NKANGALA DISTRICT MUNICIPALITY

RAISING OF DULLSTROOM DORP SE DAM
PRELIMINARY DESIGN REPORT

FEBRUARY 2014

Version 1.3

DOCUMENT PREPARED BY:

SCIP Engineering Group (Pty) Ltd.
N4 Business Park
Smokey Mountain
Witbank
1035

Tel 013-656-2660
Fax 013-656-3325

DOCUMENT CONTROL SHEET

Report No.	
Title Sub Title Project No,	***Dullstroom Dorp se Dam : Preliminary Design Report*** **2115/12**

Rev No:	Date of Issue	Originator		Checked		Approved		Description
		Initials	Signature	Initials	Signature	Initials	Signature	
1.3	February 2013	CRHC						

ACRONYMS, DEFINITIONS AND ABBREVIATIONS

Dam definitions	
Capacity	Stored volume at FSL
Max Height	Difference between NOC and lowest downstream / external toe level
CP	Change Point for level survey
DSO	Dam Safety Office
DWA	Department of Water Affairs
EIA	Environmental Impact Assessment
ELU	Existing Lawful Use
EMP	Environmental Management Plan
FSL	Full Supply Level
g	Acceleration due to gravity – (approximately 9.81m/s^2)
GA	General Authorisation
HTH	Commercial chlorine granules
MDEDET	Mpumalanga Department of Economic Development , Environment and Tourism
lugeon	A seepage flow rate of 1 litre/second/m hole at a pressure of 1 bar
NWA	National Water Act (Act No. 36 of 1998)
NOC	Non-overspill Crest – nominal top of dam embankment or wall
PCD	Pollution Control Dam
SANCOLD	South African National Commission on Large Dams
w/c	water / cement ratio by weight of grout mix

CONTENTS

TABLE OF FIGURES

TABLE OF TABLES

1 TERMS OF REFERENCE

The Nkangala District Municipality appointed *SCIP Engineering Group* in January 2013 to investigate the feasibility of raising the current Dullstroom Dorp se Dam by a nominal 3m.

2 BACKGROUND

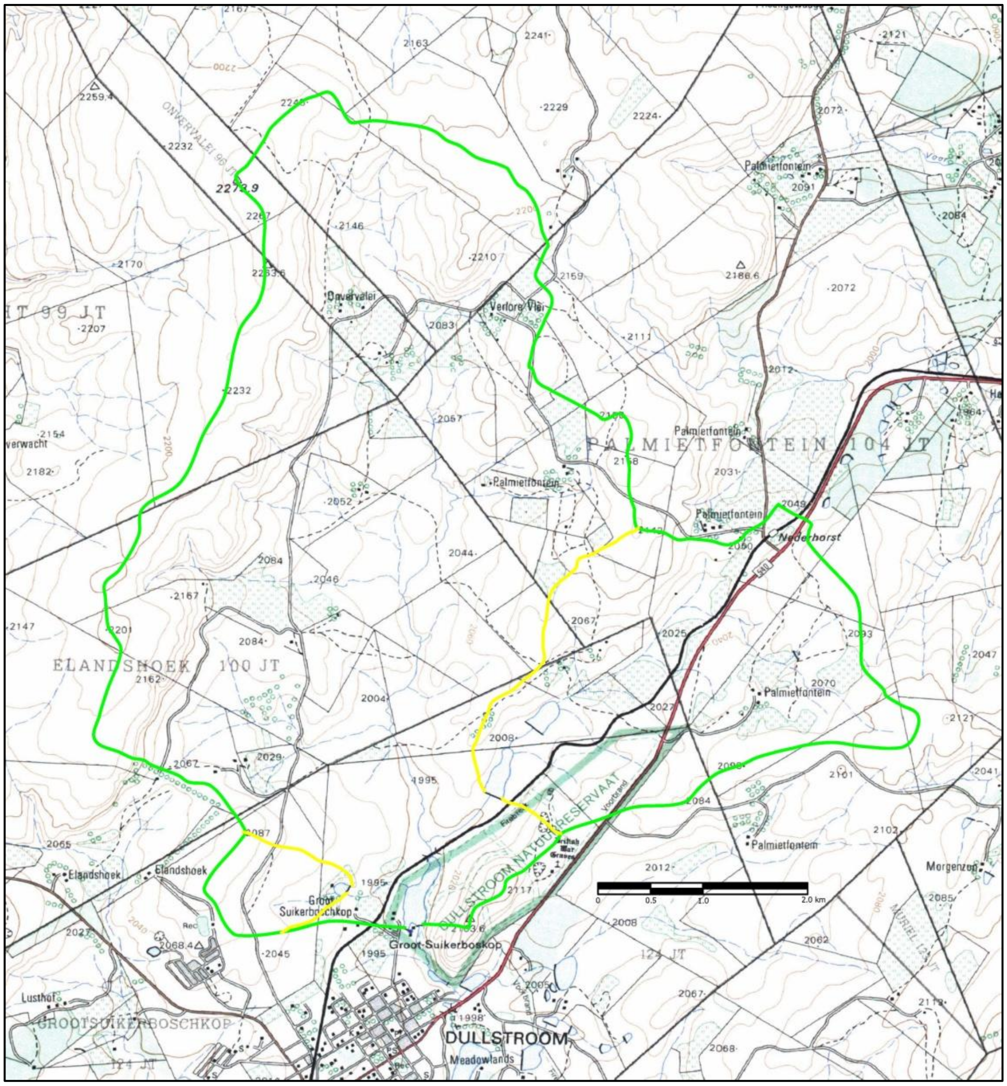
The Dam is municipal owned and used for provision of water for the town of Dullstroom. It is also used for recreational activities with fly-fishing being a favorite among locals and visitors. The left bank the dam is bordered by the Suikerbosch Kop Nature Reserve and a caravan park is located on the right bank of the dam. The Dullstroom / Leydenburg rail line passes just north of the current dam reservoir.

Figure 2-1 : Dullstroom Dorp se Dam : looking from left abutment to right bank caravan park

3 CATCHMENT

The Dullstroom dorp se dam is situated on the Crocodile river in the tertiary catchment X21A [(1)] . The catchment is a rural catchment covered largely by Highveld type grass (85%), with some bush and forest (5%) and cultivated land (10%). The mean annual precipitation in the area of the catchment is 839mm/year[2].

Figure 3-1 : Dullstroom dorp se dam catchment (from 1:50 000 mapping)



The catchment area is approximately 33.2[1] km^2 and drains predominantly from north to south. The longer of two rivers feeding the dam is approximately 8.2km long. The average slope along the longest river is 1.8% with slopes varying from 10% upstream in the mountain areas to 2.7% on the hilly areas and 1.3% towards the lower reaches.

4 DAM YIELD HYDROLOGY

4.1 WRC Information

4.1.1 Quaternary Catchment details

Quaternary Catchment : X21A

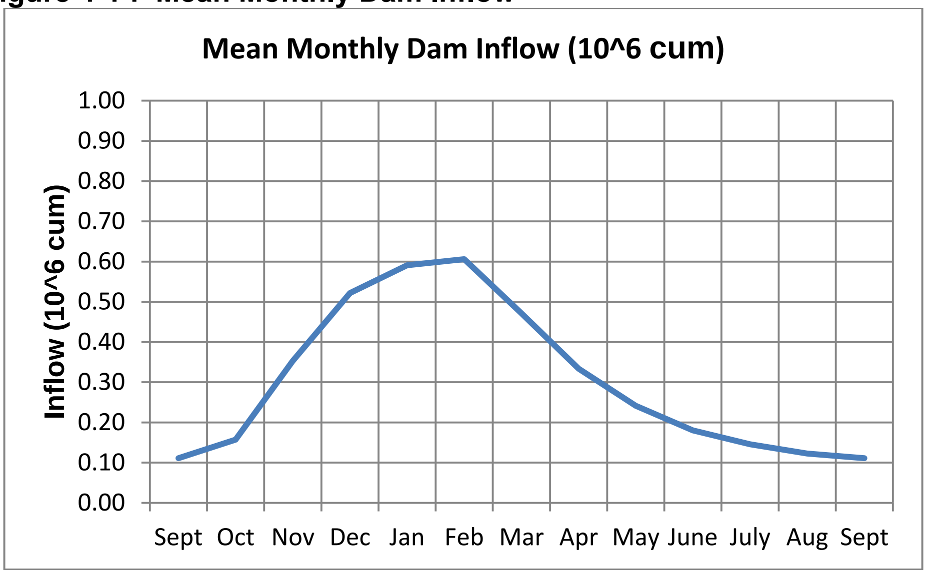
Gross area (km^2)	265
Net area (km^2)	265
Forest area (km^2)	2
Irrigated area (km^2)	11.6
Evaporation Zone	5A
Mean Annual Evaporation (mm)	1400
Rain Zone	X2A
Mean Annual Precipitation (mm)	839
Mean Annual Run-off (mm)	146
MAP-MAR response	6
Net MAR (10^6m^3)	38.6
Gross MAR (10^6m^3)	38.6
Coefficient of variation	0.468
Hydro Zone	B
Dullstroom Dam – current volume (estimated)	$0.274 \times 10^6m^3$
Dullstroom Dam – proposed volume	$0.602 \times 10^6m^3$
Catchment area (km^2)	33.2
MAP at Station 0554175	761
Variation in MAP	-9.3%
MAR from Rainfall Runoff response chart (mm)	108
Variation in MAR	-26.0%
Gross quaternary MAR for dam catchment (10^6m^3)	3.84
Current Dam as % MAR	7.1%
Raised Dam as % MAR	15.7%
Storage Draft frequency curves - % MAR	
Gross yield 1:100 yr failure	44%
1:50 yr failure	47%
1:20 yr failure	52%
Storage Draft frequency curves - (10^6m^3) of current dam	
Gross yield 1:100 yr failure	1.69
1:50 yr failure	1.80
1:20 yr failure	1.99

4.1.2 Mean Monthly River Discharge

Table 4-1 : Mean Monthly River Discharge

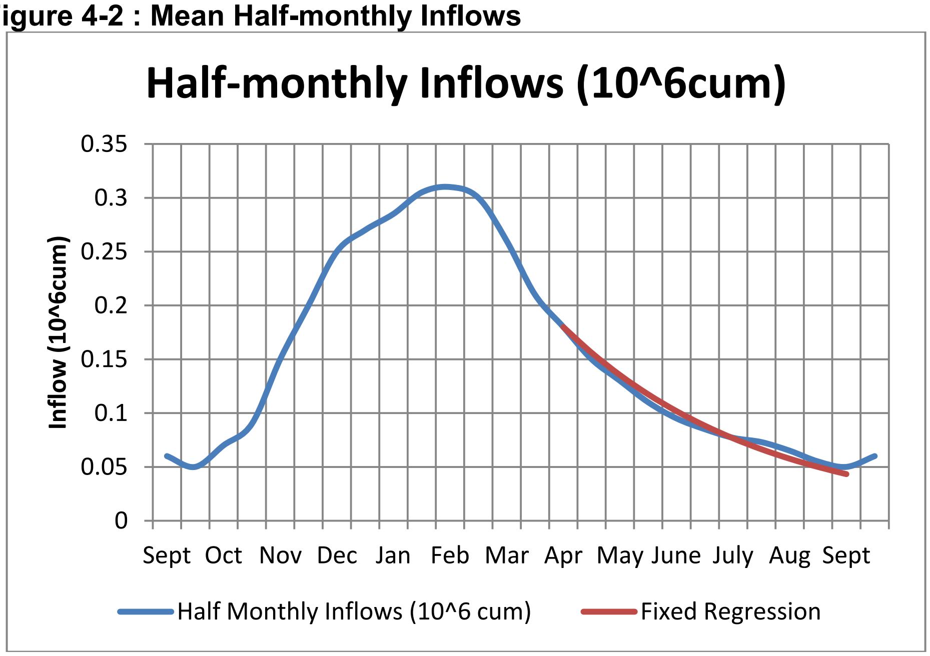
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept
% MAR	4.1	9.2	13.6	15.4	15.8	12.3	8.7	6.3	4.7	3.8	3.2	2.9
$10^6 m^3$	0.16	0.35	0.52	0.59	0.61	0.47	0.33	0.24	0.18	0.15	0.12	0.11

Figure 4-1 : Mean Monthly Dam Inflow



Fixed regression can be used for spreadsheet computation, where the start April inflow is entered and the rest of the hydrological year is calculated on a fixed decline factor. In this case, the factor is approximately 87%.

Figure 4-2 : Mean Half-monthly Inflows

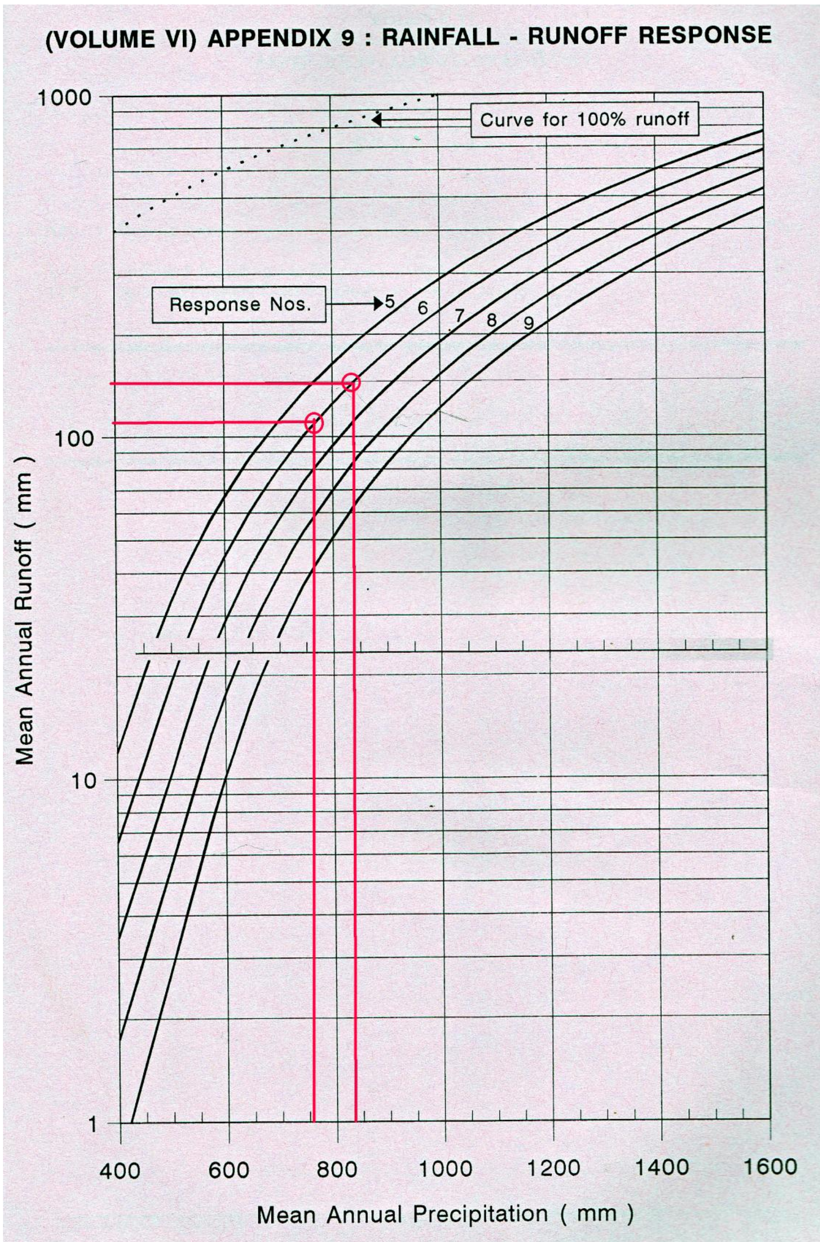


4.1.3 Rainfall – Runoff Response

Figure 4-3 : Catchment Rainfall / Runoff Response Chart[2]Figure 4-3 shows the variation in run-off with rainfall for the various response numbers, which are allocated to catchments. The Dullstroom Dam response number is 6, and the figure below shows derivation of a MAR of 108mm for a MAP of 761mm .The total catchment MAP is 146mm for a MAP of 839mm. If the local MAP can be shown to be different from the assumed MAP of 761, a revised MAR can be applied.

However, the difference in dam yield is not significant, as discussed later, because the reservoir volume is small, compared to the MAR.

Figure 4-3 : Catchment Rainfall / Runoff Response Chart[2]



4.1.4 Synthesised Long Term Records

A 70 year record was synthesised for 1920 to 1979 by the WRC researchers[2]. The worst 5 years are plotted below, including the 1981-1982 seasons,

