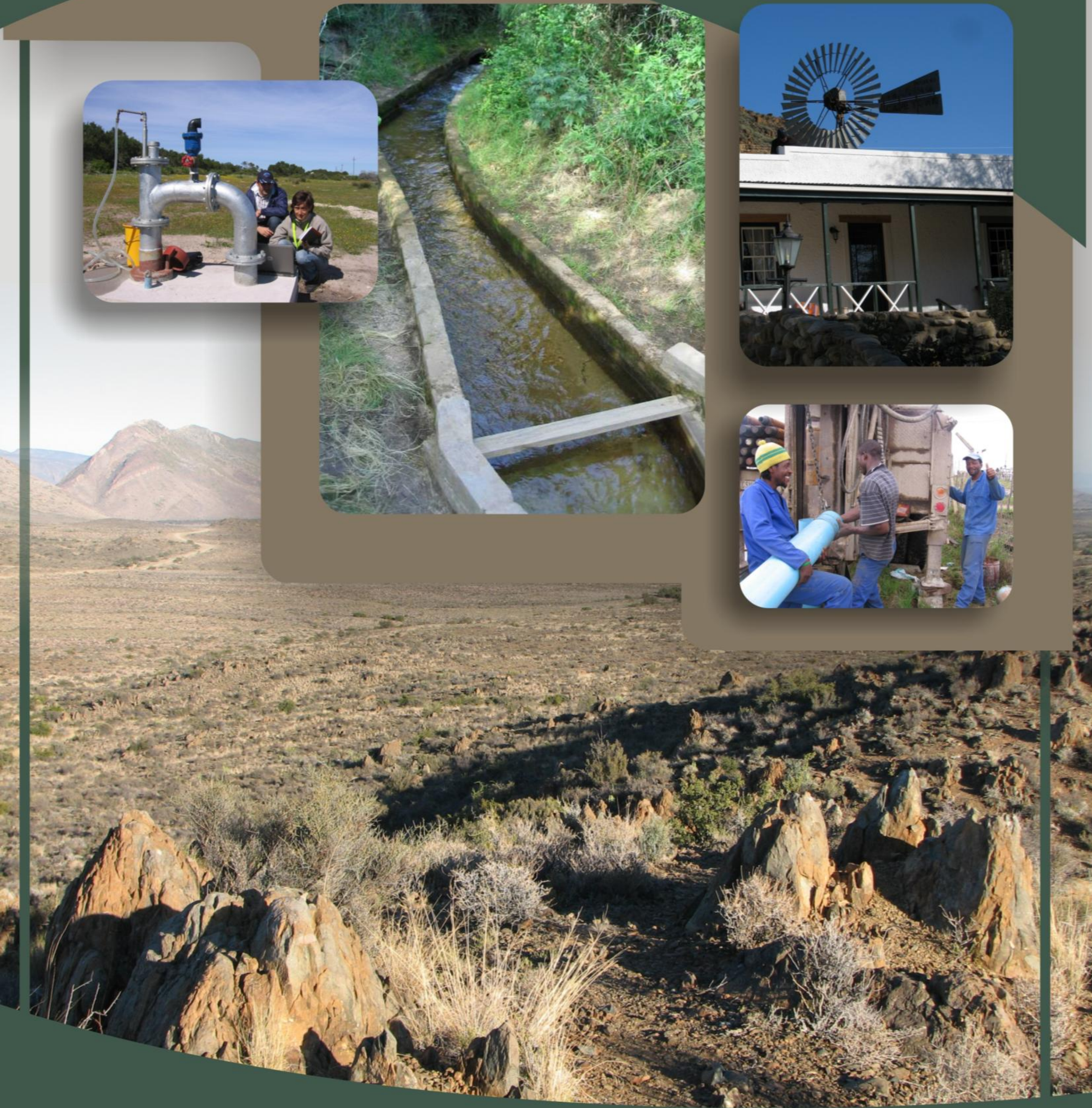
Potential Artificial Recharge Schemes: Planning for implementation



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Potential Artificial Recharge Schemes: Planning for implementation

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FIGURES AND TABLES

1 INTRODUCTION

In 2007 DWA published the first Artificial Recharge Strategy (DWAF, 2007) with a clear vision of maximising the use of sub-surface storage.

> **Artificial recharge vision**
>
> **To use natural sub-surface storage as part of Integrated Water Resource Management wherever technologically, economically, environmentally and socially feasible.**
>
> *Source: Artificial Recharge Strategy, DWAF, 2007*
> **(available at www.artificialrecharge.co.za)**

This report focuses on two of the DWA-supported artificial recharge assessments undertaken during the roll-out of the artificial recharge strategy, namely those at Prince Albert and Plettenberg Bay. It also summarizes the other studies undertaken during the roll-out project, namely those at Sedgefield, Hermanus and in the Vermaaks River Valley near Oudtshoorn, as well as the recent borehole injection tests carried out on the Langebaan Road Aquifer. Brief summaries have been included on other areas where artificial recharge has been proposed, like the Sand Dams of the Limpopo and Mpumalanga Provinces, the Lephalale artificial recharge assessment, and the Kenhardt and Kathu proposals. The aim of the report is to provide examples of various levels of investigation into artificial recharge. The examples provided are shown in **Figure 1.1**.

The **Prince Albert** case study is an example of how an aquifer can rapidly be replenished to ensure it is full prior to the onset of summer. This example is described in detail and can serve as a guide for future studies of this nature.

The **Plettenberg Bay** study provides an example of an opportunity to utilize more of the aquifer’s storage by drawing water levels down deeply and then transferring winter runoff to replenish the aquifer prior to summer.

The **Sedgefield** study presents an opportunity to utilize a sand aquifer to treat the town’s waste water for re-use.

The **Langebaan** study proposes a way forward after an initial borehole injection test.

The **Hermanus** study looks at the possibility of diverting household roof runoff (and potentially urban runoff) into the sand aquifer for household irrigation.

The **Vermaaks River Valley** assessment provides an example of opportunistic artificial recharge with storm runoff.

The **Sand Dams** section gives examples of areas potentially suitable for augmenting small-scale irrigation supplies.

The **Lephalale** case study presents a recommendation of simultaneously developing wellfields and artificial recharge schemes.

Kenhardt is an example of how an existing dam with a low assurance of supply can be used for opportunistic artificial recharge.

Kathu provides an example of how mine water abstracted during the de-watering process can be used to replenish a town’s groundwater supplies.

