A SAFETY RISK REQUIRED IN TERMS OF REGULATIONS 35 AND 36 OF THE REGULATIONS REGARDING SAFETY OF DAMS PUBLISHED IN GOVERNMENT NOTICE R. 139 OF 24 FEBRUARY 2012: NGODWANA DAM SITUATED ON THE FARM GROOTGELUK 477 JT, DISTRICT OF NELSPRUIT

(a) In terms of Regulation 35(1)(a) of the above Regulations, the owner of a Category III dam must have a dam safety evaluation of the dam carried out at his/her expense when requested to do so, and within the period specified by the Director-General.

(b) In terms of Regulation 35(1)(c) of the above Regulations, the owner of a Category III dam must have further dam safety evaluations carried out at intervals of not more that 5 years.

(c) In terms of Regulation 36(1)(a) and (b) of the above Regulations a dam safety evaluation of a Category III dam, must be carried out by an approved professional person assisted by a professional team, to identify any actual or potential shortcomings in the condition of the dam or in the quality and adequacy of the procedures followed for the maintenance, operation and monitoring of behaviour that might endanger human lives, damage of property, or have an adverse impact on resource quality.

(d) In terms of Regulation 35(3) of the above Regulations, your company is hereby instructed to have a report on the dam safety evaluation compiled by the approved professional person assisted by a professional team, who must be approved beforehand by this Department, and the report must be submitted to the Dam Safety Regulation (Office) not later than **28 September 2018**.

The following documentation are available on the Dam Safety Regulation (Office) website at http://www.dws.gov.za/DSO/ for your reference and submission:

(i) The above Government Notice, containing the requirements of dam safety evaluations.
(ii) Application forms (DW699E and DW698E) to be completed by a professional person and a professional team to notify the Dam Safety Regulation (Office) of the task to be undertaken.
(iii) A list of consulting engineers. *It is suggested that you obtain at least three quotations from consulting engineers on the cost involved for execution of the said evaluation and thereafter select the most suitable quotation.*

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In order to keep the cost of the **sixth** dam safety evaluation as low as possible, your company is advised to gather the basic information required for the evaluation yourself, so that it is available to the approved professional person. The following information, if available, is required:

- Planning reports
- Geological reports
- Material investigation reports
- Design reports and calculations
- Design drawings
- Specifications and contract documents
- Construction reports and photographs taken during construction
- "As built" drawings
- Operation and maintenance manuals
- Previous evaluation and monitoring reports
- Articles on the dam published in technical magazines

It is also recommended that your company makes the necessary arrangements in advance to ensure accessibility to all important points on the structure. Any obstructions such as trees and shrubs that will hamper the evaluation, must be removed beforehand. Trees on earthfill embankments with a trunk diameter of 50 millimetres or larger should, however, be left in place in order to allow the approved professional person to evaluate their effect and advise your company on any required action.

Although the evaluation must be executed by an approved professional person, it will be necessary for you or senior members of your staff to participate actively in the evaluation. The approved professional person must evaluate the adequacy and quality of operation and maintenance procedures for the dam and therefore you or the staff concerned must be present to answer questions and to furnish additional information.

Yours faithfully

HMGroenewald

Ms H M Groenewald
Assistant Director: Dam Safety Regulation
Date: 2018-03-19

Enclosure: 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/ X201/68 ***WARMS Dam ID:***

Water management area 3 ***Dam Status:*** REG ***Drainage Nr:*** X21H

Name of dam NGODWANA DAM

Latitude 25 35 0 ***Longitude*** 30 40 24 ***Lat sec:*** 0.00

Town nearest: NELSPRUIT ***Long sec:*** 24.00

Distance from town (km) 40 ***WMA*** Inkomati-Usuthu

Name of farm GROOTGELUK 477 JT

Magisterial District NELSPRUIT

Province: MPUMALANGA ***Water Management Region:*** MPUMALANGA

Date of completion 1982

Raising or Alteration Date

River NGODWANA

Wall type EARTHFILL

Wall height (m) 44

Crest length (m) 450

Spillway OGEE

Capacity (1000 cub. m) 10400

Surface area of water (ha) 87 ***Catchment area (sq km)*** 229

Purpose MUNICIPAL USE & INDUSTRIAL USE

Owner

SAPPI MANUFACTURING (PTY) LTD.

PRIVATE BAG X1001

NGODWANA

1209

Tel no. (013)734-6111

Cell no. 082 876 7685

Email / Fax (013)734 6450

Person in Control (if not the same as the owner)

MR

THOM B.

Tel no. (013) 734-6210

Cell no.

Email / Fax

Designer

BGA LUND & PARTNER

Contractor

PETER FABER

Registration date: 1986/10/09 ***Status*** Dam Registered as a Dam with a Safety Risk

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

Classification date: 1986/12/02 ***Date Last DSE*** 2009/08/27

Date Completion Report: ***Number Last DSE:*** 5