Figure 4-4 : Monthly Inflows of Worst 5 of 70 years – April flow

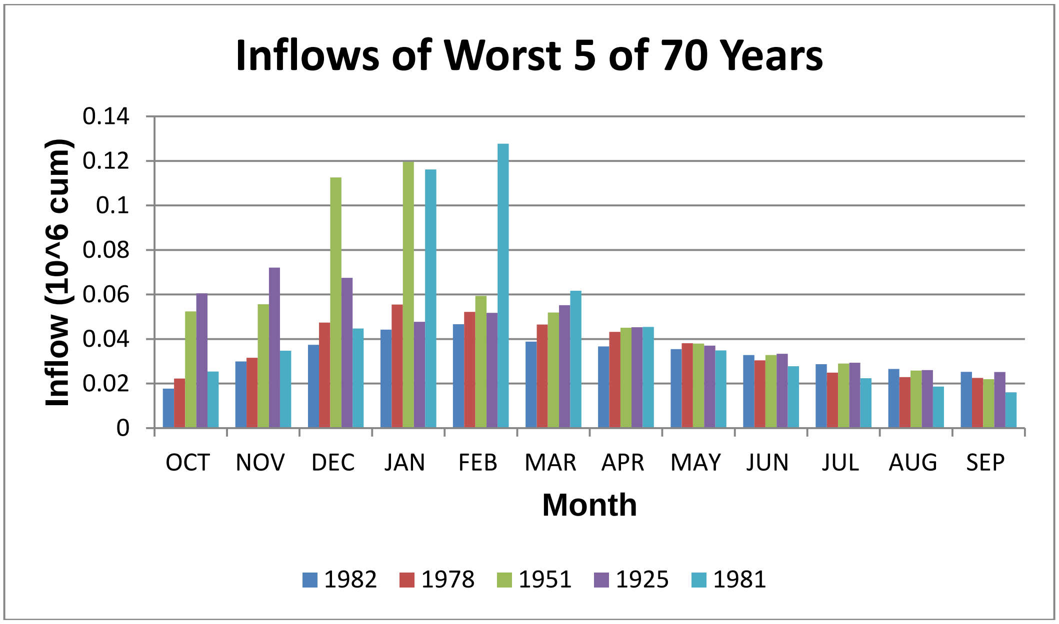


Figure 4-5 : Monthly Inflows for Worst April to September

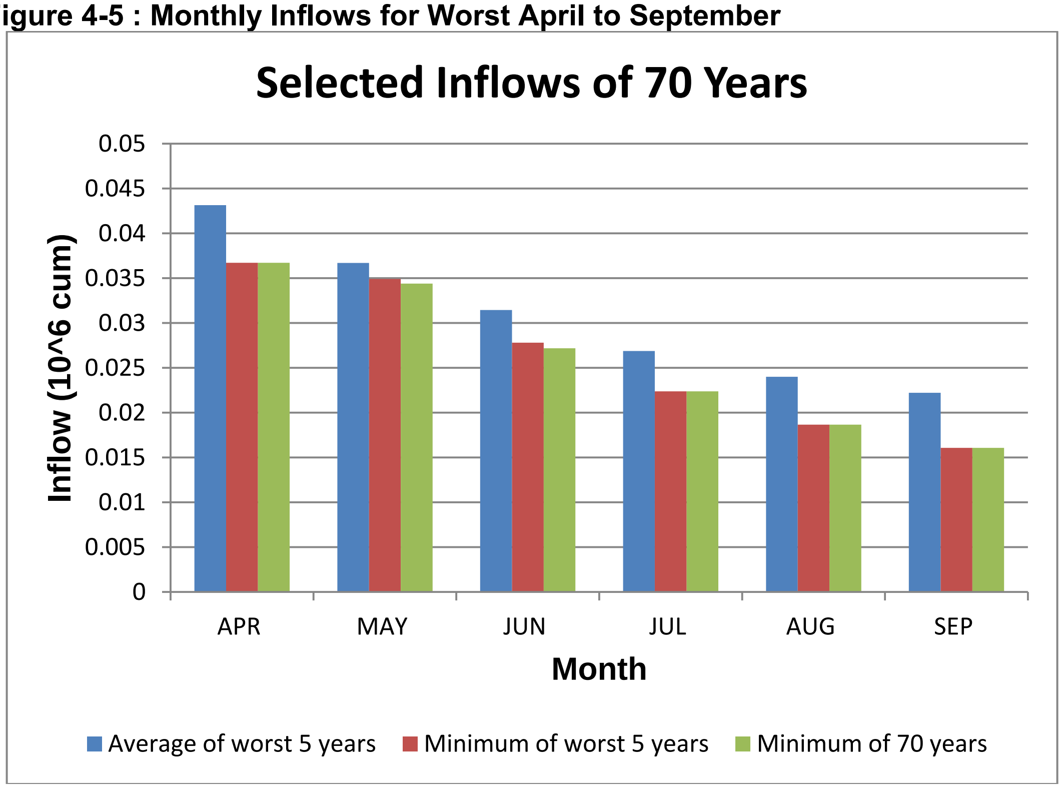
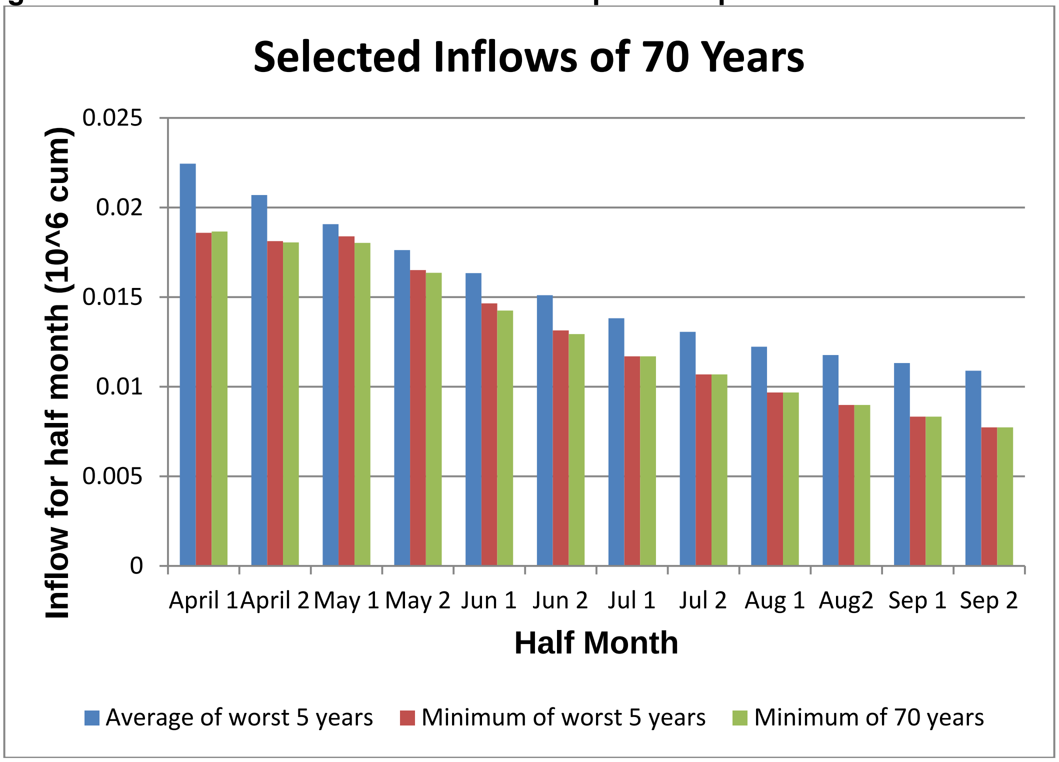


Figure 4-6 : Inflows for Worst Half Months April to September



Assurance of Supply

Definition

The reliability at which a specific quantity of water can be provided.

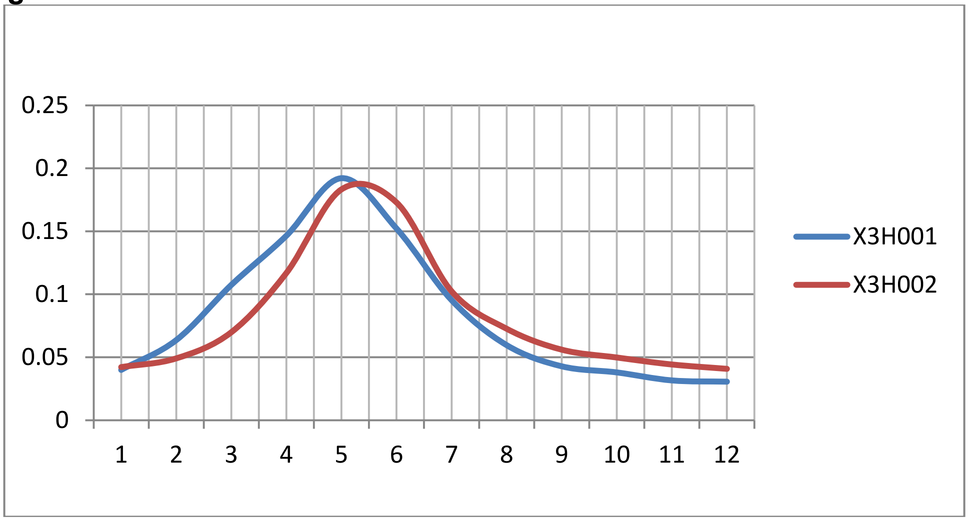
Description

The reliability at which a specific quantity of water can be provided, usually expressed either as a percentage or as a risk. For example 98% reliability means that, over a long period of time, the specified quantity of water can be supplied for 98% of the time, and less for the remaining 2%. Alternatively, this situation may be described as a '1 in 50 year risk of failure' meaning that, on average, the specified quantity of water will fail to be provided in 1 in 50 years, or 2% of the time.

4.1.5 Naturalised Stream flows

The closest gauging weirs were on the X3H catchment, in a higher rainfall area, but of similar catchment area and characteristics. The average monthly flows are plotted below

Figure 4-7 : X3H Catchments Naturalised Streamflow



4.1.6 Effective Yield of Raised Dam

The raised dam volume is approximately 15.7% of the estimated MAR, which is unusually small. The conventional methods of estimating the safe yield of a dam is to simulate the , volume of the dam being drawn down over a number of low-inflow years to augment the received inflow. With such a small dam, in terms of the %MAR, this method is not applicable because the critical period is reduced to the tail end of the natural flow regression, where the volume of the dam is effectively depleted over two or three months.

4.1.7 Simple Mathematical Management Model

A mathematical model has been developed using a two week / half month time interval. This is informed by the dam volume and surface area / depth relationships, the inflow regression, water demand and evaporation profiles, as well as the management rules governing the application of water use restrictions. This can be used for estimating safe yields as well as the management of water use in dry periods.

The model covers the period from April to the following year December. The rationale of this is that the yield – albeit diminished by operating rules factors – has to be provided from April, through a poor wet season to the start of the following wet season.

The observed behaviour of the dam can be checked against estimated inflows and draw-offs and predictions can be made on future behaviour, **always presuming the catchment conditions do not change significantly.**

It is essential that development in the catchment is regulated to ensure the retention of run-off in actual or simulated wetland bodies. Rapid release of run-off, such as from urban development, will reduce the regression flow volumes and directly adversely affect the dam yield.

(VOLUME VI) APPENDIX 4.2 : NATURALISED STREAMFLOW (contd.)

GAUGE X3H001 (Units - million cubic metres)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	TOTAL
1965	1.58	4.56	4.82	6.92	19.65	8.49	2.53	2.17	1.94	1.70	1.50	1.40	57.26
1966	2.49	3.18	11.51	13.34	21.36	11.81	18.43	9.85	4.63	3.75	3.12	2.42	105.89
1967	2.69	4.86	6.06	5.15	3.93	5.86	5.48	4.21	3.11	2.55	2.05	1.74	47.69
1968	1.84	5.04	8.97	8.04	8.14	13.67	8.44	3.98	2.90	2.42	2.60	2.48#	68.52
1969	4.89	5.45	8.53	5.14	4.64	3.69	2.66	2.43	1.90	1.77	1.56	1.28	43.94
1970	1.69	3.59#	3.69	13.49	9.15	4.49	3.41	2.50	2.09	1.74	1.36	1.54	48.74
1971	1.85	3.87	9.00	23.06	18.41	21.02#	12.28#	5.38	3.79	2.89	2.40	1.99	105.94
1972	2.86	3.76	4.20	5.04	5.50	6.70	11.64	6.60	3.80	3.11	2.72	4.09	60.02
1973	7.26	7.37	21.17	20.63	24.04	16.47	11.14	8.34	5.48	4.86	3.88	3.31	133.95
1974	3.50	5.50	9.12	9.88	19.32	14.95	7.97	5.84	4.74	3.94	3.29	2.92	90.97
1975	2.96	2.85	9.91	21.80	29.31	24.12	15.56	6.54	4.28	3.69	3.07	2.40	126.49
1976	2.71	5.15	6.82	12.12	30.99	17.46	9.11	4.67	3.50	3.02	2.66	3.31	101.52
1977	3.84	3.77	13.49	38.59	22.78	19.13	8.63	5.03	3.84	3.39	2.81	2.59	127.89
1978	3.08	5.24	7.31	5.38	7.20	8.69#	4.74	3.41	2.69	2.47	2.43	2.29	54.93
1979	2.29	7.03	8.56	8.28	15.33	14.66	5.76	3.65	3.02	2.68	2.24	2.22	75.72
1980	2.58	9.62	10.87	14.82	26.38	15.87	6.73	5.03	3.82	3.20	2.68	2.90	104.50
1981	3.57	4.31	5.20	8.28	7.04	4.99	4.30	3.99	2.96	2.53	2.22	1.91	51.30
1982	1.37	2.99	3.26	4.81	3.76	4.08	3.99	3.47	2.67	2.21	2.12	1.78	36.51
1983	2.43	7.18	9.46	10.21	7.27	8.36	7.57	3.73	2.64	4.14	3.78	3.17	69.94
1984	3.64	6.04	9.69	9.97	20.67	9.94	4.37	3.37	2.91	2.60	2.25	2.26	77.71
1985	4.02	5.18	8.06	11.99#	15.35	10.13	7.70	6.42	4.03	3.36	2.82	2.57	81.63
1986	2.76	3.52	5.10	7.12	6.02	10.04	7.92	3.89	3.08	3.00	2.89	3.96	59.30
1987	4.80	4.32	10.63	10.48	22.99	22.48	7.87	5.08	3.80	3.91	3.36	3.94	103.66
1988	5.91	5.61	7.76	8.03	22.31	15.76	4.98	4.55	4.47	3.80	3.47	2.79	89.44
1989	3.65	7.72	12.55	11.63#	14.53	12.83	8.26	5.63	3.84	3.44	.33	.15	84.56
MEAN	3.21	5.11	8.63	11.77	15.44	12.23	7.66	4.79	3.44	3.05	2.54	2.46	80.32

GAUGE X3H002 (Units - million cubic metres)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	TOTAL
1963	.58#	.93#	.94	1.39#	.88	.82	.74	.44	.25	.13	.12	.08	7.30
1964	.19	.56	.81	1.92#	1.02	.80	.69	.55	.46	.42	.38	.37	8.17
1965	.34	.72#	.59	.68	3.45#	2.16	.54	.38	.45	.39	.39	.34	10.43
1966	.39	.38	1.26#	.69	3.58#	2.13	2.43	2.01	.77	.70	.49	.49	15.32
1967	.54	.54	.73	.68	.56	.61	.59	.59	.56	.49	.45	.37	6.71
1968	.38	.55	1.20#	.66	.92	2.93	2.72	.78	.63	.57	.51	.42	12.27
1969	.60	.69	1.02	.90	.70	.55	.50	.50	.41	.42	.41	.36	7.06
1970	.51#	.46#	.48	1.43#	.69	.71	.49	.56	.51	.47	.39	.48	7.18
1971	.39	.50	.77	4.46#	4.57#	5.43#	2.32#	1.50	1.10	.92	.71	.63	23.30
1972	.70	.78#	.70	.87	.80	1.07#	1.30	1.07	.80	.68	.57	.64	9.98
1973	.77	.77	4.07#	4.89#	6.04#	3.88	1.95	1.52	1.14	1.03	.88	.78	27.72
1974	.75	.83	.95	1.03	1.12	1.34	1.23	1.04	.88	.76	.64	.61	11.18
1975	.60	.64	1.00	3.65	6.93#	5.09	4.05	2.09	1.50	1.26	1.13	.90	28.84
1976	.91	.98	1.02	1.18	5.07	5.77	2.94	1.80	1.32	1.14	1.03	1.02	24.18
1977	.98	.95	1.54#	6.94#	6.30#	4.87	3.07	1.86	1.46	1.35	1.14	1.00	31.46
1978	.96	1.02	1.08	1.08	1.00	1.46#	1.00	.98	.81	.84	.80	.77	11.80
1979	.78	.88	.91	.98	1.66#	1.83	1.32	1.11	.87	.76	.72	.64	12.46
1980	.60	1.08#	.97	4.26#	6.03	3.22	1.52	1.17	.90	.82	.78	.74	22.09
1981	.69	.68	.73	.79	.85	.98#	.90	.87	.74	.67	.62	.52	9.04
1982	.51	.59	.61	.65	.58	.64	.59	.62	.62	.54	.62	.45	7.02
1983	.47	.87#	.83	.93	.94	.95	.83	.87	.66	.62	.68	.68	9.33
1984	.63	.75	.96	1.27	5.57#	4.76	1.22	.96	.75	.75	.61	.57	18.80
1985	.53	.59	.74	.85	.89	1.27#	1.21	1.11	.95	.77	.60	.53	10.04
1986	.54	.54	.60	.68	.92#	1.17#	.95	.74	.59	.55	.50	.56	8.34
1987	.56	.53	.89	1.01	5.68#	7.58#	2.43	1.34	1.09	.89	.75	.77	23.52
1988	.85	.77	.82	.84	2.97	3.23	1.13	.91	.87	.74	.67	.58	14.38
1989	.65	.71	.92	.94	1.93	2.12	1.20	1.09	.84	.79	.68	.61	12.48
MEAN	.61	.71	1.01	1.69	2.65	2.50	1.48	1.05	.81	.72	.64	.59	14.46

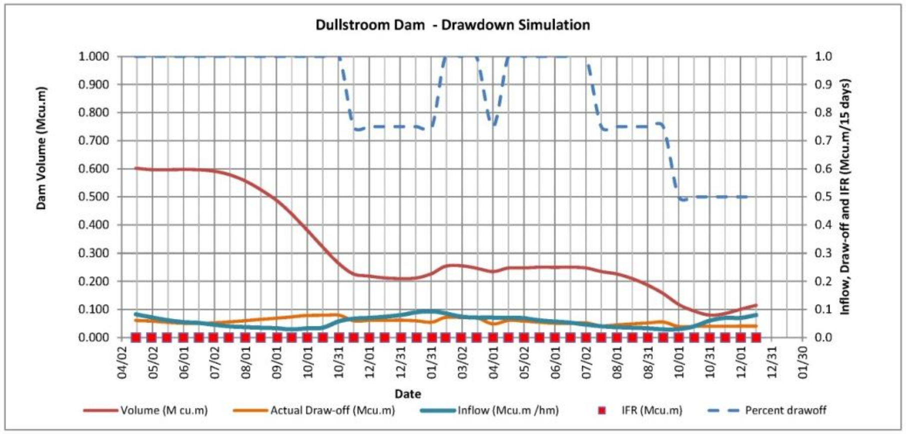
Original streamflow data have been patched

Figure 4-8 : Simple Dam Yield Model (spreadsheet – excludes part of model period) –

Yellow squares indicate input areas - white areas are fixed or calculated - however, you can change anything you like - so keep a "clean" copy of this and work on a copy. If you change the dam storage characteristics, you need to synthesise the Volume/Area/Depth curve and insert in i5 i6 and i7

SCENARIO 3 - Worst two years on record, yield 4.5Ml/day, with restrictions, Dam capacity 602 x 10^3 cum.

DULLSTROOM DAM	Depth @FSL	10.5 m		Inflow first 15 days of April	0.09 Mm3	Draw off Restrictions (apply also to EFR)	Volume <	Release
	FSL	103 mamsl	0.602 Mm3	Evaporation	1400 mm/a		40.0%	75.0%
	LDL	96 mamsl	0.038 Mm3	Yield 1.64 Mcum/a	4.5 Ml/day		25.0%	50.0%
	LD depth	3.5 m		Water level =	92.5-6.66855*V+15.750456*V^0.5+2.5801892*V^0.25			
hm=half month	Bed lvl	92.5 mamsl		Surface Area =	.2055811*v+0.0839372*v^0.5-0.000686*v^0.25			
	FSL Area	0.082 sq.km		Volume =	0.0089335*d+.00004178d^4			
	Start Level 103	Mid-term Volume 0.21		End Volume 0.14	8mo drawoff 1.21	9-20mo drawoff 1.56		
Total 8month inflow	Year 1	0.807 Mm3			IFR 8 mo 0	IFR 9-20mo 0		



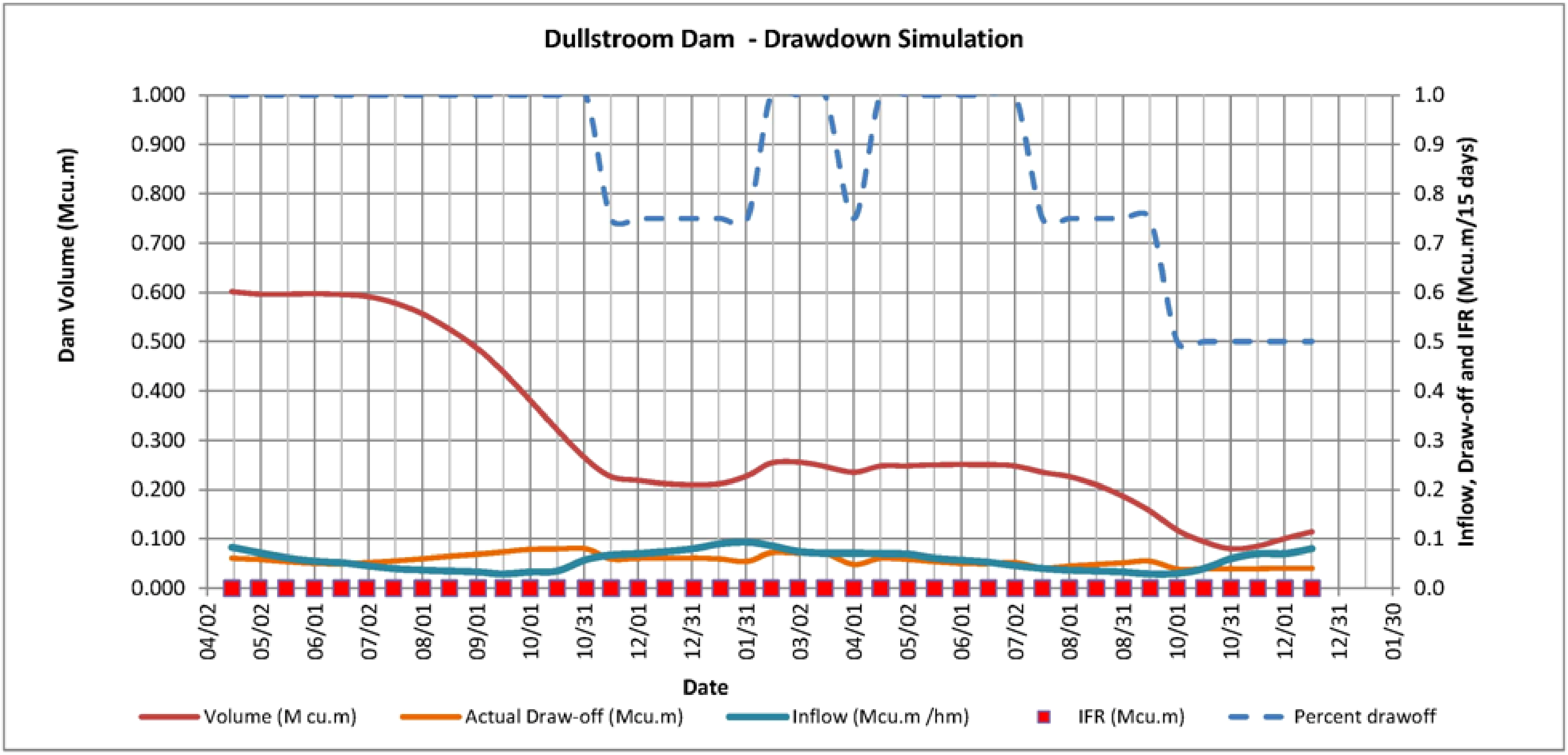
NOTE : hm = half month

Period	Nominal EFR	Specified Release	Inflow Year 1	Inflow Year 2
	Mm3/hm	Mm3/hm	Mm3/hm	Mm3/hm
January				0.08
				0.09
February				0.093
				0.086
March				0.075
				0.071
April			0.09	0.07
			0.08	0.07
May			0.07	0.07
			0.06	0.06
June			0.06	0.06
			0.05	0.05
July			0.05	0.05
			0.04	0.04
August			0.04	0.04
			0.04	0.04
September			0.03	0.03
			0.03	0.029
October			0.03	0.03
			0.04	0.04
November			0.058	0.06
			0.067	0.07
December			0.07	0.07
			0.074	0.08
Total	0.00	0.00	0.97	1.45
Default low flow regression hm>hm				87%

Erase what you don't want - don't use the space bar.