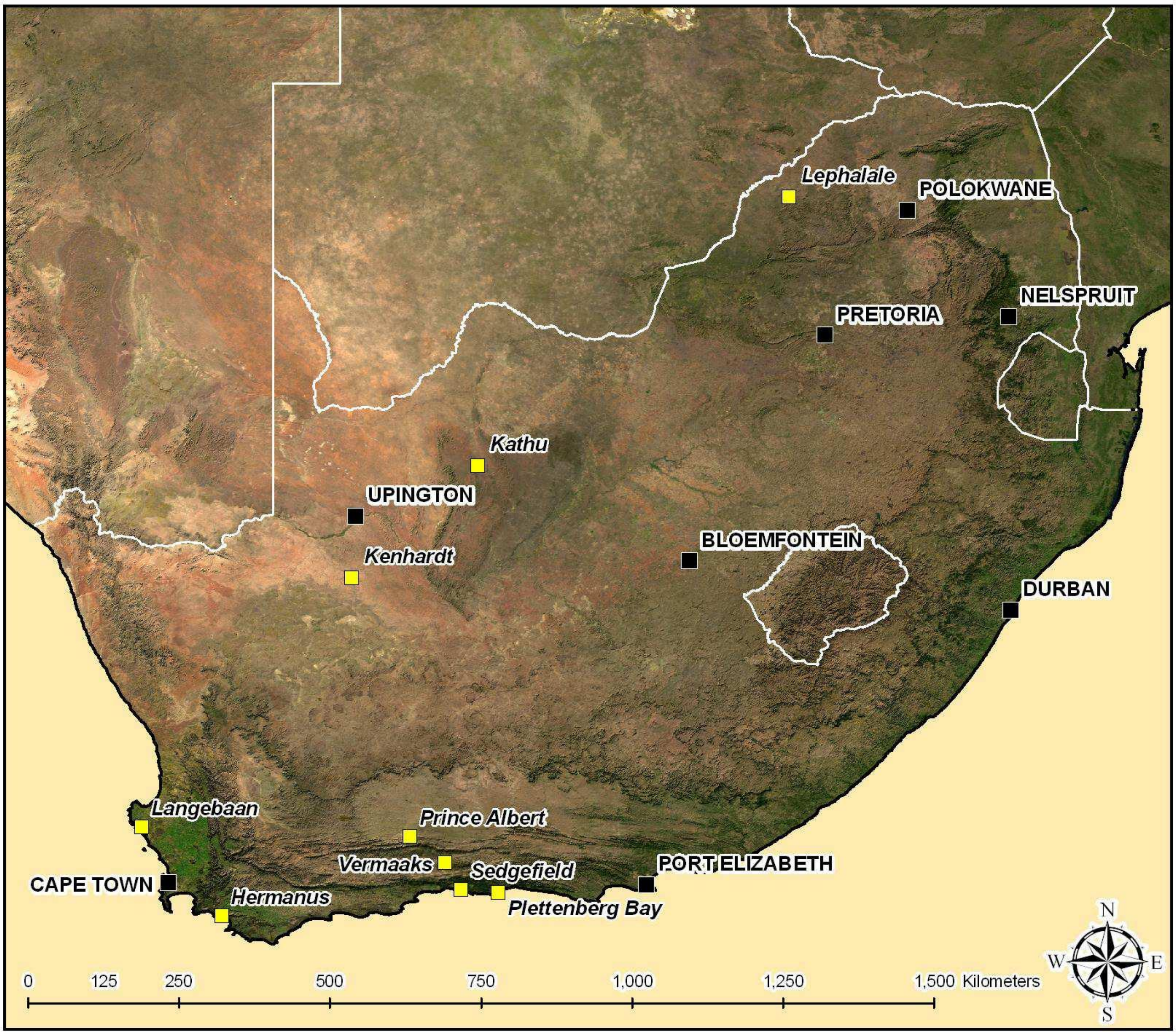


FIGURE 1.1 LOCATION OF ARTIFICIAL RECHARGE TEST SITES

2 BOREHOLE INJECTION IN PRINCE ALBERT

2.1 Introduction

This report follows on from the initial investigation into artificial recharge by Groundwater Africa (Murray, 2007) where most components of a feasibility study were completed. Oustanding components include injection testing in the borehole P5 - P8 area (**Figure 2.1** - **2.4)** with runoff from the Swartberg Mountains. The study was completed in 2010 and the results are presented below together with the remaining components that fulfill the requirements of a feasibility study.

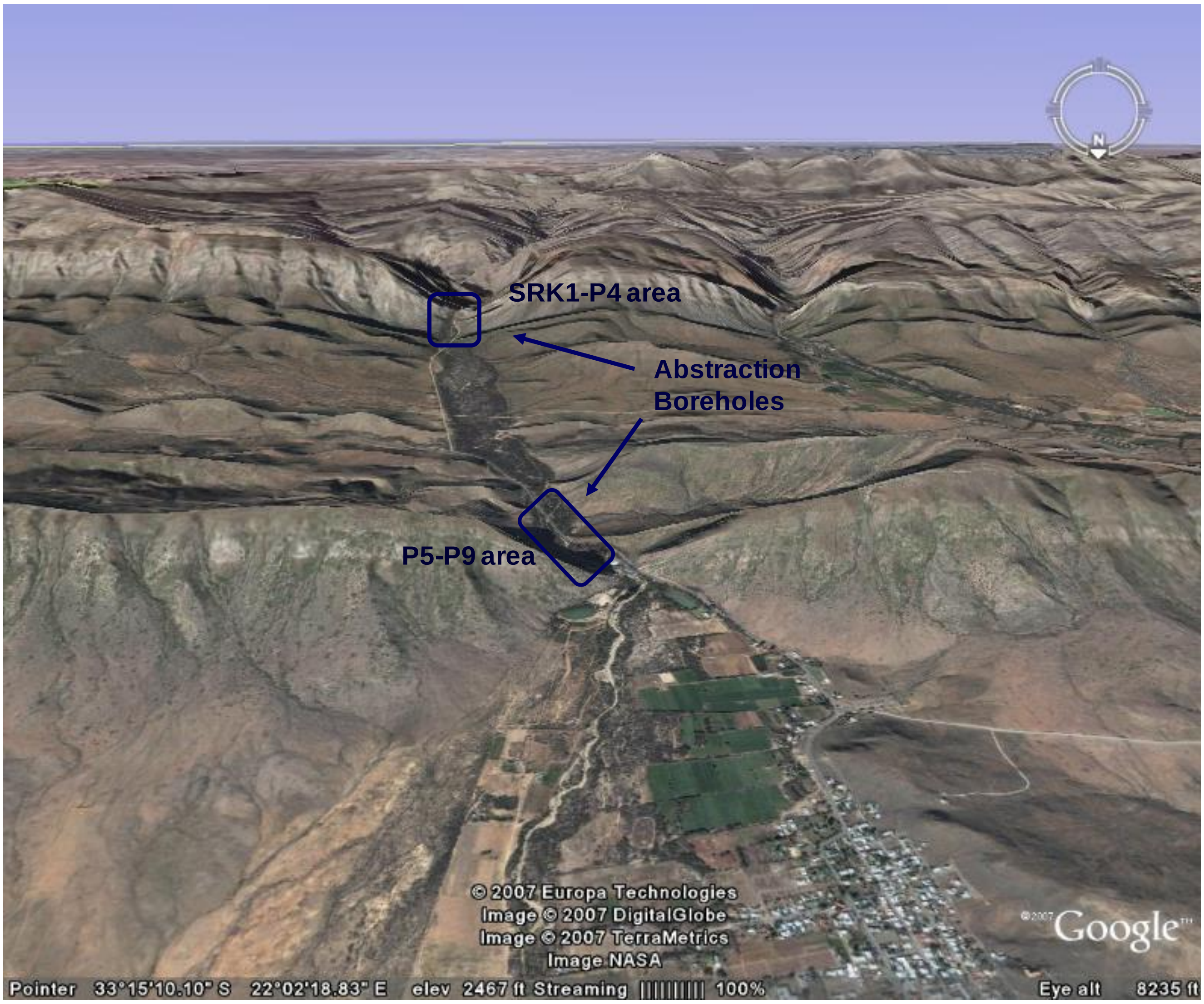


FIGURE 2.1 GOOGLE EARTH IMAGE OF THE WELLFIELDS (GOOGLE EARTH, 2010)