Date	04/16	05/01	05/16	06/01	06/16	07/01	07/16	08/01	08/16	09/01	09/16	10/01	10/16	11/01	11/15	12/01	12/16	01/01	01/16	01/31
% Full	100.00%	99.17%	99.17%	99.27%	99.02%	98.38%	96.28%	92.51%	87.28%	80.73%	72.89%	63.39%	53.44%	43.65%	37.60%	36.35%	35.27%	34.85%	35.26%	37.90%
Level (masl)	103.00	102.98	102.98	102.96	102.95	102.93	102.88	102.77	102.61	102.39	102.11	101.71	101.23	100.67	100.26	100.17	100.09	100.06	100.09	100.28
depth	10.50	10.48	10.48	10.46	10.45	10.43	10.38	10.27	10.11	9.89	9.61	9.21	8.73	8.17	7.76	7.67	7.59	7.56	7.59	7.78
Volume (M cu.m)	0.602	0.597	0.597	0.60	0.60	0.59	0.58	0.56	0.53	0.49	0.44	0.38	0.32	0.26	0.23	0.22	0.21	0.21	0.21	0.23
Area (sq km)	0.188	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19
Evaporation (mm/hm)	45.3	41.7	36.1	31.1	30.4	33.2	37.5	43.9	50.9	58.0	65.8	75.0	76.4	77.1	77.1	80.6	80.6	82.4	76.4	65.1
Evaporation (Mcu.m)	0.009	0.008	0.007	0.006	0.006	0.006	0.007	0.008	0.010	0.011	0.013	0.014	0.015	0.015	0.015	0.016	0.016	0.016	0.015	0.013
Nominal Draw-off (Mcu.m)	0.061	0.059	0.054	0.051	0.050	0.052	0.055	0.060	0.065	0.069	0.073	0.078	0.079	0.080	0.080	0.081	0.081	0.082	0.079	0.073
Actual Draw-off (Mcu.m)	0.061	0.059	0.054	0.051	0.050	0.052	0.055	0.060	0.065	0.069	0.073	0.078	0.079	0.080	0.060	0.061	0.061	0.062	0.059	0.055
Draw off restricted to	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	100.0%	75.0%	75.0%	75.0%	75.0%	75.0%	75.0%
IFR (Mcu.m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Inflow (Mcu.m /hm)	0.083	0.072	0.062	0.055	0.052	0.046	0.040	0.037	0.035	0.033	0.029	0.033	0.035	0.058	0.067	0.070	0.074	0.080	0.090	0.093
rainfall (mm)	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Rainfall (Mcu.m)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Net Volume change	0.01	0.01	0.00	0.00	0.00	-0.01	-0.02	-0.03	-0.04	-0.05	-0.06	-0.06	-0.06	-0.04	-0.01	-0.01	0.00	0.00	0.02	0.03
end volume	0.602	0.602	0.597	0.596	0.592	0.579	0.557	0.525	0.486	0.439	0.381	0.321	0.263	0.226	0.219	0.212	0.210	0.212	0.228	0.254
end level	102.98	102.98	102.96	102.95	102.93	102.88	102.77	102.61	102.39	102.11	101.71	101.23	100.67	100.26	100.17	100.09	100.06	100.09	100.28	100.57

Figure 4-9 : Simple Dam Yield Model – Graphic Output



5 FLOOD HYDROLOGY

The dam is currently a category II dam with high hazard rating. The raised dam will become a category III dam with high hazard rating. The RDF should therefore be taken as the 1:200 year flood and the Safety Evaluation Flood as the RMF+Δ. There are numerous small dams in the upper catchment, which have been ignored for the purposes of this study essentially because they would not influence the SEF or RDF significantly. The individual estimated volumes are small and – even if a dam break did occur upstream, the effect on Dullstroom dorp se dam would be minor.

5.1 Regional Maximum Flood (RMF) and Safety Evaluation Flood (SEF)

The upper Crocodile river catchment has a Kovacks K-value of 5, which gives the RMF as 595 m^3/s and the SEF for a K+Δ value of 5.2 as 740 m^3/s.

5.2 Recommended Design Flood

The flood hydrology calculations were based on various methods packaged in software called Utility Programs for Drainage developed by the University of Pretoria and Sinotech Consulting Engineers.

Table 5-1 : Comparison of flood peak estimates by various methods.

Method	Recurrence intervals		
	1:50	1:100	1:200
Rational	235 m^3/s	300 m^3/s	
Alternative Rational	230 m^3/s	280 m^3/s	315 m^3/s
Unit Hydrograph	200 m^3/s	270 m^3/s	
Standard Design Flood	235 m^3/s	290 m^3/s	355 m^3/s
Empirical Methods (Midgley & Pitman)	165 m^3/s	210 m^3/s	
Proportion RMF (RMF = 595 m^3/s)	245 m^3/s	310 m^3/s	380 m^3/s

The different flood prediction methods tabled above were used to arrive at a RDF of 350 m^3/s for the Dullstroom Dorp se Dam.

Data on flood peaks observed within the larger catchment is limited to those downstream of Kwena Dam at Montrose, with the highest flood peak in the past 54 years being 227 m^3/s. Taking the flood attenuation of Kwena Dam into consideration, these observed flood peaks were considered of no value in estimating flood peaks at Dullstroom.

5.3 Flood Volumes

Flood volumes are important as they assist in determining the shapes of the floo hydrographs. The shapes of the hydrographs were approximated by using the Unit-hydrograph flood prediction method.

The volumes were calculated to be as follows:

Table 5-2 : Flood Volumes

Flood	RDF	SEF
Calculated volume	2,0 Mm^3	4,2 Mm^3
Ratio to current dam estimated volume	≈ 10,0	≈ 21,0
Ratio to raised dam volume	≈ 3.3	≈ 7,0

6 SPILLWAY DISCHARGE AND FLOOD ROUTING

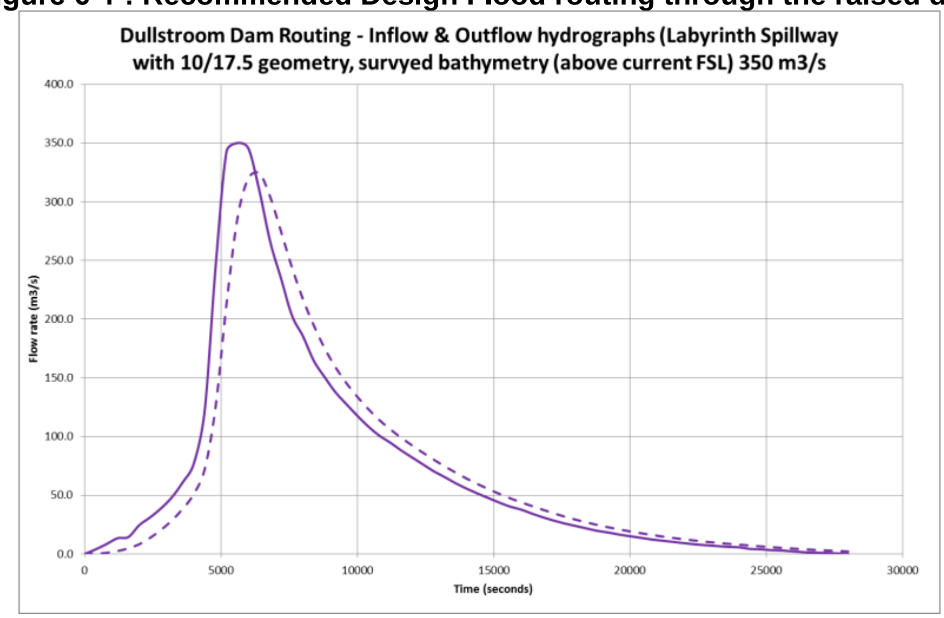
The proposed raised Dullstroom Dorp se Dam, with an estimated volume of 600,000 m^3 and surface area at full supply level of 9,7ha, provides only a minor degree of attenuation of flood peaks. The routing allowed for the calculated 1:200 (RDF) and RFM+Δ (SEF) flood events as inflow to the dam with outflow governed by the Labyrinth spillway characteristics. Initially, a mean coefficient of discharge (Cd) value of [$0.34 \times (2g)^{0.5}$] was adopted for the 5 cycle spillway with 10m wide approaches and 17.5m upstream to downstream face distances.

The routing calculation was based on the principle of level pool routing and adopting the bathymetry of the dam above the current Full Supply Level of 1985.200m. The bathymetry was calibrated from a contour survey that was completed during May 2013. The routing of both the RDF and the SEF through the dam was calculated and resulted in the following:

Table 6-1 : Flood attenuation

Flood	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Water Level Rise (m)	Attenuation (%)
RDF	350	324	1.4	7.3%
SEF	740	690	2.3	6.8%

Figure 6-1 : Recommended Design Flood routing through the raised dam.



Taking the following into consideration:

- The limitations of the unit-hydrograph method to describe the shape correctly of the RDF hydrograph for the specific catchment.
- The higher 1:200 year flow rate that the proportional RMF method yielded, i.e. 380 m^3/s compared to the adopted 350 m^3/s from the Standard Design Flood.

It is recommended that the un-routed RDF of 350 m^3/s is used as Recommended Design Discharge for sizing the spillway and not the routed RDF of 324 m^3/s.

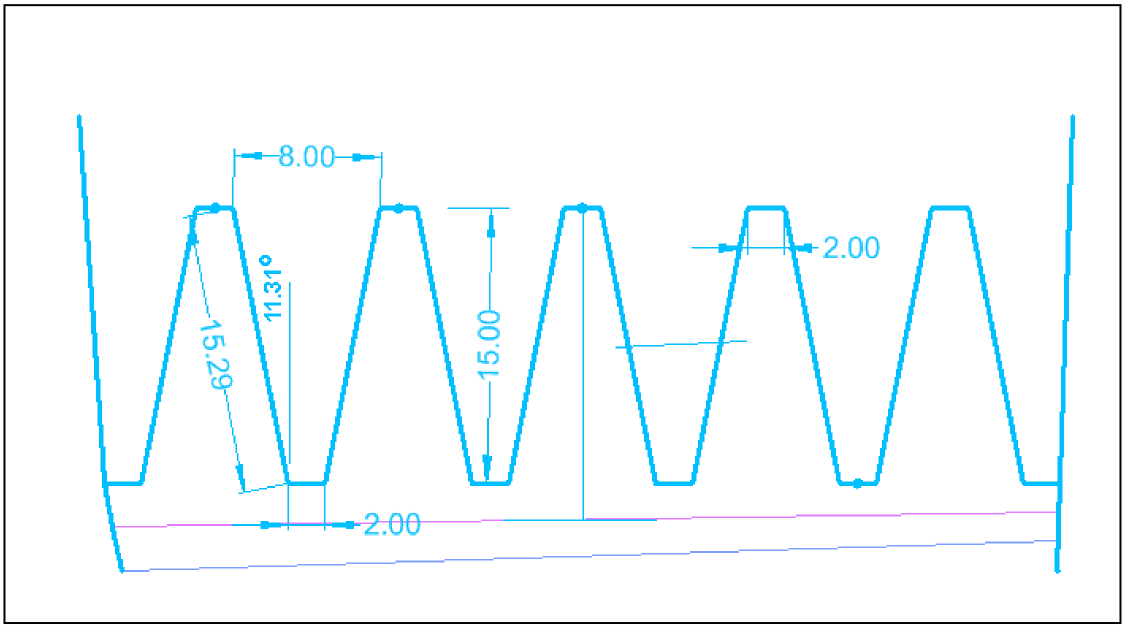
7 SPILLWAY DESIGN

7.1 Type Selection and Basic Dimensions

It was evident from the outset that the only suitable spillway design suitable was a labyrinth type, with uncontrolled discharge. This could provide the necessary discharge with least overspill depth, as dictated by the levels of buildings in the adjacent camping area and the railway bridge at the head of the reservoir area, and fit within the confines of the site and existing water treatment plant.

Two particular formats were compared, based on segments with 2m headwalls and 8m between headwalls, with 4 full bays and two side half-bays. The variations were the width, or effective upstream to downstream dimension, altering the angle of incidence of the long overspill sections. The narrower the angle, the more the flow tends to choke and the discharge is reduced. With an upstream / downstream width of 15m, the incidence angle is 11.31^0, which reduces to 9.73^0 for a 17.5m width. Spillway channel width remains the same at 52m net.

Figure 7-1 : Labyrinth spillway layout with 15m width, 2m headwalls and 8m bays.



Discharge coefficients vary with depth and angle of incidence, depth of overflow and width of bay. Calculations for the two options selected showed the following (Table 7-1) as illustrated in
Figure 7-2 and Figure 7-3 .

Figure 7-2 : Discharge over 15m wide Labyrinth Spillway option

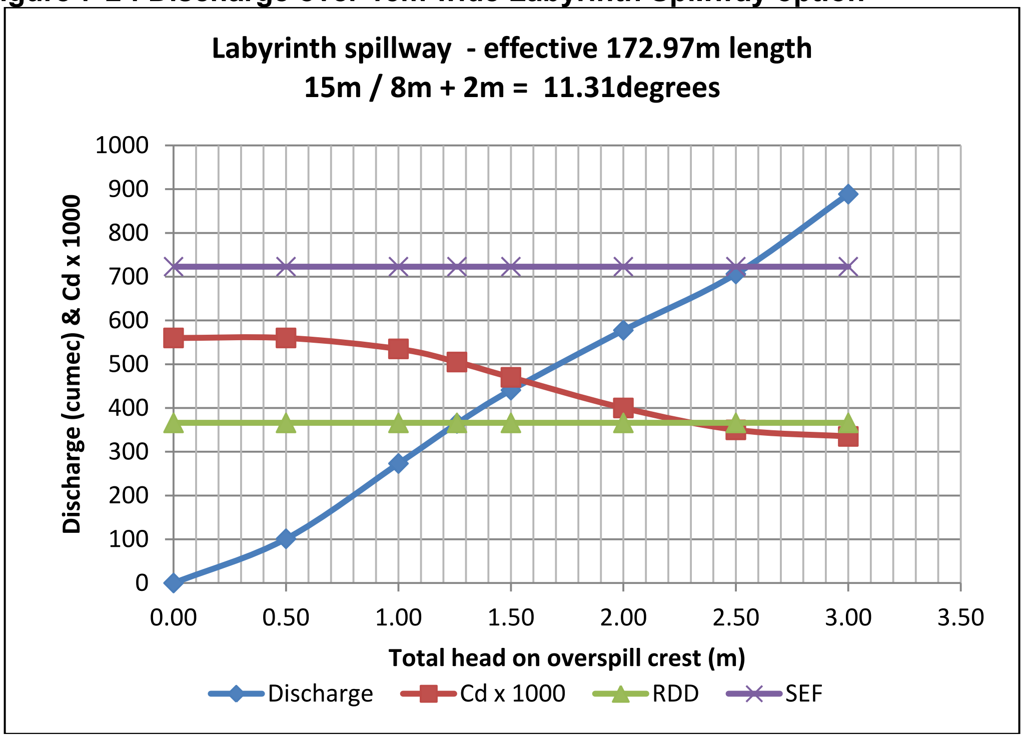


Figure 7-3 : Discharge over 17.5m wide Labyrinth Spillway option

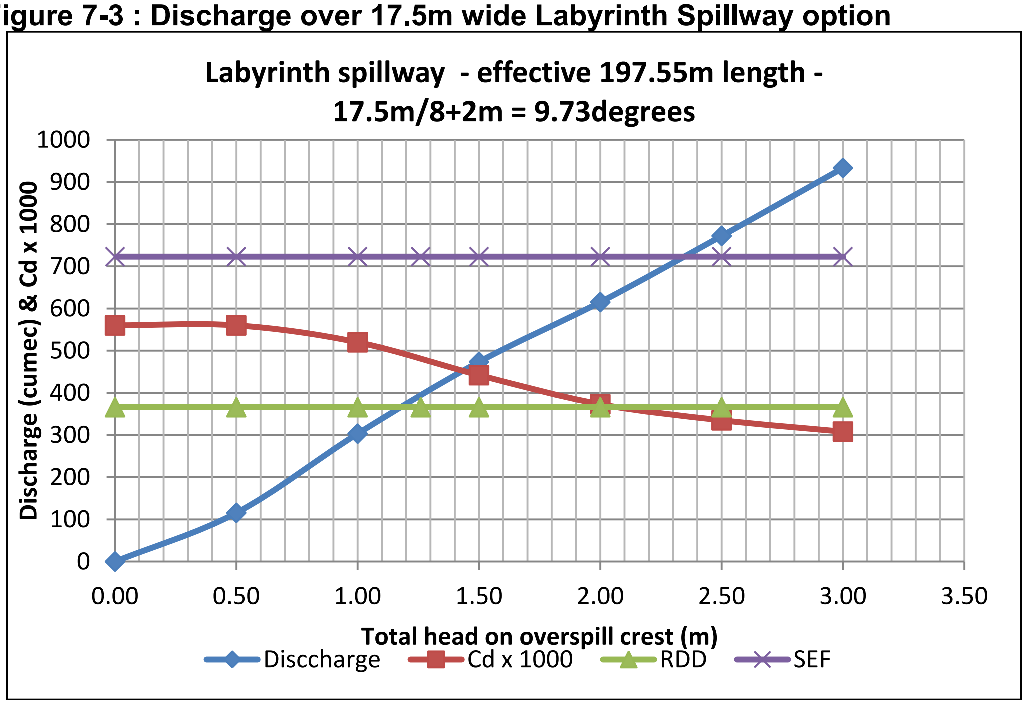


Figure 7-4 : Labyrinth Spillway Options – theoretical and simple calculation discharges

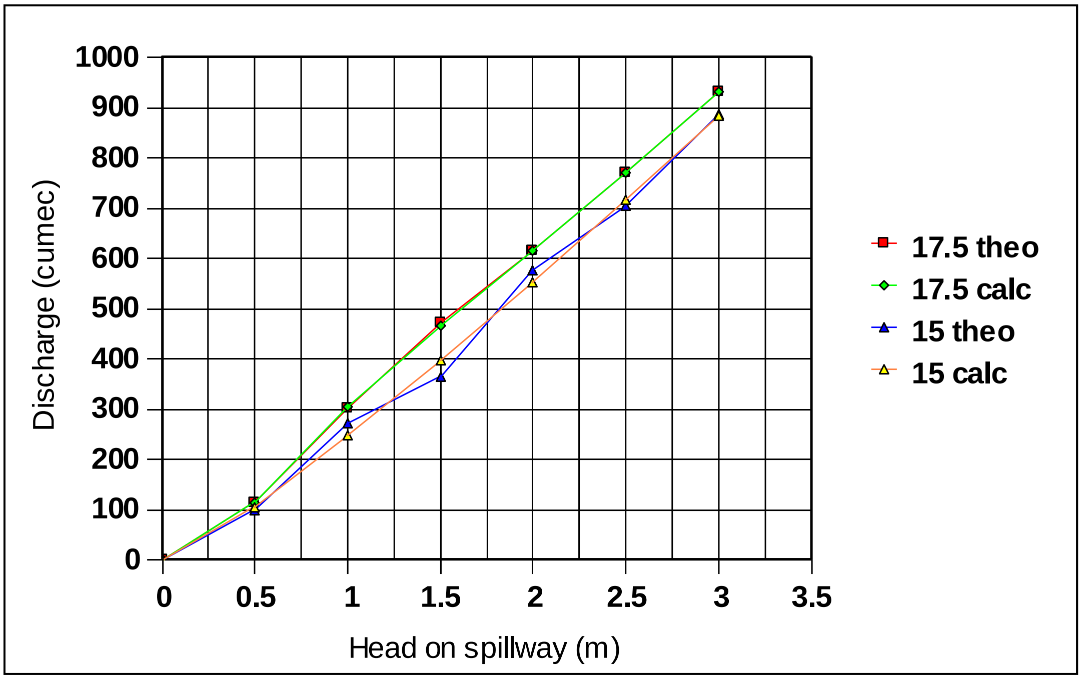


Table 7-1 : Labyrinth Spillway Dimensions and Discharges

Width	Angle	Effective Length	Overspill Depths	
			RDD	SEF
15.0 m	11.31^0	173.0 m	1.26 m	2.56 m
17.5 m	9.73^0	197.6 m	1.18 m	2.36 m

The conclusion from this is that the 17.5m wide design will allow a reduction in height of the embankment by 0.2 to 0.3m, the cost saving of which is offset by the additional labyrinth spillway concrete. The 17.5m width design is that selected for further design development.

The sections shown in Figure 7-6 and Figure 7-8 are somewhat idealistic. The spillway will require energy dissipation devices in the discharge chute as indicated in Figure 7-7. and will have to be tested in a hydraulic model for effective energy control.

However, the chute discharge level will have to be lower than the full supply level of the downstream dam, to avoid major disruptions of fauna and flora under low flood conditions.

Figure 7-5 : Stylized Spillway and Raised Embankment with Access

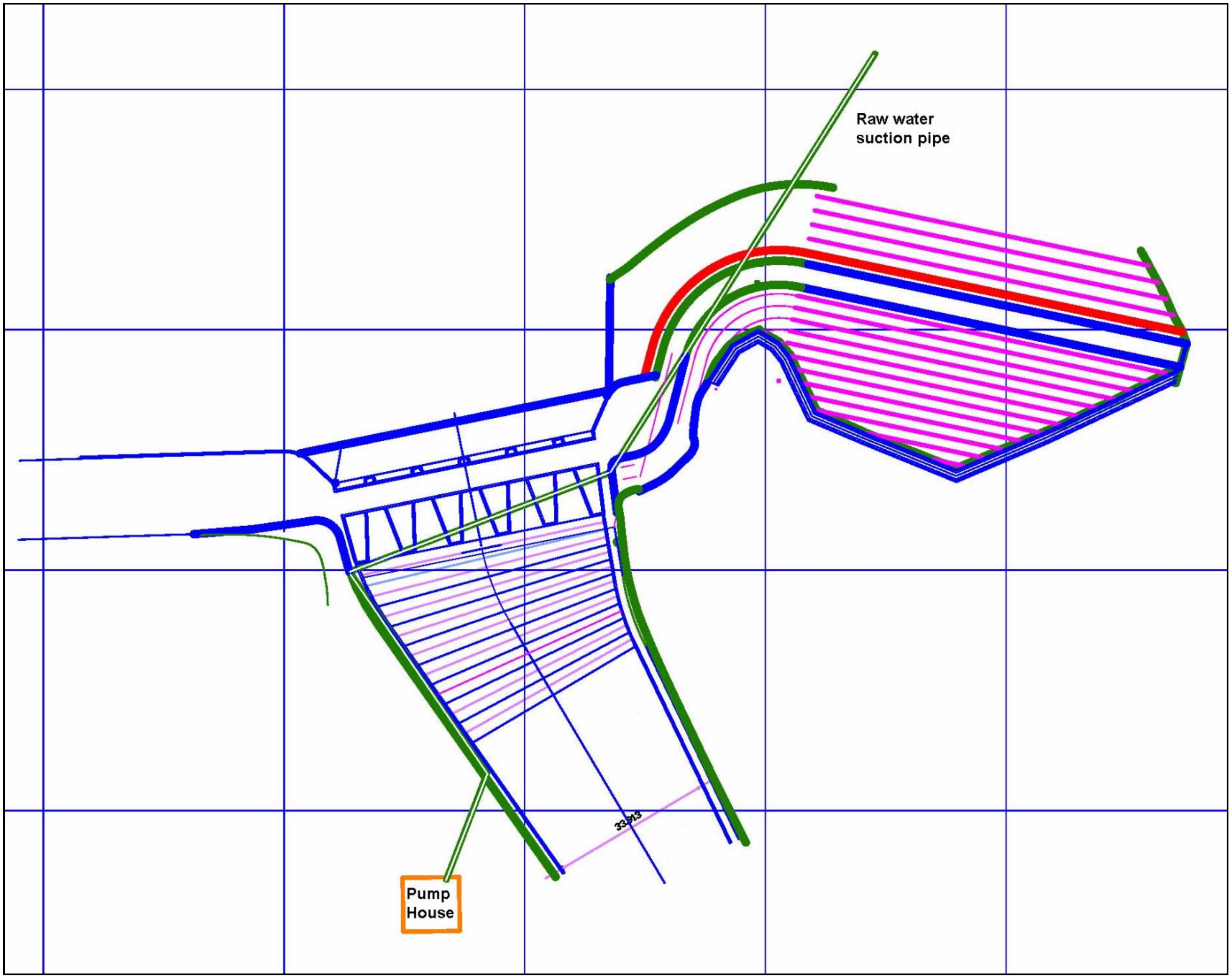


Figure 7-6 : Spillway Longitudinal Section with Ground Lines

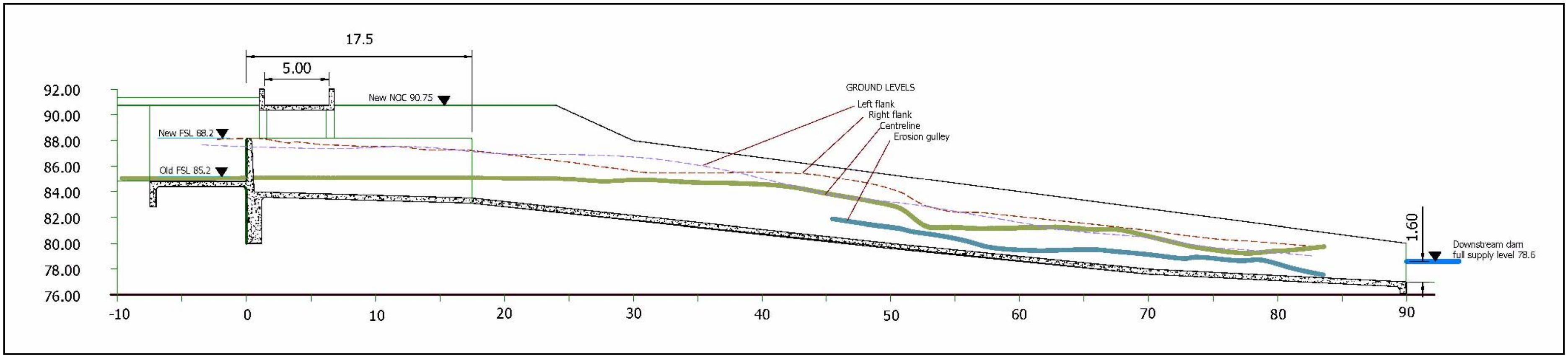


Figure 7-7 : Spillway Longitudinal Section with typical energy dissipater

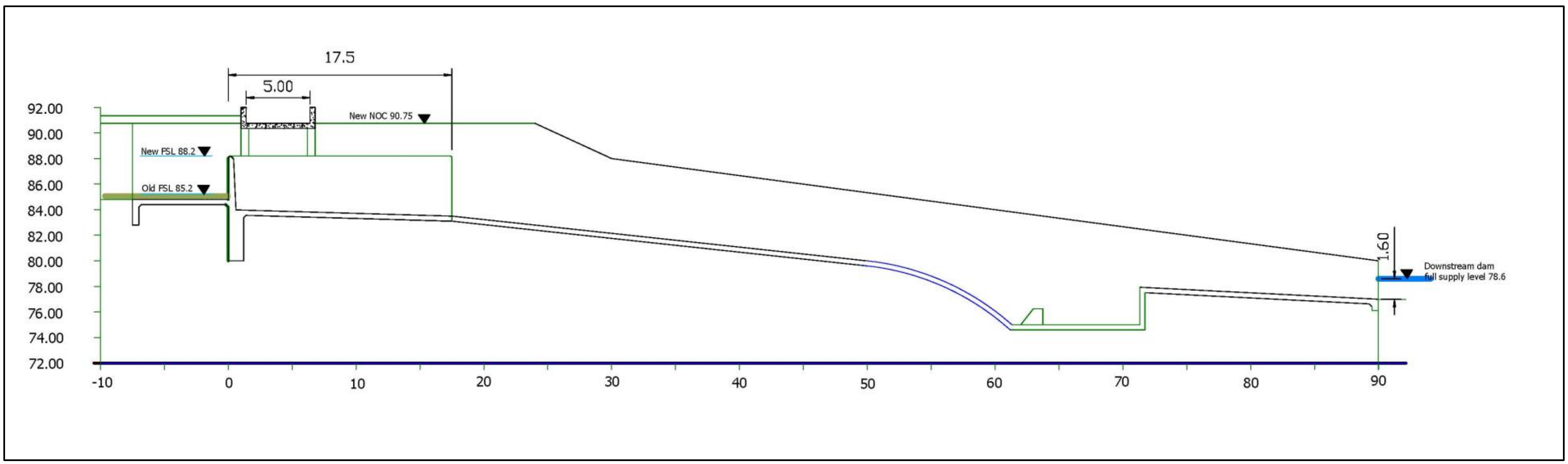
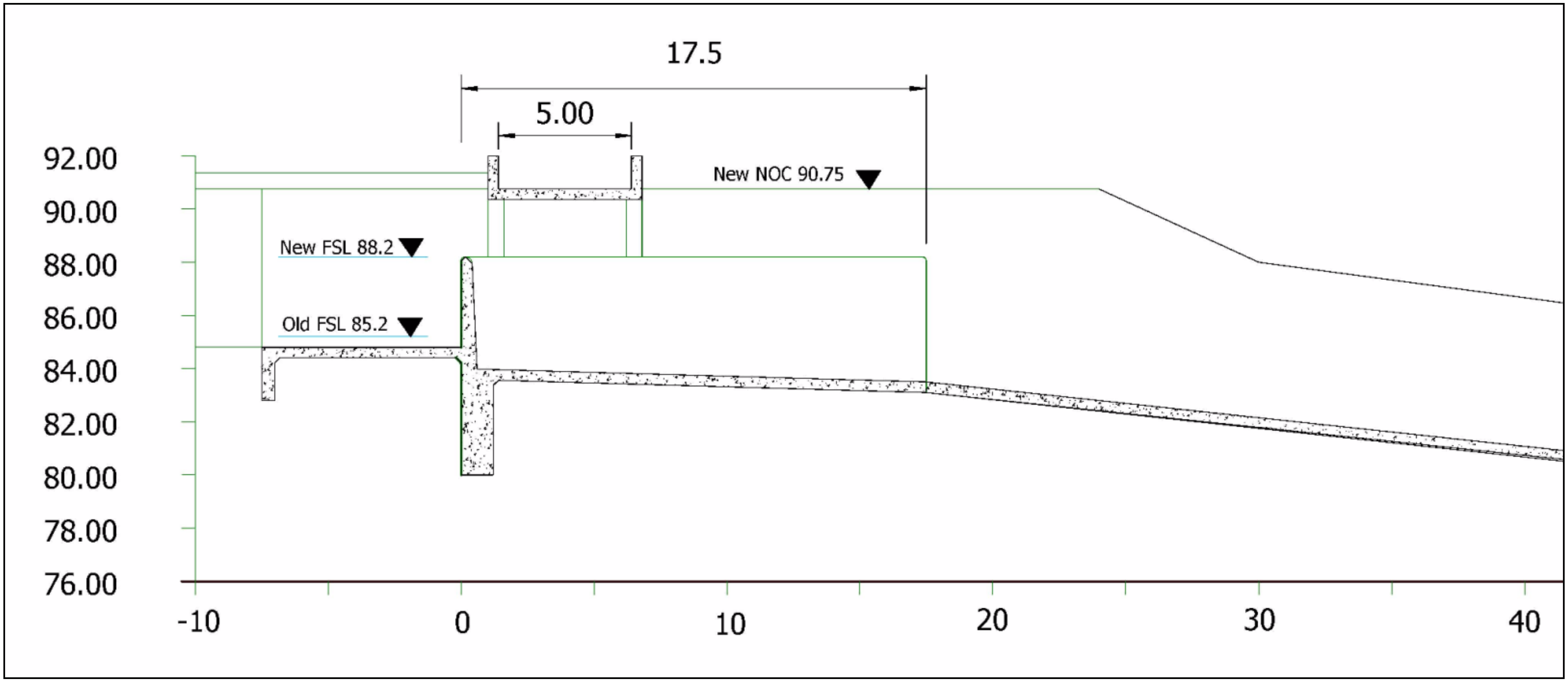


Figure 7-8 : Section on Spillway and Bridge

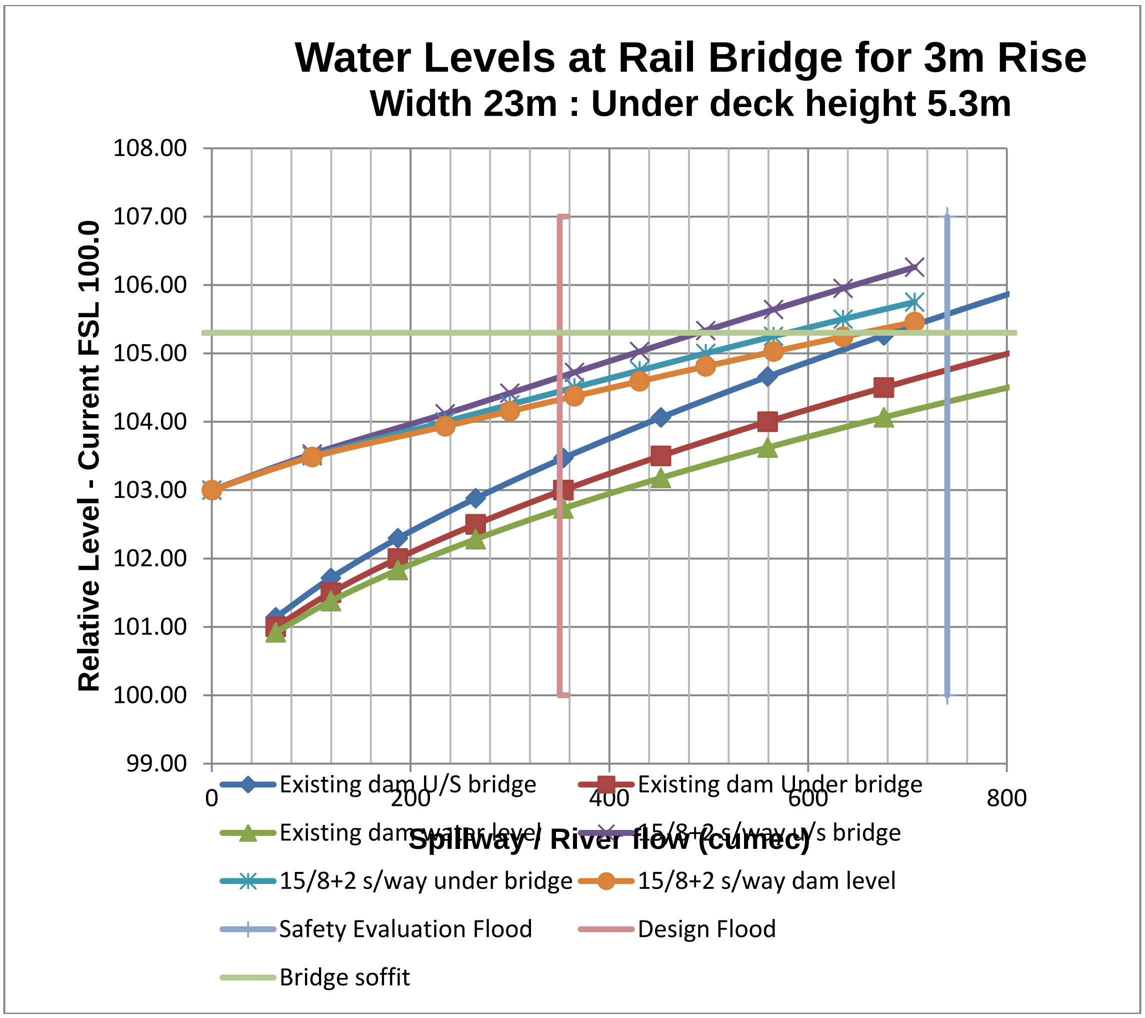


8 RAILWAY BRIDGE HYDRAULICS

The existing dam reservoir full supply level is the same as the stream low flow level at the bridge, with a very shallow stream flow depth. The effect of dam raising at the railway bridge will therefore be significant, raising the no-flow water level by the proposed 3.0m and creating a backwater under flood flow conditions.

These conditions are illustrated in Figure 8-1, which shows two families of curves illustrating the water levels upstream of the bridge, through the bridge structure, where the water is accelerated and therefore lower than upstream, and the dam water level downstream of the bridge. The design 1:200 flood will be safely accommodated but the safety evaluation flood will be just above the level of the bridge deck underside. The adoption of the 17.5m labyrinth would lower the SEF level by 0.2m. It is not considered necessary to increase the size of the spillway to accommodate the catastrophic flood below the bridge deck.

Figure 8-1 : Water Levels under Rail Bridge before and after dam raising



The railway service road crosses the watercourse above the rail bridge, through a drift, which is not passable at any significant river flow. There is also a small plank footbridge downstream of the rail crossing.

As would be expected, flow velocities under the bridge are reduced because of the deeper flow. At the estimated 1:200 year flood, the velocity reduces from 5.1m/s to 3.5m/s with the raised dam.

Raising the dam will require reconstruction of the rail crossing as follows :

- A service road bridge will be required with a minimum deck level of 0,5m above final full supply level. This should be upstream of the rail bridge and should be as slender as possible, to minimize impeding flood flows.

- The rail bridge embankment should be thickened up – the side slopes are too steep for permanent inundation and need to be at 2:1 slopes. The embankment should also be armoured for 30m on either side of the bridge aperture in the same way as the dam embankment is protected by riprap.

- The river bed should be cleared of any obstructions, such as large boulders. However, care should be taken not to disturb the bridge abutment footings. The footings should be assessed and – if necessary – additional protection provided against scour.

- Footpaths and styles over fences should be in place to permit a circular walk of the dam reservoir area, as is the case now.

Figure 8-2 : Rail bridge from upstream

Figure 8-3 : Rail bridge underneath showing steep embankment slopes

Figure 8-4 : Service road drift and upstream footbridge

9 DAM BREAK EVALUATION

A simple dam break study was conducted as the budget for the preliminary design did not allow for any detail modeling for this.

The aim of the study was to determine the increase in risk of the failure of the raised dam, i.e. water levels were calculated for the base scenario, i.e. the original dam failing and the second scenarios the raised dam failing.