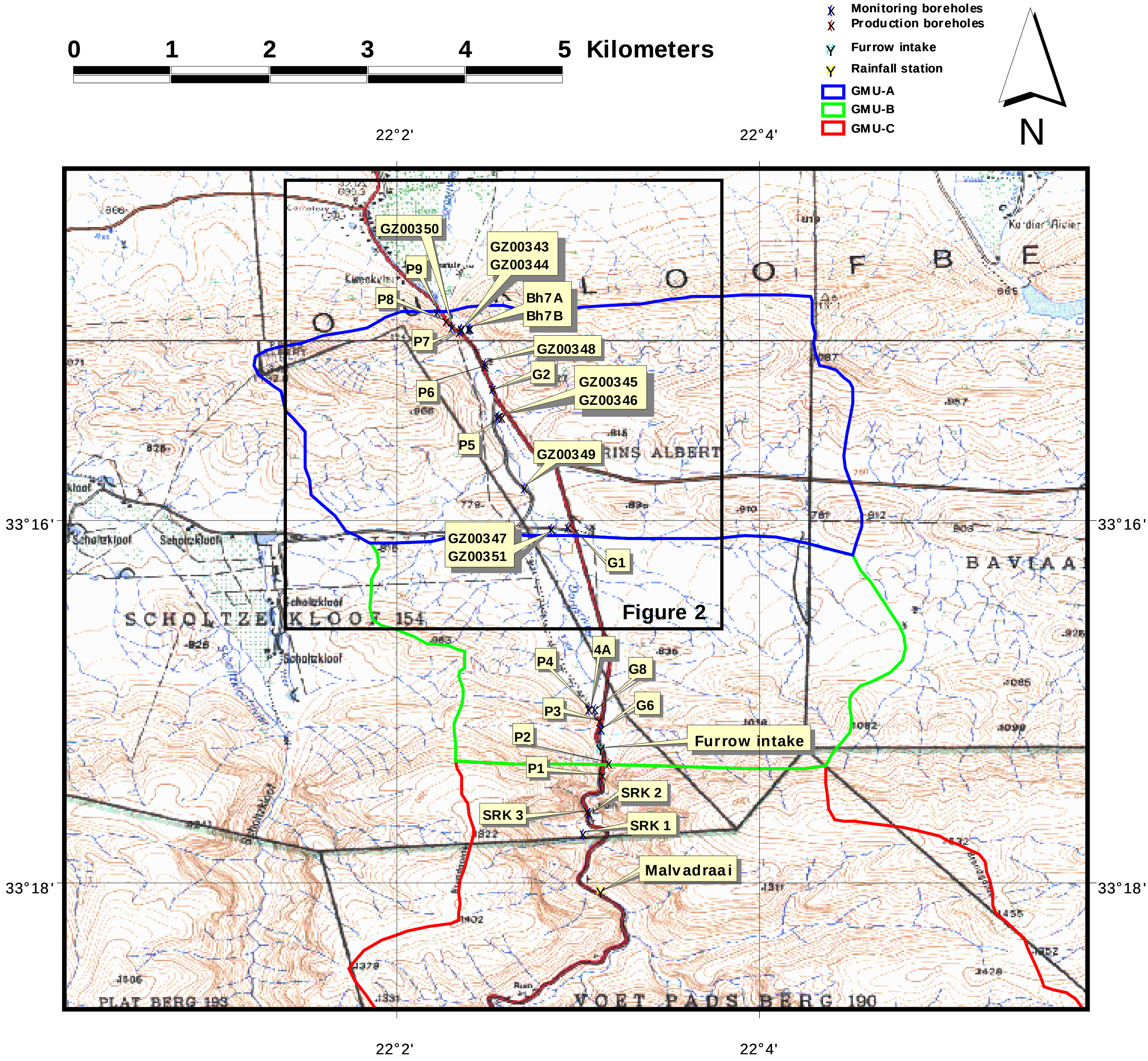


FIGURE 2.2 BOREHOLE LOCATIONS SHOWING THE ARTIFICIAL RECHARGE AREA (P5-P9) FALLS WITHIN GROUNDWATER MANAGEMENT UNIT A (GMU A)

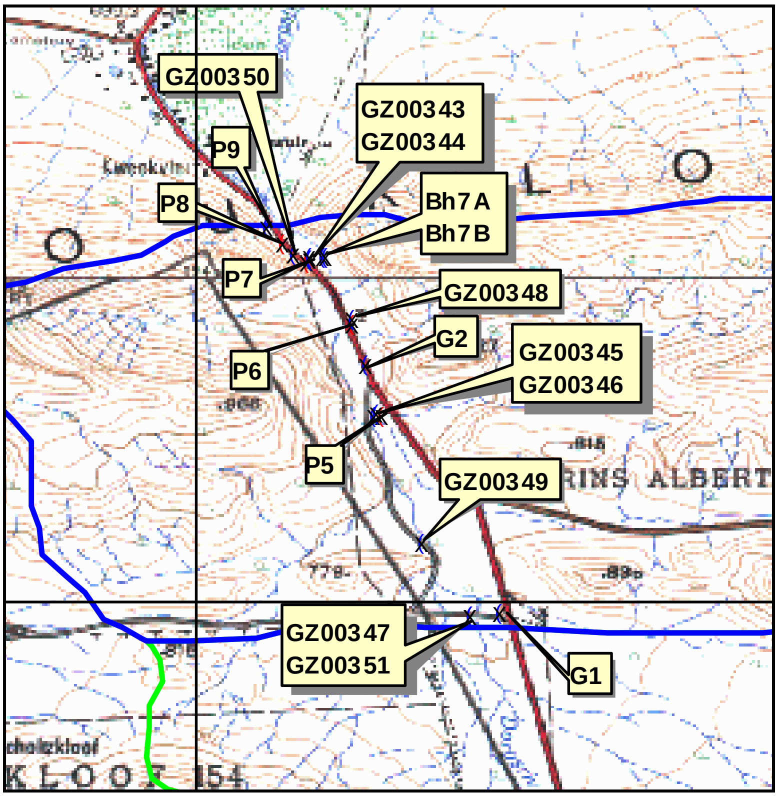


FIGURE 2.3 BOREHOLE LOCATIONS IN THE P5 – P9 AREA

FIGURE 2.4 BOREHOLE LOCATION IN THE P5-P7 AREA (GOOGLE EARTH, 2010)

2.2 The need for artificial recharge

2.2.1 Groundwater's role in assuring summer water security

Groundwater accounts for over 70% of Prince Albert's water requirements, the balance being made up with surface water which is piped into town from an intake immediately below the Swartberg Mountains in the Dorps Rivier (Murray, 2007). The town's water use taken over one year of total supply (2008-2009) prior to the piping of the supply furrow is summarized below and in **Figures 2.5 a & b**.

Total supply

- Total (average): 2 400 m^3/day
- Groundwater (average): ~1 750 m^3/day (estimated 73% of total, Murray, 2007)

Summer supply

- Total (average): 2 800 m^3/ day with daily peak demand up to 3 400 m^3/ day
- Groundwater (average): ~2 400 m^3/ day (estimated at 85% of total, Murray, 2007)

Winter supply

- Total (average): 1 900 m^3/ day
- Groundwater (average): ~990 m^3/ day (estimated at 52% of total, Murray, 2007)