<u>The following assumptions were made with the dam break study:</u>

1. The same pattern of failure was adopted for both the existing dam failing and the raised dam failing.
2. Energy principles were used to determine the maximum flow rate, i.e. assuming a control forming in the river section where the main embankment is
3. At the onset of dam failure, the water surface level is 0.5m higher than the non-overflow crest.
4. The lower dams, weir or road does not fail during any of the two dam scenarios, i.e. the hydraulic control remains intact during the maximum flow rate.
5. That all culvert sections on road crossings were clogged completely with debris and not functioning as through-flow apertures during the extreme event.
6. The base scenario involves a dam with a volume of 200,000 m^3 to fail over 5 minutes
7. The raised scenario involves a dam with a volume of 600,000 m^3 to fail over 5 minutes
8. The river sections that were modeled cover a distance of ___km over which critical low lying houses were identified and of which floor levels were surveyed.

The hydraulic modeling was done by routing the hand-calculated flow rates (maximum flow rates) through the river section downstream of the Dullstroom Dorp se Dam. Only selected sections were surveyed given the limited budget on the project.

The flood routing involved modeling the flow levels at four (4) critical sections downstream of the Dullstroom Dorp se dam. These are:

1. Suikerbosch Kop Dam (apparently not classified)
2. An old weir structure just downstream of the Suikerbosch Kop Dam
3. The R540 Lydenburg Provincial Road river crossing
4. A privately owned dam downstream of the Lydenburg Road (recorded as B401/47 Ingifell : Category 1)

Figure 9-1 through Figure 9-4 indicate the flooding risk indicated in flow depth at specific houses for both scenarios of the existing dam failing and the raised dam failing.

Figure 9-1 : Google image of area affected by dam break

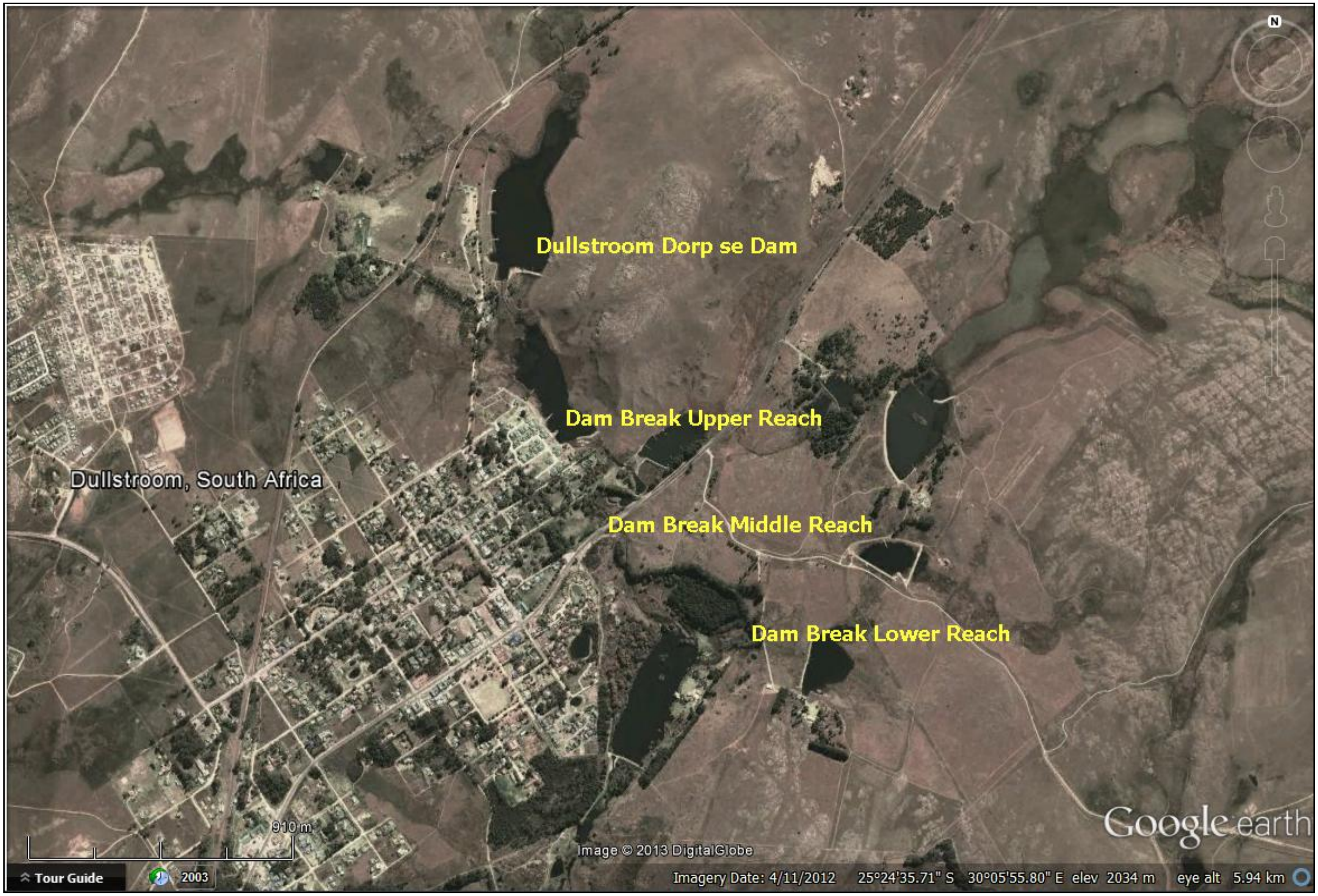


Figure 9-2 : Dam Break Assessment : Upper Reach

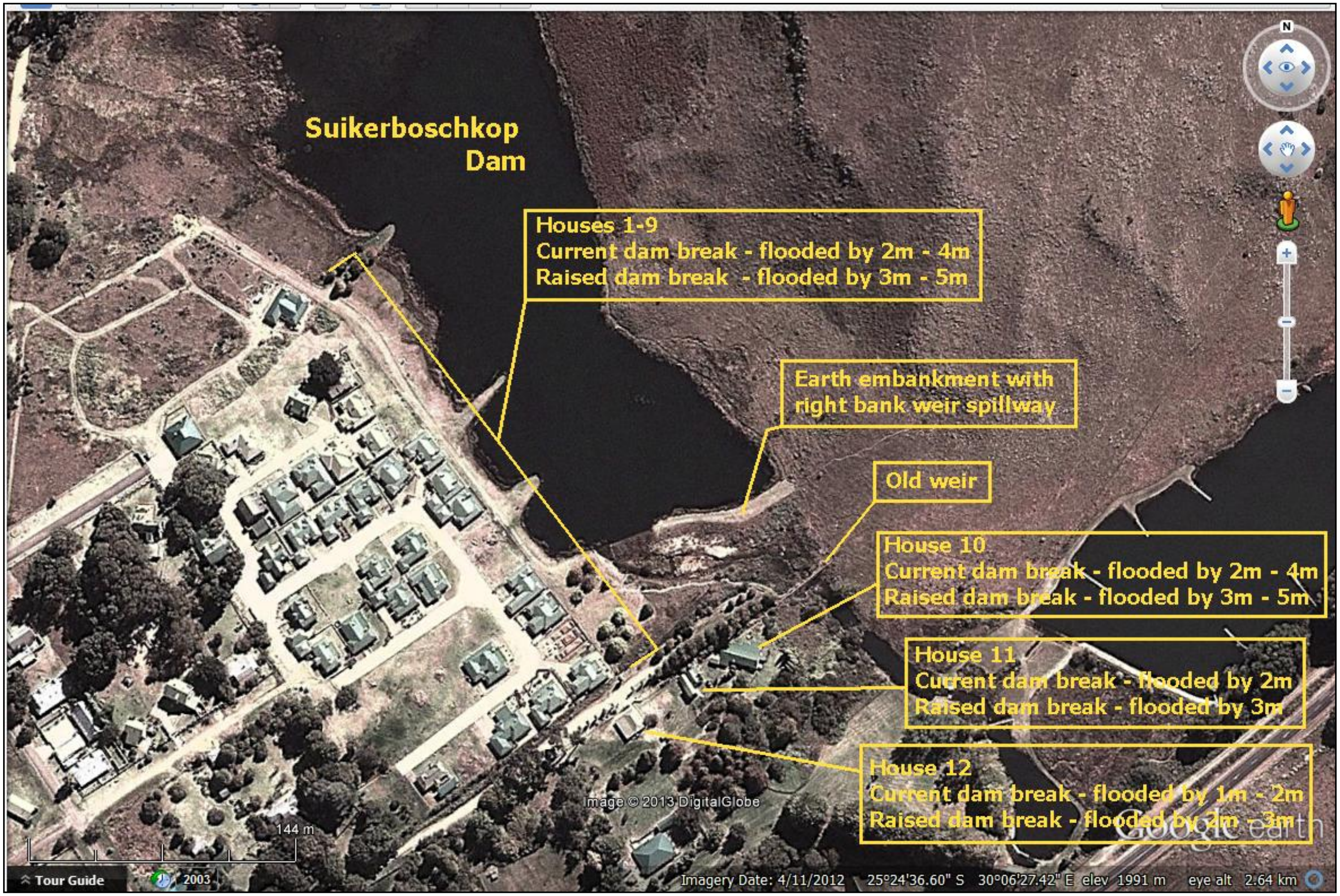


Figure 9-3 : Dam Break Assessment – Middle Reach

Figure 9-4 : Dam Break Assessment – Lower Reach

Figure 9-5 shows the output from the hydraulic model that was used to simulate the dam break scenario for the existing dam. Figure 9-6 shows the output for the raised dam break.

It is clear from the hydraulic model how controls are formed at all dams and the R540 Lydenburg Road culvert.

Figure 9-5 : Original dam failing

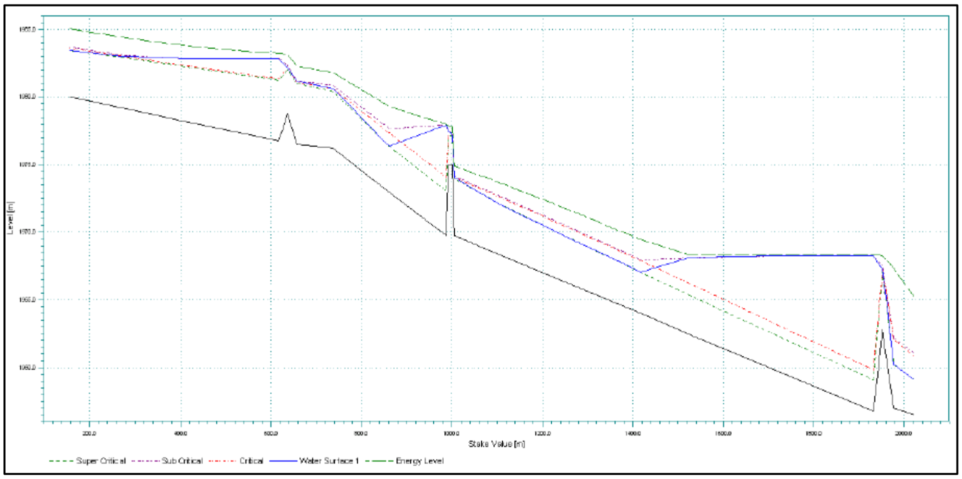
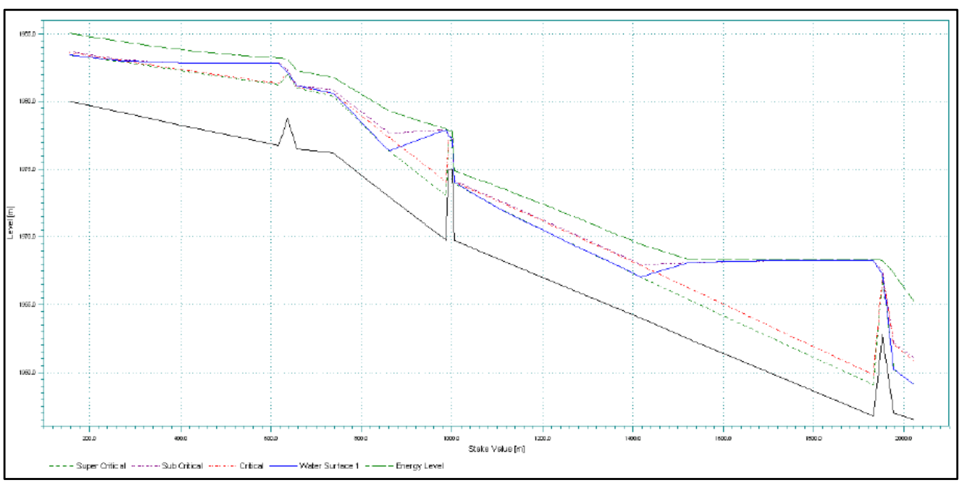


Figure 9-6 : Raised dam failing



The increase in flow depth at the 4 areas modeled varies from 0.5m to 1m. The increased flooding during the dam break of the raised dam compared to the existing dam is estimated as follows:

Houses 1,9	Flooded by 0.5m more (estimated 1m flooding prior to dam raising)
House 10	Flooded by 1m more up from 2-4m flooding
House 11	Flooded by 1m more up from 2m flooding
House 12	Flooded by 1m more up from 1-2m flooding
House 13	Flooded by 1m more up from 1m flooding
Business 14	Flooded by 0.5m up from estimated 1m flooding

House 15 Flooded by 0.7m more up from 0.5m flooding

Looking at the additional flooding of houses and businesses as a result of the raised dam breaking, it is fair to state that the additional flooding would not substantially increase the risk posed to loss of human lives. The flooding expected with the existing dam breaking poses already a fair risk to the loss of human lives estimated at more than 20.

10 RAISED EMBANKMENT FREEBOARD REQUIREMENTS

Freeboard requirements are the combination of the required head on the spillway to enable passage of the routed design flood – or Recommended Design Discharge (RDD) and necessary protection against wave run-up with the reservoir at that level.

Wave run-up is assessed on the 1:100 year hourly mean wind speed, the fetch, the slope of the embankment upstream face and the type of surface of the face.

Figure 10-1 : 1:100 year hourly wind speed[3] (with longitude corrected by author)

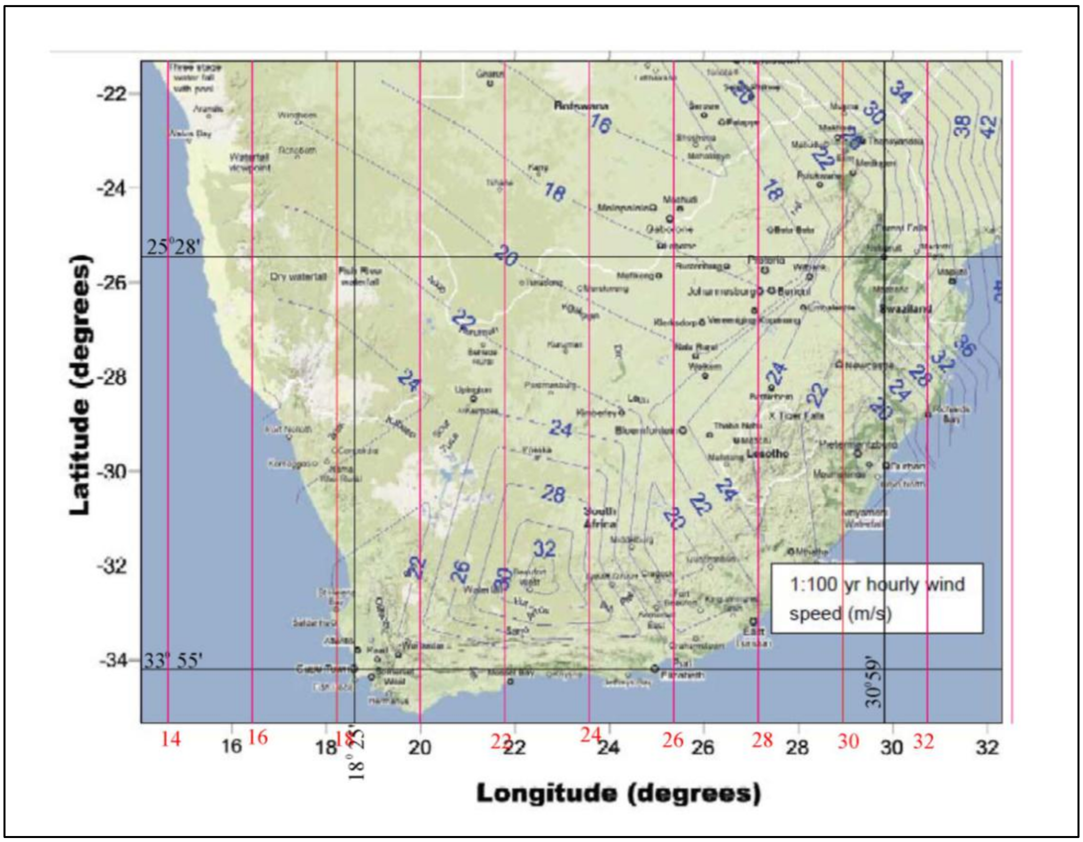


Table 10-1 : Freeboard Requirements to accommodate Wave Height and Run-up
(after SANCOLD / WRC recommendations)

Effective fetch	0.6	km
From Table A4 significant wave height is (25m/s)	0.60	m
Design versus Significant	1.10	
Design wave height	0.66	m
Wave run-up (V^2F/4850/D)	0. 52	m

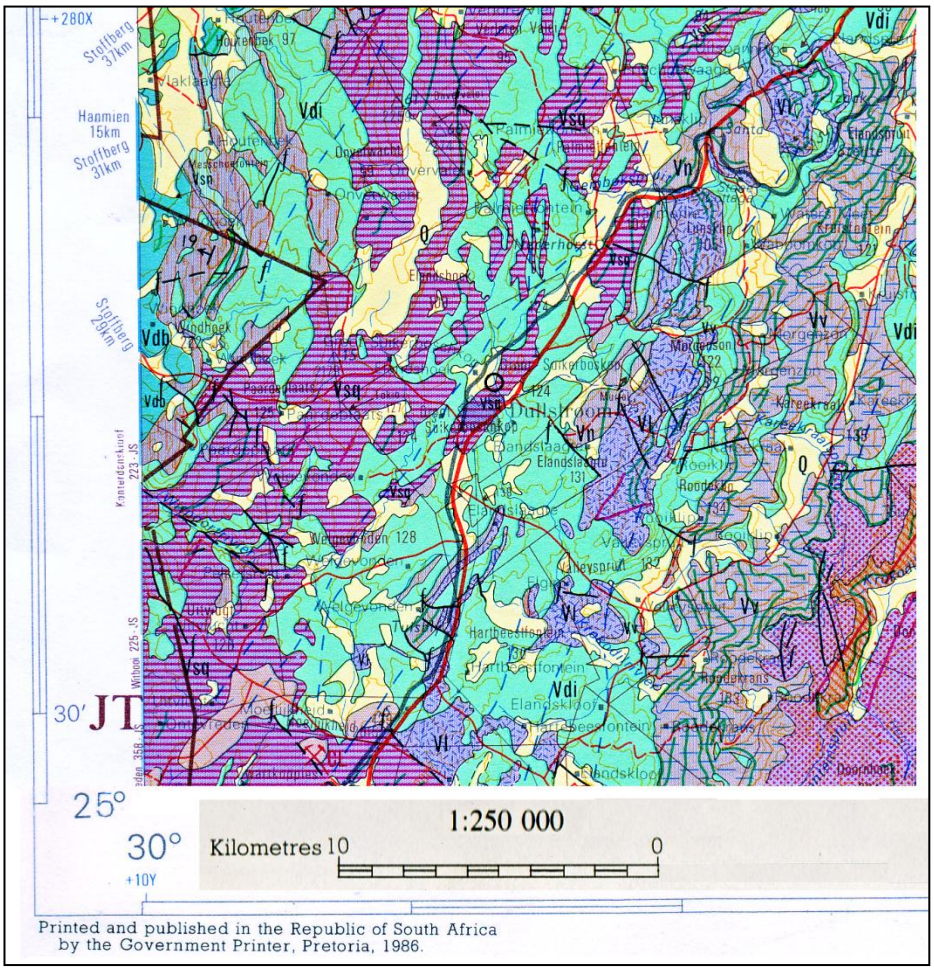
Run-up ratio/Wave height for 1:3 rip rap =	1.00	
Run-up height	0.52	m
Freeboard = Run-up + Design wave height	1.18	m

This, added to the RDD level is almost identical to the SED level and an additional 0,2m freeboard has been allowed.

11 REGIONAL GEOLOGY

Information on the regional geology has been abstracted from the 1:250 000 mapping produced by the then Department of Mineral and Energy Affairs 1986

Figure 11-1 : Regional Geology excerpt from 1:250 000 mapping

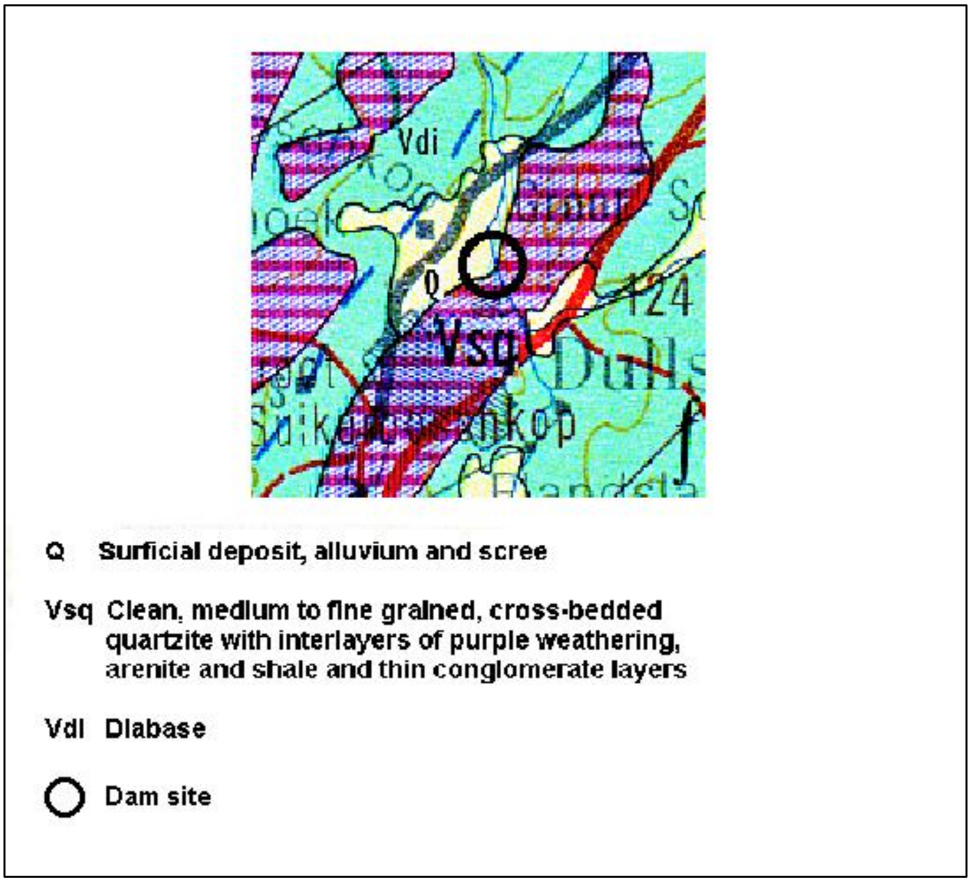


This shows the dam site to be sited partially on deep alluvial deposits on the right bank and quartzite / quartzitic sandstone on the left bank as shown in the enlargement below.

Comments on the geology of the dam site given by the geotechnical investigation seismic team are in accord with this :

"The area is underlain by Pretoria Group sedimentary rock, in particular quartzite thatprotrudes as hills, and by dolerite intrusions. The left (east) end of the dam wall abuts one such quartzite hill whilst the right end rests against a hillock of like composition. To the west of the dam wall is a spillway that contains a narrow concrete-lined channel and then gradually rising land that forms the right bank of the dam."

Figure 11-2 : Site Geology from regional 1:250 000 mapping



12 SITE INVESTIGATIONS

The Feasibility Study for raising Dullstroom Dam is hampered by a lack of historic information on the design and construction of the existing dam. There are no "As Built" drawings or reports that could assist in defining the dam materials (soils, rock fill, zoning, filters etc.) or what foundation treatment and grouting were done on the original dam. Consequently, contracts were let, through normal supply chain procedures, for topographical survey, geotechnical seismic survey and materials investigations, and drilling and grouting trials to be carried out. These were programmed initially for March – May 2013, but were delayed by awards of tenders. Topographical surveys were completed in June 2013, but the drilling and grouting was carried out in November 2013 and seismic traverse, trial pitting etc. were executed in January 2014.

13 TOPOGRAPHICAL SURVEYS

Topographical surveys were carried out of the whole reservoir area and immediate surrounds, as well as the potential borrow area on the Lydenburg road and the areas affected by a possible dam break.

14 GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations were intended to be in two phases. Initial seismic traverses, cone penetrometer testing and trial pitting were intended to inform drilling, water testing and grouting trials on the dam site. In the event, the drilling and grouting contract was awarded three months before the contract covering seismic and other work. This was disadvantageous to efficient site investigation. Reporting below is therefore in the chronological order of investigations carried out.

14.1 Drilling, Water testing and Grouting Programme

14.1.1 General Discussion

Drilling, water testing and grouting trials were completed in order to assess the permeability of the area and to give information on which to estimate costs for the sealing of the foundations of the raised dam structure. Seven holes were drilled on essentially original ground along the potential line of the dam embankment and spillway structure. Holes were site on the existing dam left and right abutments, along the ridge between the existing dam and the spillway outfall, in the spillway outfall and on the far right flank. The report on drilling and grouting is appended as Appendix 2.

In brief, the findings were that the altered rock foundations in the dam embankment area are indeed pervious and will need to be grouted. It is surmised, from the lack of seepage from the dam solum, that the existing dam foundations were competently grouted, but that the adjoining abutment on the right (west) flank was not. The area between the dam embankment and the spillway is very pervious and will require a carefully designed grouting programme. The tests have also shown that the spillway area is underlain by deep clays which have water eroded pipes at random and which will need special attention to create an effective cut-off under the spillway structure.

The drilling and grouting programme has provided valuable information for incorporation in the design process, which will influence design selection and will dictate significant costs in the construction phase.

If the geotechnical investigations had been carried out prior to the drilling and grouting, as originally programmed, the drilling programme might have been modified to intercept particular strata and there would have been indications of the low strength clays beforehand.

14.1.2 DELIVERABLES

Water tests and grout takes at specified or lower pressures, as may be achieved, with grout mixes being varied according to the permeability as defined by the **lugeon** value calculated from the water test flow rate and pressure sustained over 5 minutes.

The lugeon is a seepage or water loss of 1 litre/second at a head of 1bar over a length of hole of 1m. Grout mixes were required to be as follows, for the encountered permeability conditions :

Table 14-1 : Linking Lugeon to Permeability and Grout Mix

Lugeon	Permeability	Start Grout Mix W:C Ratio	
		By Volume	By Weight
0 - 3	Impervious	6:1	4:1
3 - 10	Low	4:1	3:1
10 - 30	Medium	2:1	3:2
30 - 60	High	3:2	2:1
> 60	Very High	1:1	1:1

14.1.3 SITE WORK

A plan of the hole layout is shown in Figure 14-3 below. Seven holes were investigated.

Holes were drilled to 3m depth and plastic pipe casings of 2m length were set in at a nominal 100mm above ground, leaving about 1.1m of uncased hole for the initial water testing. Casings were grouted in and the holes flushed out or re-drilled.

Water tests were carried out on the holes stage by stage, followed by grouting with a mix suitable for the permeability of the hole. The holes were again flushed or redrilled, extended to 6m and then 9m, with water tests and grouting. Pressures for testing and grouting were limited to 150kPa/metre depth at the top of the stage, to reduce the likelihood of causing hydraulic fracturing,

On completion, the holes were grout filled and covered,

14.1.4 RESULTS

Results are given in Appendix 1 below. Grout takes are in 100litre batches as no flow meter was used for grouting and the residual was not measured. This is indicated by the uniformity of the results. However, the important information is that a grout mix of the consistency used was dictated by the permeability measured and could be injected – in most cases more than one mix volume.

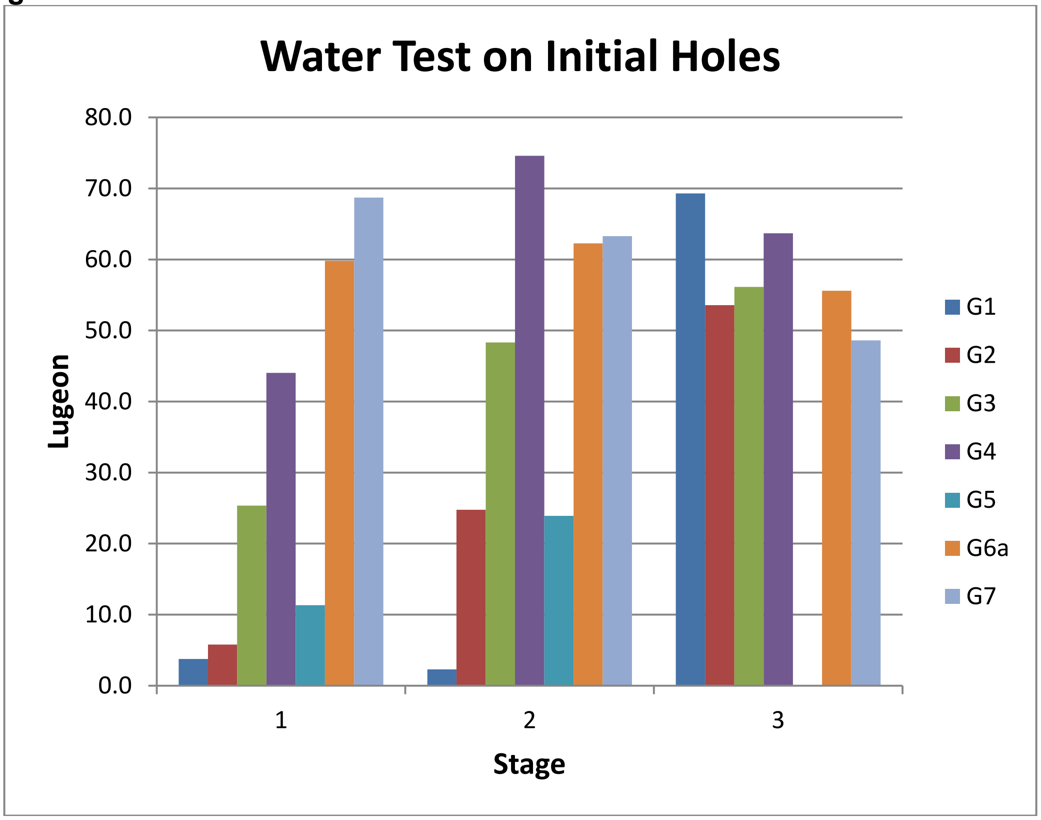
An indicator called “grout acceptance” has been used, which is the total grout take (or mixed, in this case) divided by the water cement ratio. This means that a high volume of very thin grout will have a similar grout acceptance value as a low volume of thick grout.

Referring back to Table 14-1, it is important to not that a range of 3 -10 lugeon is considered as low permeability and 10 – 30 lugeon as medium permeability. At the lower stages, the holes are highly permeable. Hole 5 is anomalous, in that there was a connection to surface established, despite the restrictions on maximum injection pressures.

Second phase water tests – immediately before grout injection – were incomplete and therefore inconclusive. Where possible, indications will be taken from these tests.

From Figure 14-2, it can be seen that hole G3 is very pervious in the top 3m, while hole G1 is far less pervious. Hole G4 is extremely pervious at depth and Holes G5 and G7 (originally G6) are also vey pervious at depth.

Figure 14-1 : Water Test results



Illustrative grout acceptance lugeon values, with take and pressure divided by the water cement ratio are also inconclusive as the flow rates are not consistently recorded.

Drilling at G5 was difficult, due to the clay soils squeezing in on the drill bit and rods.

In summary, the water pressure and grouting tests have indicated that –

- the existing dam foundations were probably grouted, as evidenced by lesser grout takes and permeability of holes G1 and G2 and the fact that the area immediately downstream of the embankment shows no seepage.

- The area between the existing dam and the spillway has moderate to high permeability, shown by holes G3 and G4, and as evidenced by the seepage through the right flank of the existing dam.

- The sandstone foundations and flanks will require systematic close spaced grouting.

- The right flank and spillway area, although apparently in deep clays situated along a fault zone between the quartzitic sandstones to the east and shales to the west, has high potential for grout take and therefore high seepage flows. This must be associated with piping developed over time from the existing dam. This area will require a deep cut-off and close spaced grouting to ensure that there are no pipe seepage zones.

- The squeezing nature of the ground indicates low foundation strength under saturated conditions, which will require special consideration during design and construction stages.

Figure 14-2 : Grout Acceptance per Stage

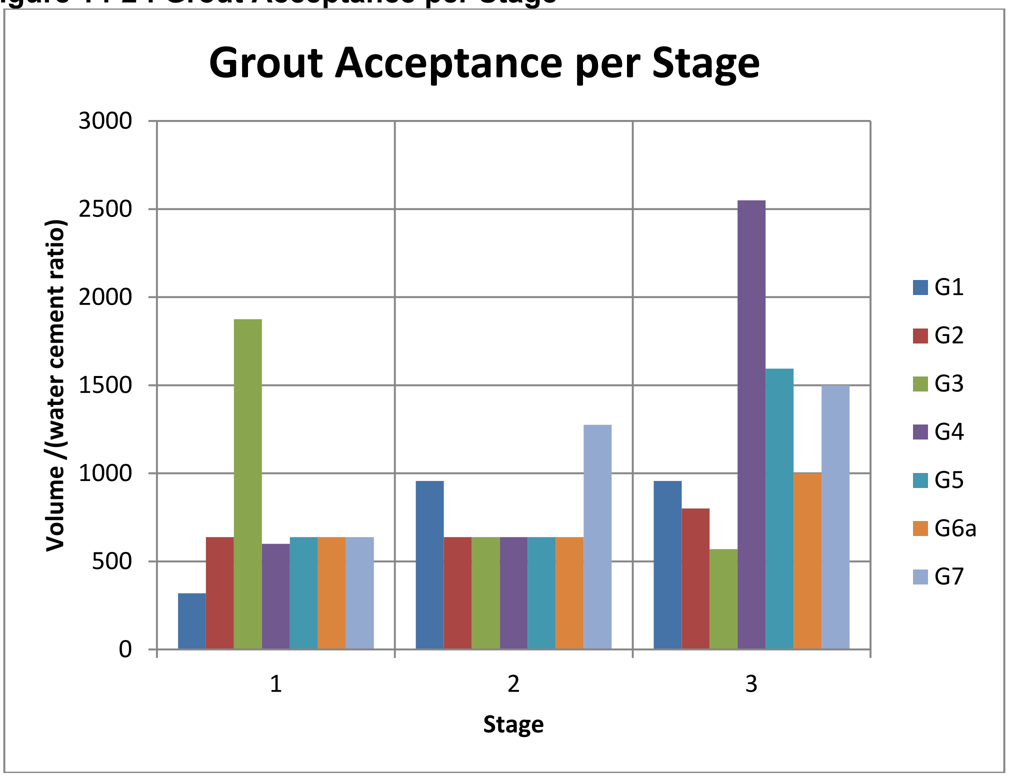


Figure 14-3 : Layout of Holes for Water Testing and Grouting

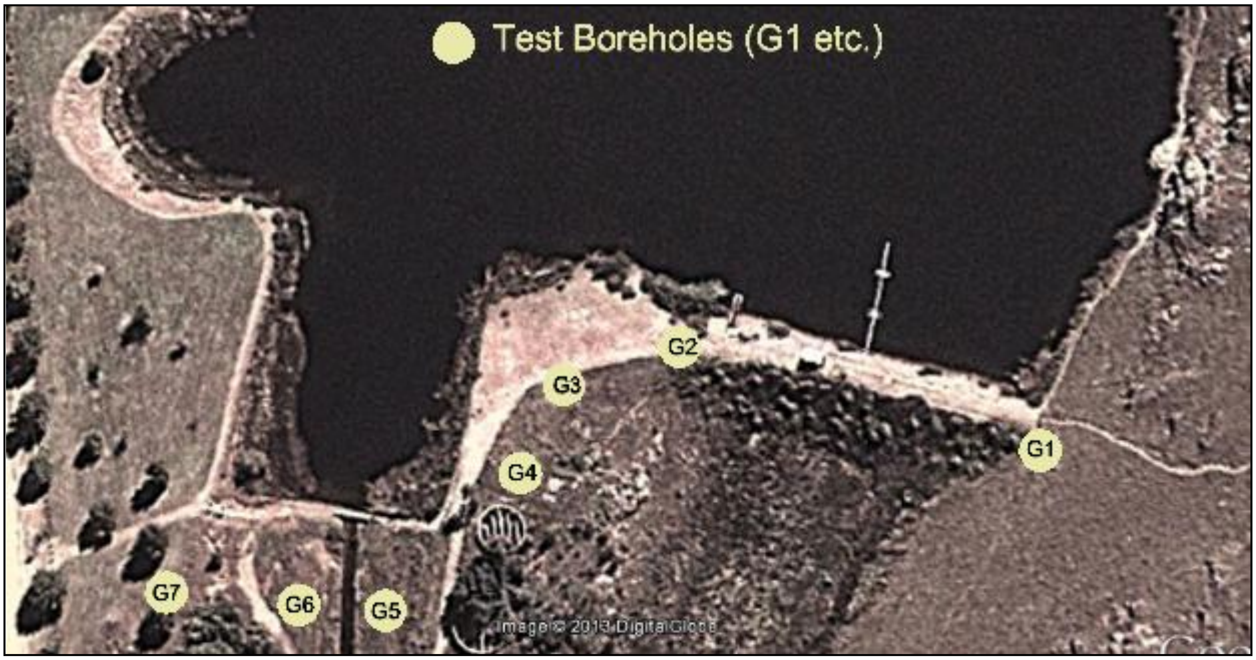
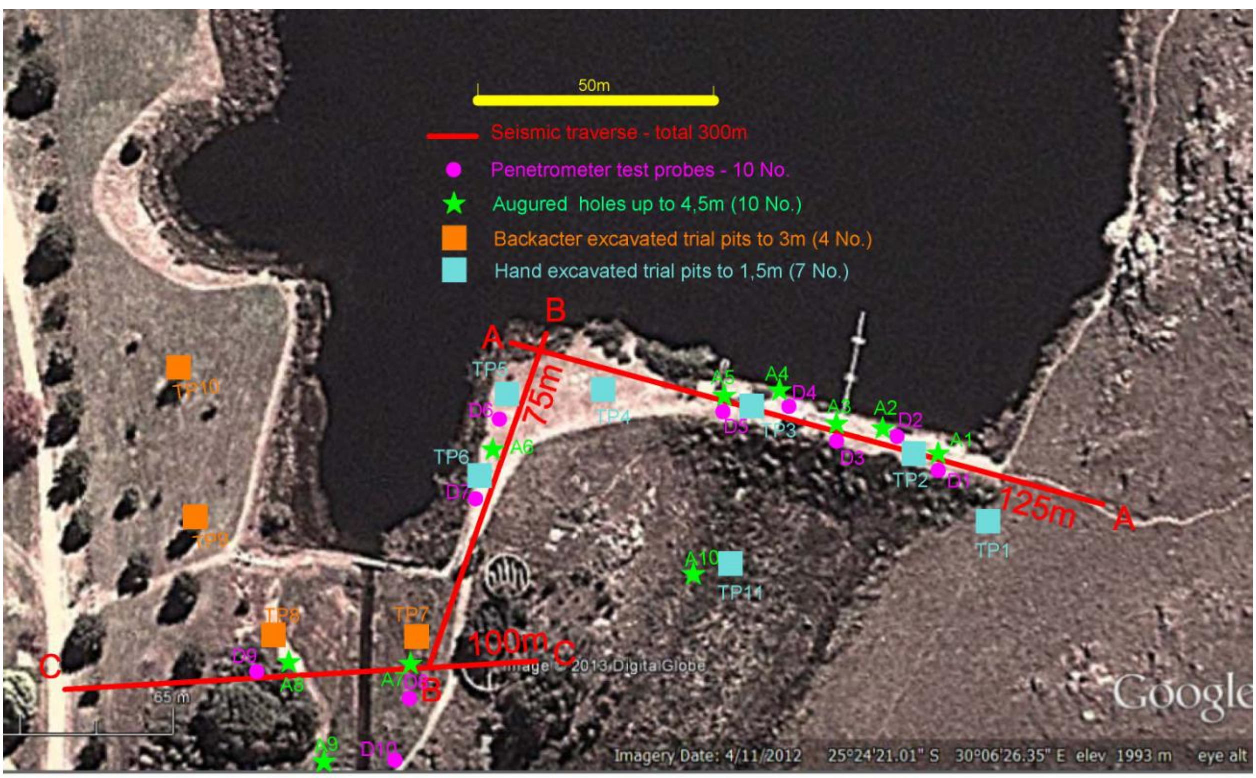


Figure 14-4 : Geotechnical investigations layout



14.2 Seismic Traverse

The tender for geotechnical investigations was awarded late December 2013. Trial pits were excavated in the potential borrow areas, in the spillway foundation area and within the dam solum in apparent original ground. .