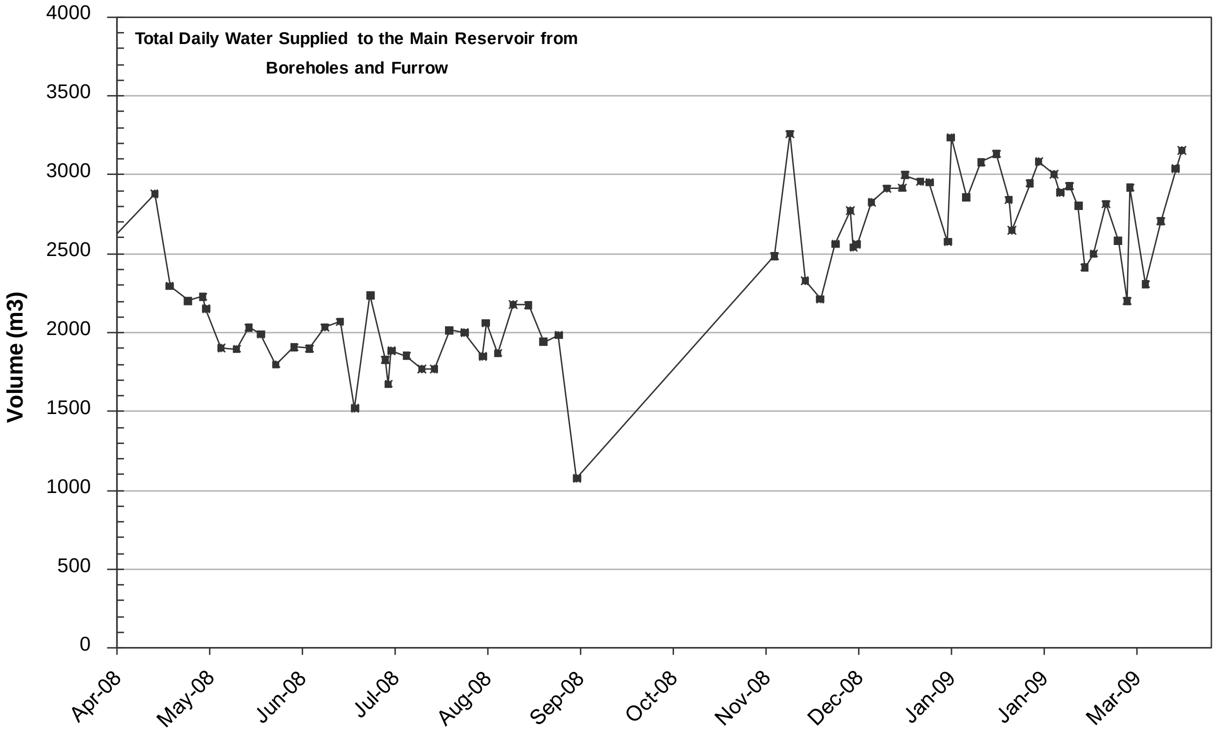


FIGURE 2.5A TOTAL WATER SUPPLY (2008-2009)

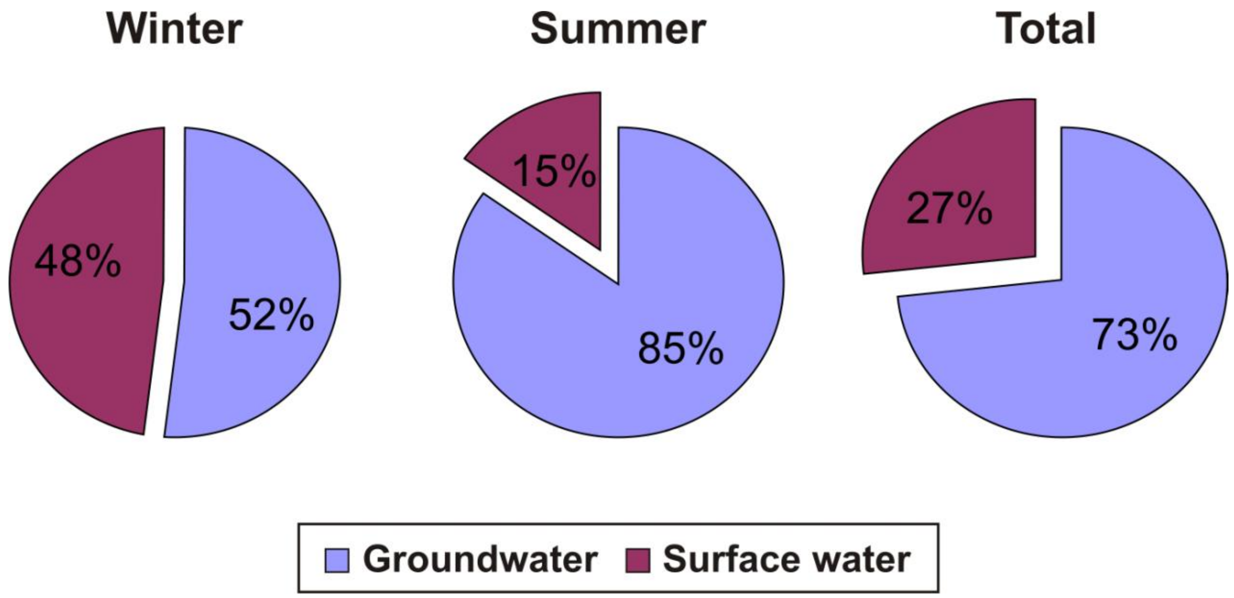


FIGURE 2.5B GROUNDWATER AND SURFACE WATER USE

In 2010 the furrow from the river intake to the water treatment works (at P9) was converted to a pipeline to minimize water losses, and as a result the proportion of surface water available to the town has increased. The figures above are pre-pipeline values. Even with the new pipeline, groundwater will remain the main water source, especially in summer when the surface water flows are low. The security of water supply to the town thus rests on the reliability of groundwater. From historical water level records (Murray, 2007) it is evident that the boreholes located at the base of the Swartberg Mountains (GMUs B & C) receive continuous recharge from the Dorps Rivier, however, the boreholes located closer to town (GMU A) do not have a permanent source of water to recharge the aquifers. The Dorps Rivier is perennial to a variable point below P4 after which it ceases to flow due to evapotranspiration from the dense vegetation along its course. The wellfields located below P4 (i.e. the P5 – P9 areas) are only recharged after heavy rains when the river flows down to these areas. If the river does not flow to these areas because of low rainfall in the catchment, the groundwater levels in these areas may not fill up prior to the onset of summer. For this reason, artificial recharge should be developed as the "back-up" security; to be done if the water levels have not recovered prior to summer.

2.2.2 Groundwater levels without artificial recharge

The natural inflow to the P5 – P7 groundwater compartments is variable according to rainfall and river flow. Since monitoring has taken place in 2006 the river flowed each winter and the aquifers filled up, however, in 2010 this did not happen, and the aquifers or groundwater compartments would not have filled up by December 2010 as can be seen in **Figure 2.6**. This assumes that the storage in the aquifer is constant with depth and that the inflow is constant. The aquifer storage with depth, however, is not constant, and decreases with depth (Murray, 2007) thus it would take much longer than shown in Figure 2.6 for the aquifer to fill if it did not receive any natural recharge. Note that the natural water level rise is not due to natural recharge but rather to lateral inflow to the aquifer, where in this area, the water table is depressed due to abstraction from borehole P5.

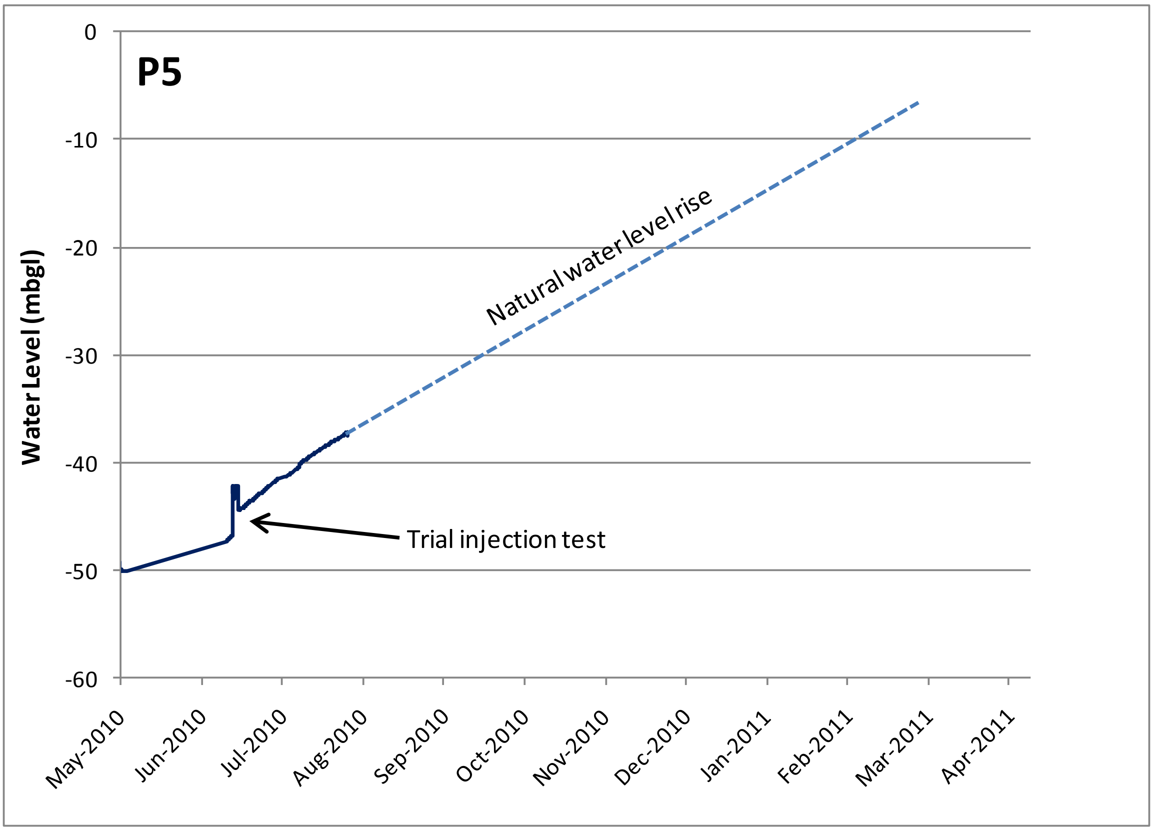


FIGURE 2.6 NATURAL GROUNDWATER LEVEL RISE IN THE P5 GROUNDWATER COMPARTMENT SHOWING THAT WITHOUT ARTIFICIAL OR NATURAL RECHARGE THE AQUIFER WOULD REACHED FULL LEVELS AFTER SUMMER (AQUIFER FULL LEVEL IS 7 MBGL).

In order to ensure the groundwater compartments in the P5 - P9 area are full prior to summer it is necessary to have the ability to artificially recharge them if needs be.

2.3 The quantity and reliability of the water source

Artificial recharge and the filling up of the P5 – P9 groundwater compartments should take place whenever there is surplus surface water and when these compartments are not already full. Water should be available for this at the following times:

i) During the 2 – 4 week annual furrow cleaning period

ii) Whenever the reservoirs are full and the municipality has surplus water available during its "buert" (allocation time)

iii) Whenever other users of the furrow do not need their allocations.

The flow through the new supply pipeline is in the order of 50 L/s. Assuming this flow is available for artificial recharge, there is potential for an artificial recharge capacity of:

- 4.3 Ml/day
- 30 Ml/week
- 130 Ml/month

2.4 Aquifer hydraulics: Can the aquifer receive and store the water?

In order to test the aquifer's ability to receive artificially recharged water, injection tests were carried out on the following boreholes:

- GZ00346 in the P5 compartment
- GZ00350 and GZ00344 in the P7 compartment

2.4.1 The P5 Compartment: Injection test at GZ00346

2.4.1.1 Introduction

The P5 area site layout is shown in **Figure 2.7**.

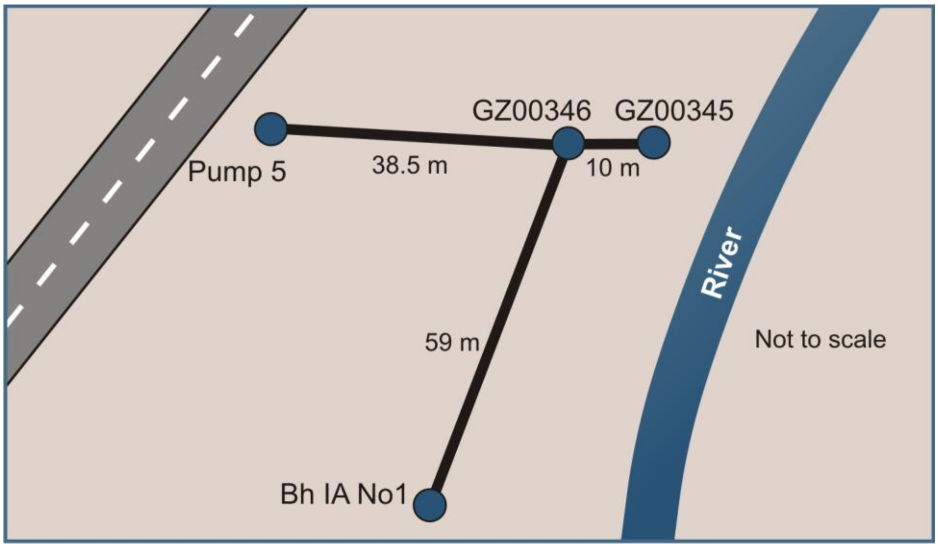


FIGURE 2.7 PUMP 5 SITE MAP

Borehole P5, like GZ00346, had a high drilling yield (**Table 2.1**).

PHOTO 1 & 2: THE BOREHOLE INJECTION RIG USED FOR THE PRINCE ALBERT ARTIFICIAL RECHARGE TESTS

TABLE 2.1 PUMP 5 COMPARTMENT: BOREHOLE CONSTRUCTION

Bh No	*Drilled depth and measured depth in brackets (m)*	*Depth and gross blow yield of water strikes*	*Depth of alluvium (m)*	*Construction*
Pump 5 (PA97/04)	90	12 m – seep 49 m – 2 L/s 57 m – 10 L/s 80 m – 15 L/s 87 m – 25 L/s 89 m – 40 L/s	9	0-12 m: Plain steel casing 12-90 m: Open hole
GZ00345	31	0	10	0-10 m: Plain steel casing 10-31 m: Open hole
GZ00346	100 (99.4)	34 m – 1.3 L/s 57 m – 6.9 L/s 69 m – 31.4 L/s	9	0-45 m: Plain steel casing 45-75 m: Perforated steel casing 75-100 m: Open hole Cement seal was inserted to 21 m to separate the alluvium from the hard-rock. Unsure if this is a true seal.

2.4.1.2 The 9-hour trial injection test on GZ00346 at an average of 18.5 L/s

The first recharge test was conducted on Bh GZ00346 on the 19th June 2010 to test the injection capacity in the Bh 5 area. The test was run for 9 hrs 15 minutes and a total of 621 400 litres was injected at an average rate of **18.5 L/s**. The monitored boreholes include P5, located 38.5 m east of the injection hole, and a newly drilled borehole (IA Bh1), located 59 m north of the injection hole. Bh IA1 is a private borehole drilled by Ian Uys, the owner of the land. Water for the test was diverted from the new pipeline that conveys water from the Dorps Rivier to town. The injection resulted in a 3.2 m water level rise in the aquifer (**Figure 2.8**).