14.2.1 . Introduction

The dam site area is underlain by Pretoria Group sedimentary rock, in particular quartzite that protrudes as hills, and by dolerite intrusions. The left (east) end of the dam wall abuts one such quartzite hill whilst the right end rests against a hillock of like composition. To the west of the dam wall is a spillway that contains a narrow concrete-lined channel and then gradually rising land that forms the right bank of the dam.

A seismic refraction survey was carried out by a team under Dr Richard Day, on the 17th January. The traverses included a line across the spillway, one along the existing dam wall and a third connecting the two. Variations in execution of the plan had an additional line on the right bank of the dam and with the spillway line ending at the concrete channel with insufficient space left to meaningfully extend the line further to the east. The gap in coverage, however, was filled in on the 30th, when the opportunity was also taken to collect a line of resistivity data along the dam wall. Traverse positions were recovered with the aid of a Garmin GPS and Google Earth image (Table 14-2 and Figure 14-12)

14.2.2 Methodology and Equipment

A Geode seismograph was employed to gather the refraction data with geophones set three metres apart in sets of 24 (spread length of 69 metres), apart from one twelve geophone line using a five metre spacing. A hammer hitting a steel plate was used to generate the seismic signal, several events being stacked to arrive at an acceptable record. Data from three in-spread shots were collected as well as end and off shots. The data were recorded onto hard disk for processing.

Ancillary equipment included a Garmin GPS, dumpy level, tripod and staff. The surveying equipment was used to determine changes in elevation across the sites so that the seismic models could be plotted beneath profiles of relative elevation. The ten-metre elevation reference level was arbitrarily selected as being the top of the concrete channel closest to the eastern end of line 1; this level is also similar to the water level in the dam at the time of the survey.

SeisImager was used for the data interpretation. First-arrival times were identified and recorded using Pickwin. A graph of arrival times was constructed using Plotrefa and arrivals were assigned to one of three layers, according to the fastest horizon the wave passed through. The travel-time data were then processed using the time-term method to arrive at starter model for the tomographic interpretation. The tomographic interpretation was then exported as an ASCII file and contoured in Surfer.

The resistivity data were collected using an ABEM LS system. The electrode separation was set at three metres and the readings collected using a Schlumberger-Wenner configuration.

Res2Dinv was used to process the resistivity data. As with seismic data, relative elevations were added during the modeling process.

Table 14-2 : Coordinates of Seismic Traverses (WGS31)

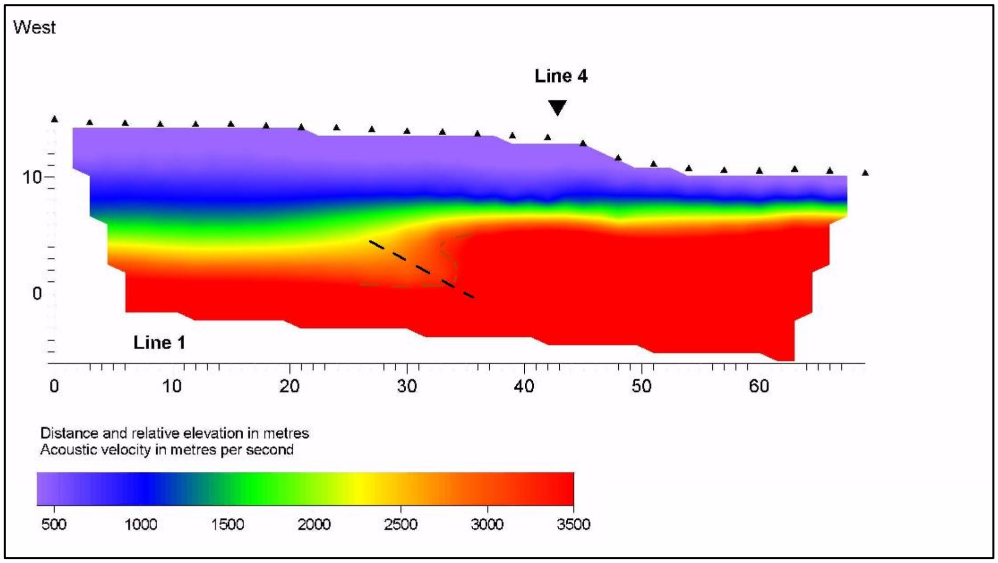
Line	Station	Lo Y	Lo X
1	0	89836	2811420
1	30	89802	2811420
1	60	89775	2811420
1	69	89765	2811425
2	0	89724	2811377
2	30	89731	2811405
2	60	89738	2811435
2	69	89738	2811444
2	90	89739	2811464
2	102	89737	2811475
3	0	89723	2811373
3	30	89694	2811377
3	60	89662	2811387
3	69	89654	2811387
3	90	89632	2811390
3	114	89611	2811394
4	0	89791	2811374
4	30	89791	2811405
4	60	89791	2811435
4	69	89791	2811446
5	0	89749	2811425
5	20	89746	2811440
5	40	89740	2811466
5	55	89736	2811476
PUMP		89664	2811386

14.2.3 Results

The seismic models are presented in figures 2 and 3 and they show acoustic velocities from less than 400 to more than 3500 metres per second (m/s). The higher velocities, say above 2000 m/s, are compatible with fractured to massive quartzite whilst lower values are indicative of weathered to completely weathered rock, and soil. Saturated material is expected to have a velocity in the range 1500 to 1800 m/s.

The models are reviewed from west to east.

Figure 14-5 Seismic Line 1



Seismic basement on line 1 approaches surface towards the east (Figure 14-5). Part of this decrease in depth is caused by a change in topography but there is also a step in basement level around 30 metres; this step coincides with a low velocity zone that dips at a shallow angle towards the east and may mark a contact between two beds. Basement is sharply deeper on line 5 with the development of a thicker intermediate layer (~1500-2000 m/s).

Line 4 straddles a contact between dolerite in the north (TP10) and quartzite in the south (TP9). The model for line 4 shows a decrease in the depth to seismic basement -inferred rock -towards the south but without any definite boundary to indicate the position of the contact (Figure 14-6).

Figure 14-6 : Seismic Line 4

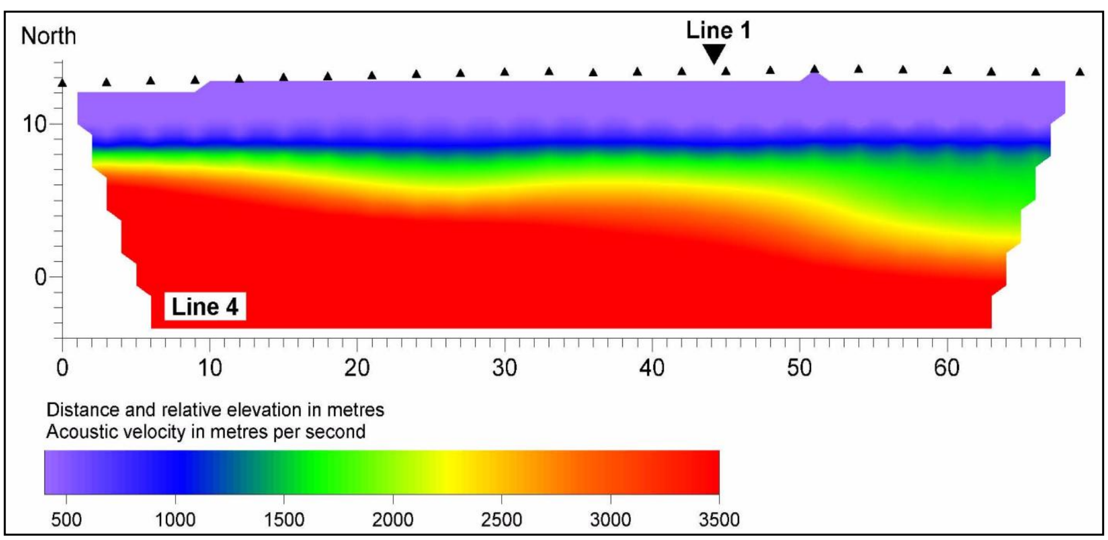


Figure 14-7 : Seismic Line 5

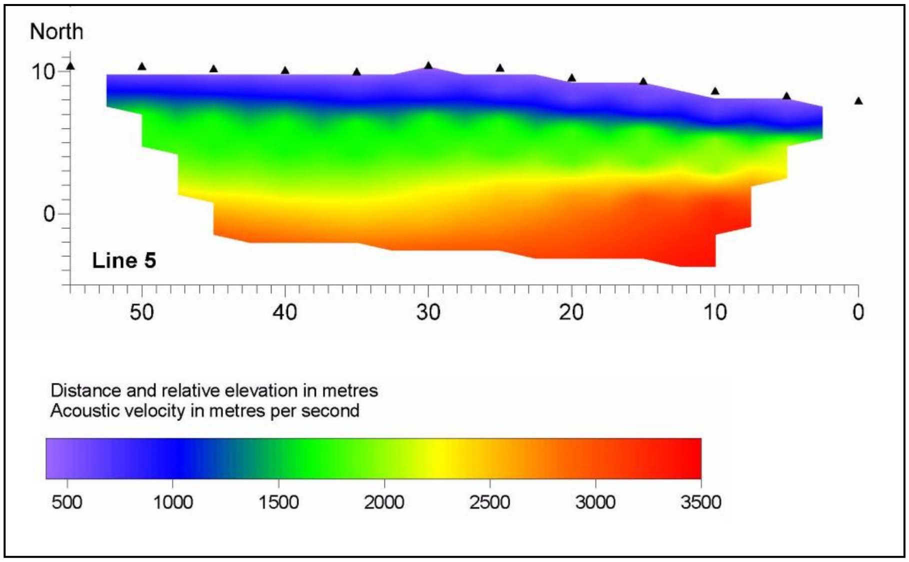
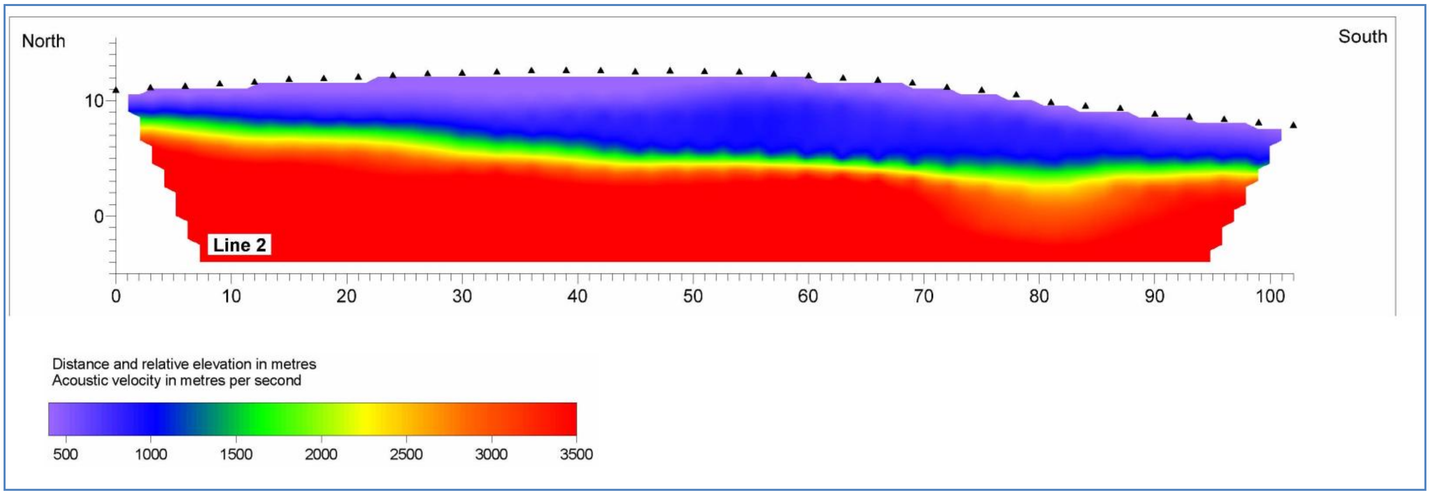
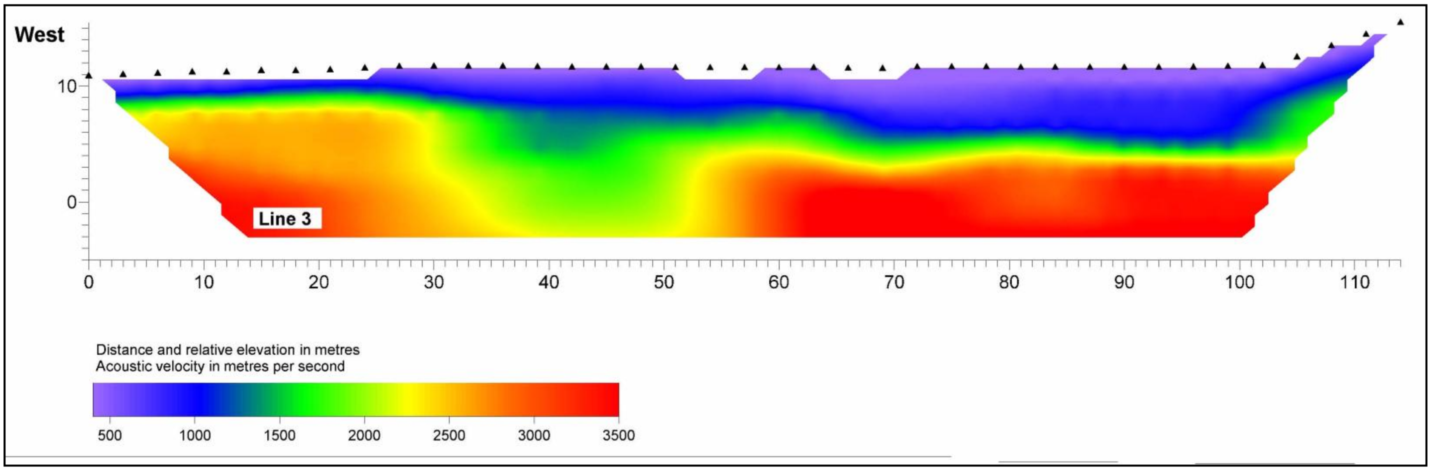


Figure 14-8 : Seismic Line 2



Basement on line 2 dips steadily towards the south, but is higher than recorded on line 5 whilst the model for line 3 shows a sharp depression in bedrock head immediately east of the hillock.

Figure 14-9 : Seismic Line 3



The resistivity model for line 3 is shown in Figure 14-10 along with a repeat of the seismic section. This is then compared with the valley line of the existing embankment, in Figure 14-11, which shows the area of low resistivity (discussed below) is well to the west of the original dam right abutment

Very resistive to reasonably conductive conditions are recorded on the model with a horizontal conductor over a resistor in the east, a break in resistive basement with presence of a vertical conductor, then a short continuation of the previous arrangement, another break this time dipping towards the west and finally a resistor at surface in the west. (The central break partly coincides with the depression in seismic basement, but the sub-horizontal break is not reflected in the seismic model at all.)

The resistor is taken to reflect the presence of quartzite at depth in the east and near surface in the west, and the horizontal conductor the dam wall. The vertical and sub-vertical conductors that cut through the resistor suggest preferential zones of weathering and and/or water-filled fractures. Faults or fracture zones on either side of the quartzite hillock are taken to be the source of the depressions in seismic and resistive basement.