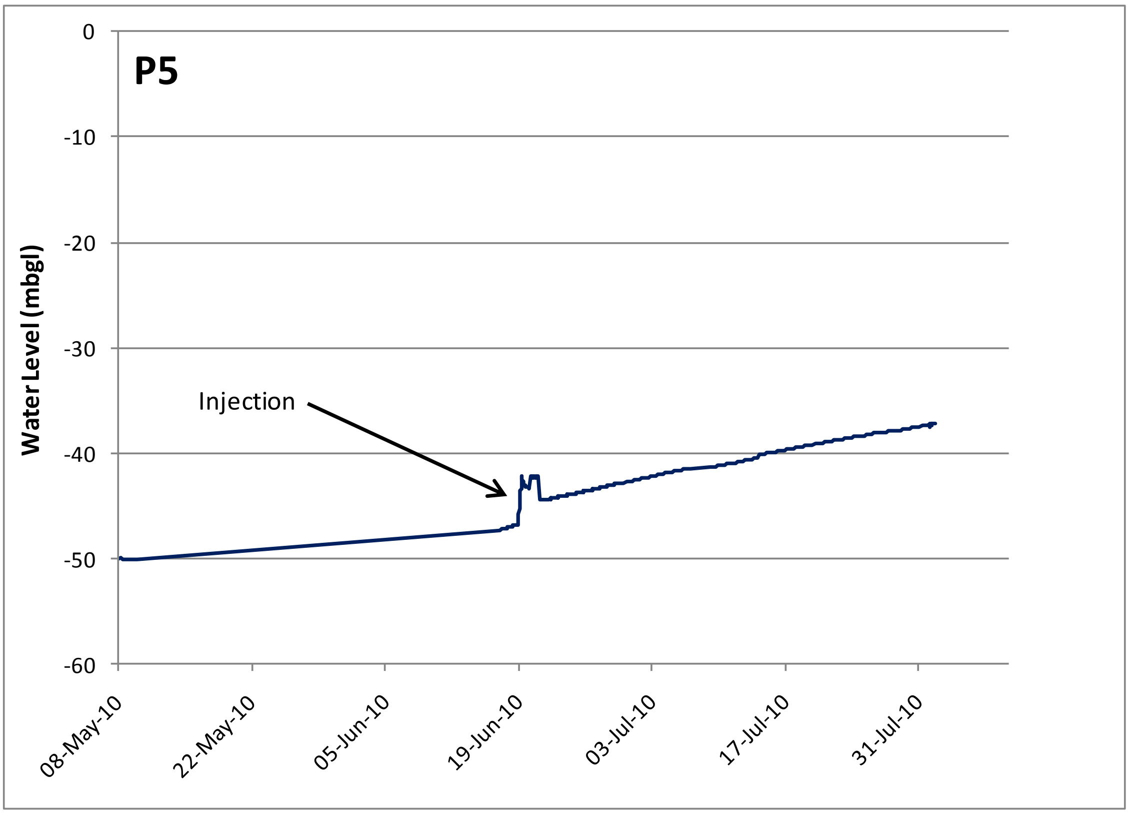


FIGURE 2.8 WATER LEVEL RISE IN BOREHOLE P5 DUE TO INJECTION AT 18.5 L/S IN BOREHOLE GZ00346

Figure 2.8 shows the water level rise (due to borehole injection) in relation to the natural water level rise that was taking place at the time. The injection water level rise translates to a volume of **200 m^3/m** in the aquifer. This reflects the low storage capacity of the shales at this depth.

The estimated abstraction potential from this compartment was determined from water level and abstraction data to be 46 000 m^3 (Murray, 2007). This comprised of:

7- 12 m:	Predominantly alluvium	16 000 m^3	3 200 m^3/m
12 – 49 m:	Predominantly shale and quartzite	30 000 m^3	800 m^3/m

The short-term injection test showed that the storage capacity of the hard-rock formations (200 m^3/m) is a quarter of the abstraction potential (800 m^3/m) of the shales and quartzites, and thus most of the abstracted water comes from inducing flow from further afield. A more realistic storage capacity of the hard-rock formations was obtained from the 7-day injection test, which gave a value of **460 m^3/m.**

2.4.1.3 The 7-day injection test on GZ00346

The second recharge test was conducted on GZ00346 from the 1st to the 8th of August 2010 (**Figure 2.9**) to assess the overall storage capacity of this compartment. The injection rate started at ~35 L/s, but the test was interrupted as water levels neared the surface. The water level decline was assessed when injection stopped. After an 18-hour break, it was re-started at a lower rate (~10 L/s). The average injection rate, taking stoppages into account, was ~20 L/s, and the total volume injected was **10 708 m^3**. The combined artificial recharge volume (from both tests) was 11 330 m^3. The stoppage showed that the upper alluvium had not received water during injection. **Figures 2.8** to **2.13** show the results of the injection tests.