14.3 Trial Pitting and Penetrometer Testing

14.3.1 General Results

The report from Engeolab CC is attached as Appendix 3, with

Figure 14-10 : Seismic Line 3 with Resistivity

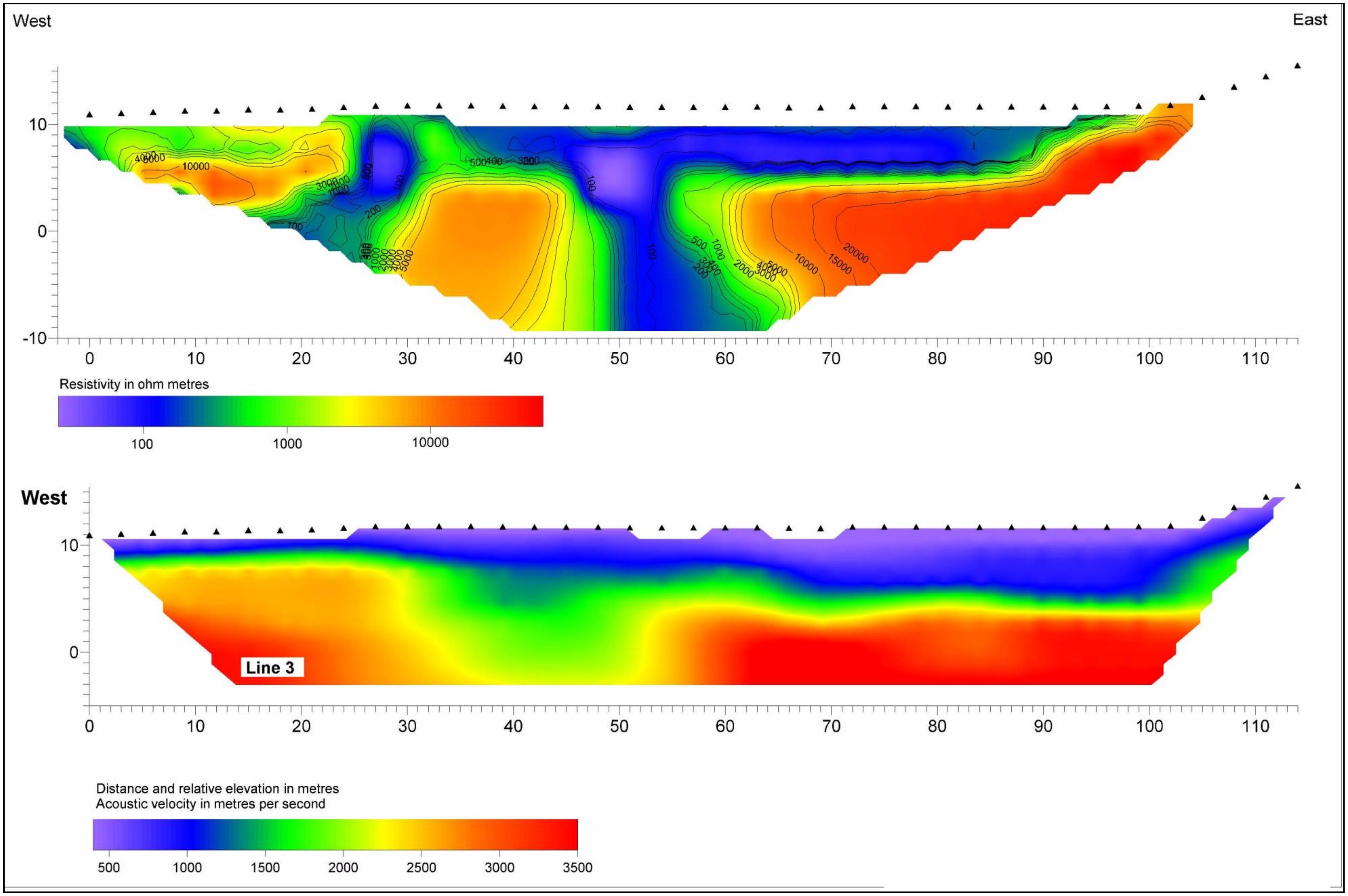


Figure 14-11 : Lines 3 with valley profile

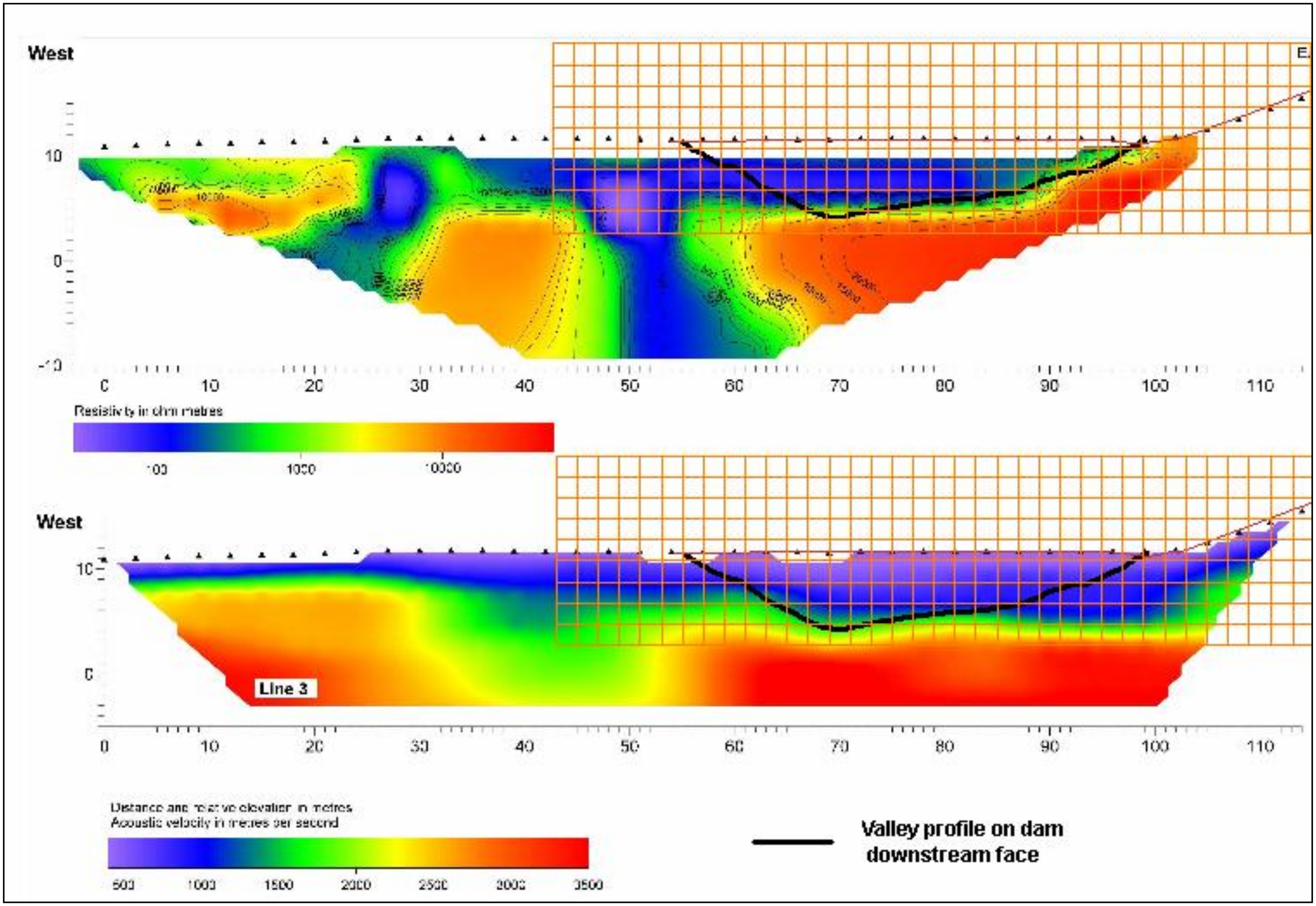
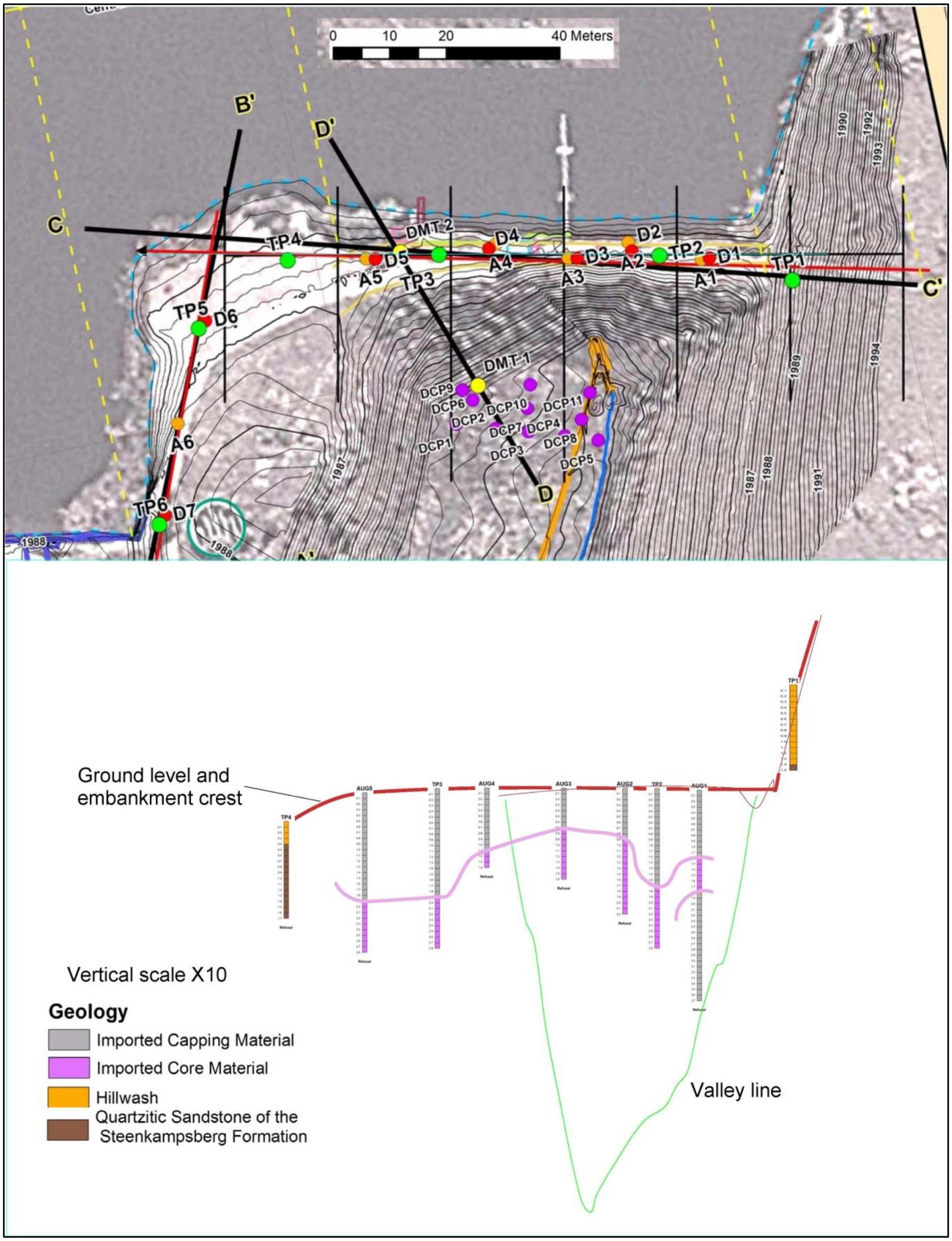


Figure 14-12 : Layout of Seismic Traverses

Figure 14-13 : Plan & long section on existing dam with auger and trial pit logs



15 RAISED DAM SECTION

15.1 Section

The proposed raised embankment section is shown below in Figure 15-1. It is envisaged that a single downstream slope of 2.5 : 1 will be stable under drained conditions and no berms will need to be incorporated. A rockfill toe may be necessary, depending on the degree of drainage achieved or expected and the foundation conditions discovered. Pioneer rockfill may be required to stabilize the toe area, which would then be the initial phases of a rock toe.

The new embankment fill will be drained by both a blanket drain on the foundations and a chimney drain on the existing embankment face.

The impervious core will be extended upward on the existing upstream face slope of 3.5:1 and then capped with a wearing course of semi-plastic material.

A low wave wall is suggested, as a formal definition of the upstream face.

The Embankment between the current embankment and the spillway will be constructed with 3:1 upstream and 2.5:1 downstream slopes, containing an impervious clay core constructed on a low concrete cutoff wall. The reason for the cut off wall is to minimize excavation in the highly jointed and altered quartzites. This area will require careful grouting.

The far right flank will be a simple earth embankment with a clay core extending into the foundation. Since the whole of this flank is clayey material, a deep cut-off is not necessary. However, drainage pipes have been formed in this residuum, which must be addressed.

15.2 Sequence of work

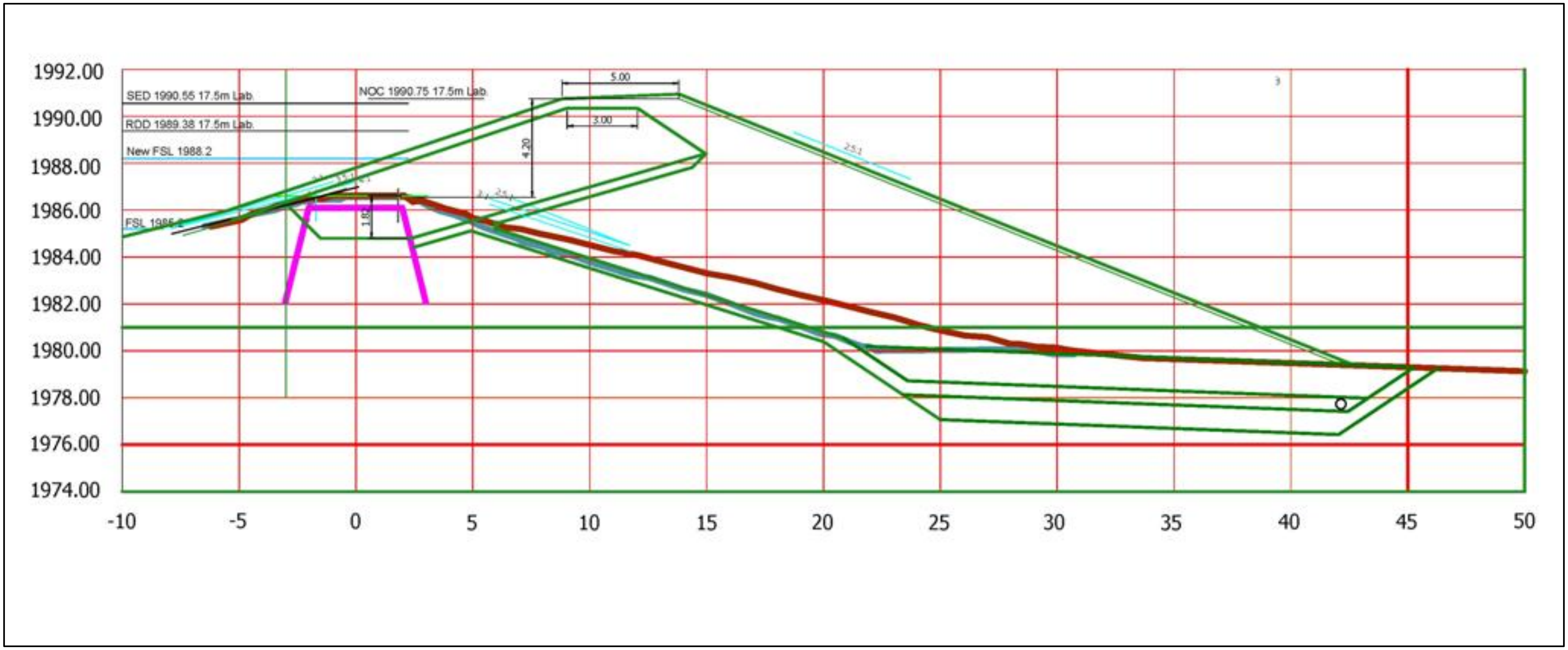
The proposal for the main embankment is that the solum and downstream face is cleaned of all organic material and prepared for a two layer filter / drain, connected to a blanket drain system on the sub-horizontal foundation. A series of surface drains must be dug in the foundation area for the extended downstream toe and pioneer rockfill placed where necessary, to allow the preparation of the surface for a blanket drain system. The blanket drain should be underlain by a pipe collector that allows for measurement of seepage from defined sectors of the foundation blanket and chimney drains.

Following on the preparation, an extension of the bottom outlet must be installed, with a sacrificial gate valve at the existing gate valve. This outlet should be available for any bottom releases necessary as well as to serve as a temporary bulk water draw off for the municipal water treatment works, allowing the dismantling of the current pump house on the dam crest.

The downstream embankment and drain systems can then be constructed to existing crest level.

While this work is in progress, grouting would be done of the main curtain along the existing embankment and new connecting embankment to the spillway structure.

Figure 15-1 : Section of Existing Embankment and Proposed Raised Embankment



16 ESTIMATED CONSTRUCTION QUANTITIES

17 ESTIMATED CONSTRUCTION COSTS

18 RECOMMENDATIONS

It is recommended that

1. The dam full supply level (FSL) be raised by 3,0m, from RL 1985.2 to RL 1988.2;
2. The embankment be raised by about 4,2m to accommodate the freeboard and wave run-up requirements, with the new non-overspill crest (NOC) at RL 1990.75;
3. the proposed raised dam is considered to be Category III, with all concomitant safety requirements;
4. the calculated Recommended Design Flood (un-attenuated) of $350m^3/s$ is adopted for the design of the spillway for the raised Dam;
5. the Safety Evaluation Flood (un-attenuated) of $740m^3/s$ be used to assess extreme conditions;
6. a labyrinth spillway is constructed on the current spillway outfall, the structure being 17.5m from upstream to downstream, with throat width of 8m, 10m per bay and a total width of 52m. This will have an effective overspill length of 197.6m.
7. the structure in this area will be founded on deep sandy silty clays and will require a stable raft foundation with a deep cut-off formed by bentonite slurry trenching or by contiguous jet grouting (the former would be preferable);
8. the outfall from the spillway should be stepped, to attain the level of the lower dam reservoir, over as short a reach as possible, to reduce the foundation problems and to provide embankment core and casing material;
9. the right flank area upstream of the spillway and right flank embankment should be used as the primary borrow area for the raised embankment core material;
10. the borrow pit on the Lydenburg road should be utilized for relatively pervious casing material;
11. all licenses and permits for borrow pits will have to be sought;
12. extensive grouting of the lower foundations under the enlarged embankment, particularly within the central ridge, should be carried out early to reduce seepage into the downstream toe area and facilitate pioneer and foundation preparation work;
13. further grouting should follow the embankment construction;
14. a new permanent intake pipeline should be constructed in the ridge between the existing embankment and spillway, passing under the new spillway structure, to enable the installation of a permanently flooded suction raw water pump station on the right flank of the dam, below the spillway;
15. the intake should have a screened floating draw-off located by pantograph rods or cables, which would abstract water from 200mm to 500mm below surface;
16. the existing bottom outlet should be fitted with a new downstream valve, before work commences on the embankment raising. The outlet should be cleared by air blast and a new inlet extension fitted, raised above the current silt level. A temporary raw water supply pipeline should be installed, which would revert to a bottom outlet after completion of the final intake;
17.

19 REFERENCES

(1) BKS (Pty) Ltd. ***"Dullstroom and Sakhelwe Water Supply Scheme : Dullstroom Dorpsdam; Decision Support System"*** ; DWA, May 2012.
(2) WRC report no 298/6.2/94 first edition 1994.
(3) SANCOLD 2011 Guidelines on Freeboard for Dams; WRC Report No. 1759/2/11, August 2011

20 DRAWINGS

Figure 20-1 : Survey plan of Existing Dam

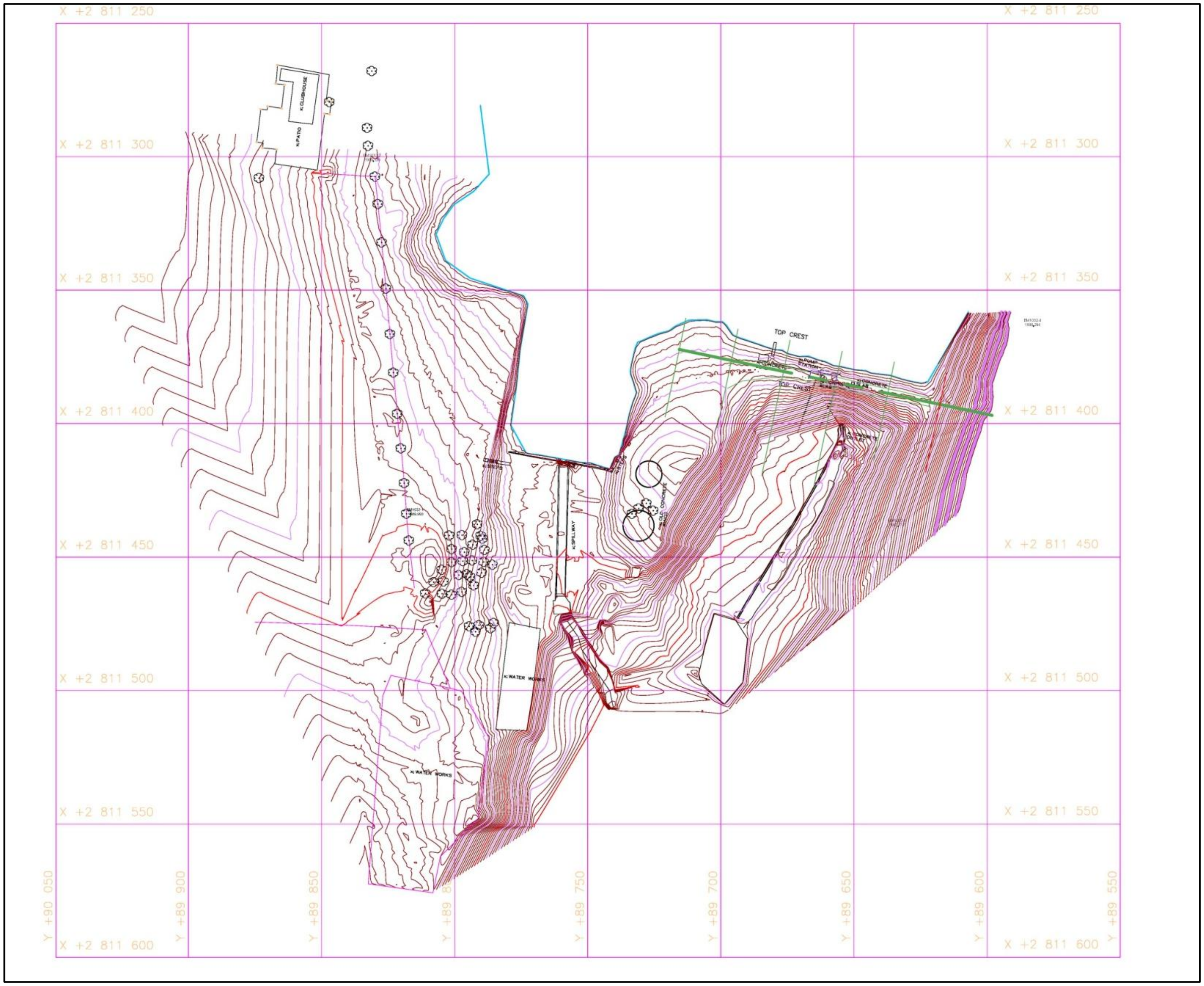


Figure 20-2 : Plan of proposed raised dam