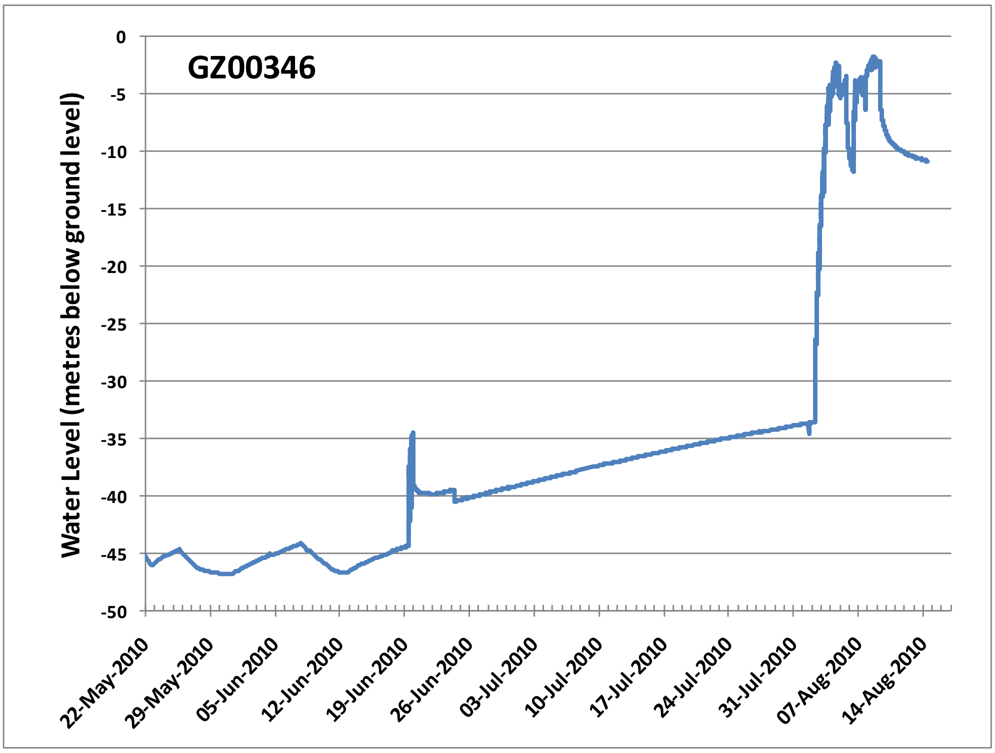


FIGURE 2.9 WATER LEVEL RESPONSE IN GZ00346 DUE TO INJECTION IN GZ00346

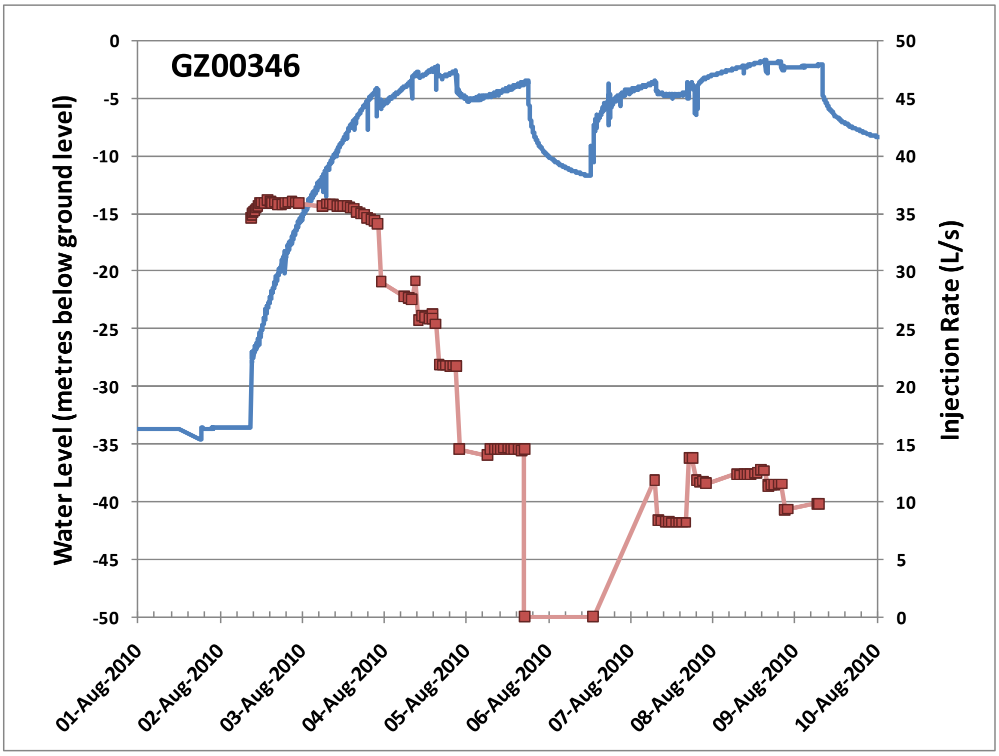


FIGURE 2.10 WATER LEVEL RESPONSE AND INJECTION RATE AT GZ00346

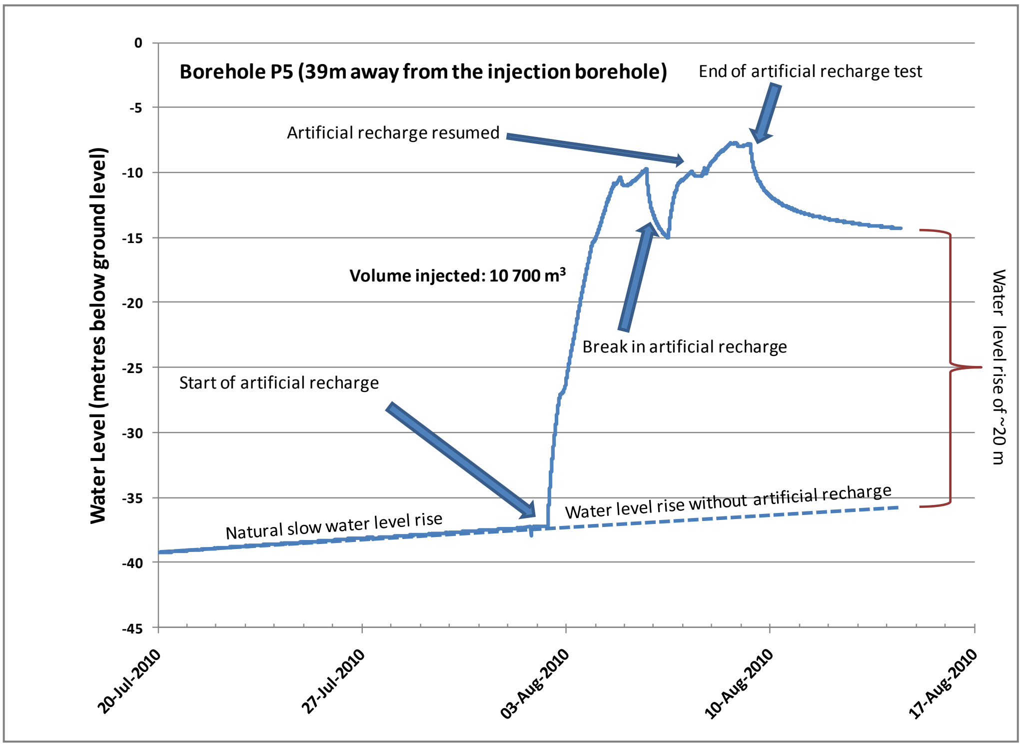


FIGURE 2.11 WATER LEVEL RESPONSE IN P5 DUE TO INJECTION IN GZ00346

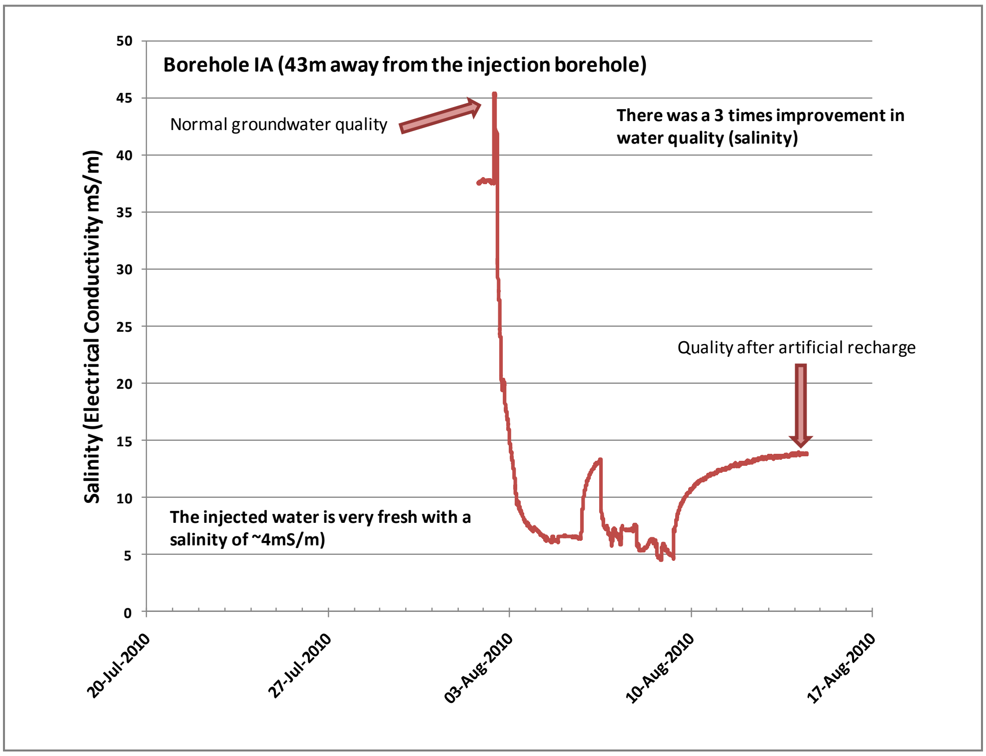


FIGURE 2.12 SALINITY RESPONSE IN BH IA DUE TO INJECTION IN GZ00346

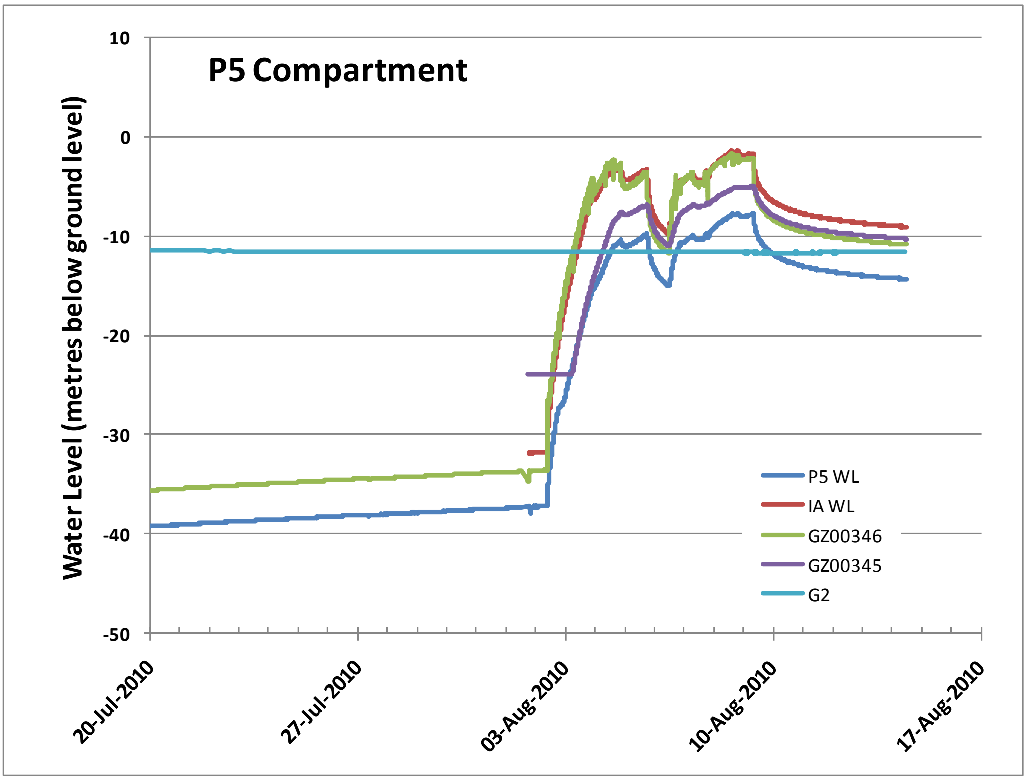


FIGURE 2.13 WATER LEVEL RESPONSE TO INJECTION IN GZ00346

2.4.1.4 Conclusions from the P5-compartment injection tests

Although the water level in the injection hole neared the surface during artificial recharge, the aquifer was not filled by the injection of ~11 000 m^3. It appears as if the artificially recharged water filled the adjacent hard-rocks (the shales and quartzites), but it did not fill the alluvium. At most, this compartment could have received another ~10 000 – 15 000 m^3.

From the 7-day injection test, the storage capacity of the hard-rock formations is in the order of **450 m^3/m**. This is the capacity up to about 12 mbgl. The storage capacity of the alluvium (predominantly) between this level and the rest water level (aquifer full level) of 7 mbgl has not been tested, but the abstraction capacity, which takes both storage and lateral inflow into account, is estimated to be 3 200 m^3/m or 16 000 m^3.

At the current pump intake depth of 50 mbgl, the storage capacity is estimated to be:

7- 12 m:	Predominantly alluvium	16 000 m^3	3 200 m^3/m (includes lateral inflow)
12 – 50 m:	Predominantly shale and quartzite	17 000 m^3	450 m^3/m

Assuming the pump intake of P5 is dropped to 1 m above its deepest water strike (i.e. to a depth of 88 m), the maximum injection capacity of the hard-rock aquifer would be:

12 – 88 m:	Predominantly shale and quartzite	34 000 m^3	450 m^3/m

If we assume the storage capacity of the alluvium is in the order of 10 000 m^3, then the injection capacity of this area is estimated to be 30 000 m^3 – 45 000 m^3 depending on the level to which the water is initially drawn down. Because there will always be a significant component of lateral inflow after the pump has been shut down, an injection capacity of **30 000 m^3** should be designed for.

The key design recommendations are:

- The designed maximum injection rate should be **30 L/s** at GZ00346 (with the longer term injection rate dropping to ~10 L/s).
- The design should cater for injection into recharge wells, pits or trenches around GZ00346 should this borehole's efficiency decrease markedly by clogging with time.

2.4.2 The P7 Compartment: Injection tests on GZ 00350 and GZ 00344

2.4.2.1 Introduction

The P7 area site layout is shown in **Figure 2.14**, and their drilling yields and constructions are shown in **Table 2.2**.

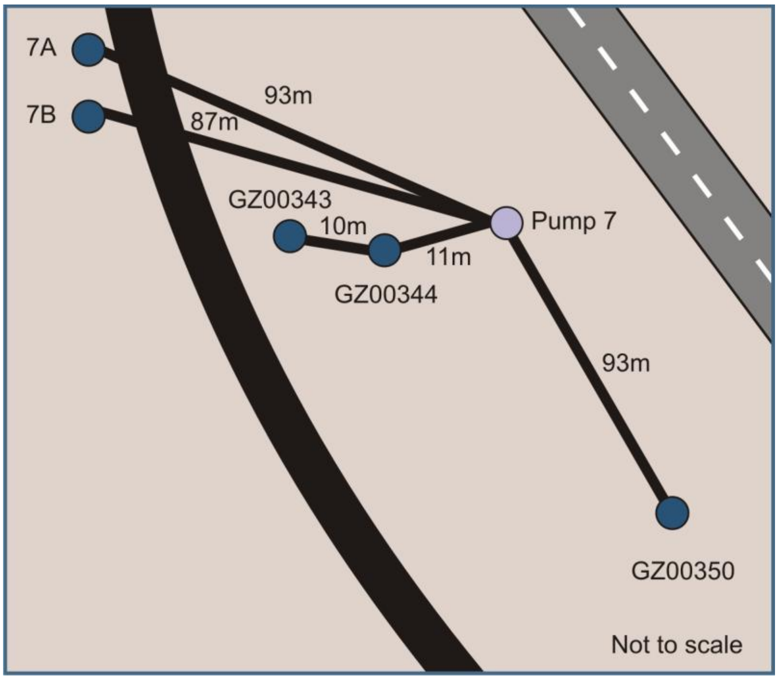


FIGURE 2.14 PUMP 7 COMPARTMENT SITE MAP

TABLE 2.2 P7 COMPARTMENT: BOREHOLE CONSTRUCTION

Bh No	*Drilled depth and measured depth in brackets (m)*	*Depth and gross blow yield of water strikes*	*Depth of alluvium (m)*	*Construction*
Pump 7 (PA97/01)	137	12 m – 2 L/s 17 m – 3.5 L/s 105 m – 25 L/s	12	0-12 m: Plain steel casing 12-17 m: Perforated steel casing
GZ00343	30	0	10	0-10 m: Plain steel casing 10-30 m: Open hole
GZ00344	91 (86.6)	38 m – seep 62 m – 1.8 L/s 81 m – 3.4 L/s 89 m – 10.2 L/s	10	0-55 m: Plain steel casing 55-91 m: Perforated steel casing Cement seal was inserted to 21 m to separate the alluvium from the hard-rock. Unsure if this is a true seal.
GZ00350	150 (138.2)	71m – 2.6 L/s 92 m – 20.1 L/s	10	0-54 m: Plain steel casing 54-93.5 m: Perforated steel casing 93.5-150: Open hole Cement seal was inserted to 21 m to separate the alluvium from the hard-rock. Unsure if this is a true seal.
7A	unknown	unknown	unknown	unknown
7B	unknown	unknown	unknown	unknown

Short-term injection tests were conducted on boreholes GZ00350 near the Water Treatment Works (and near borehole P8), and GZ00344 near borehole P7. The volumes injected were:

- GZ00350: 609.5 m^3 over 27 hours (average 6.3 L/s)
- GZ00344: 317.4 m^3 over 9 hours 27 minutes (average 9.3 L/s) and 2 625 m^3 over 4 days and 2 hours.

2.4.2.2 GZ00350: The short-term injection test at a maximum rate of 15.7 L/s

An injection test on GZ00350 was conducted over a period of 27-hours starting on the 26th June 2010 to establish the injection capacity of this borehole. The maximum injection rate was 15.7 L/s. The test was stopped a number of times to clean leaves from the filter. This problem was solved after this test when a new, easily cleaned filter was obtained. The water level response to injection is shown in **Figures 2.15** and **2.16**.

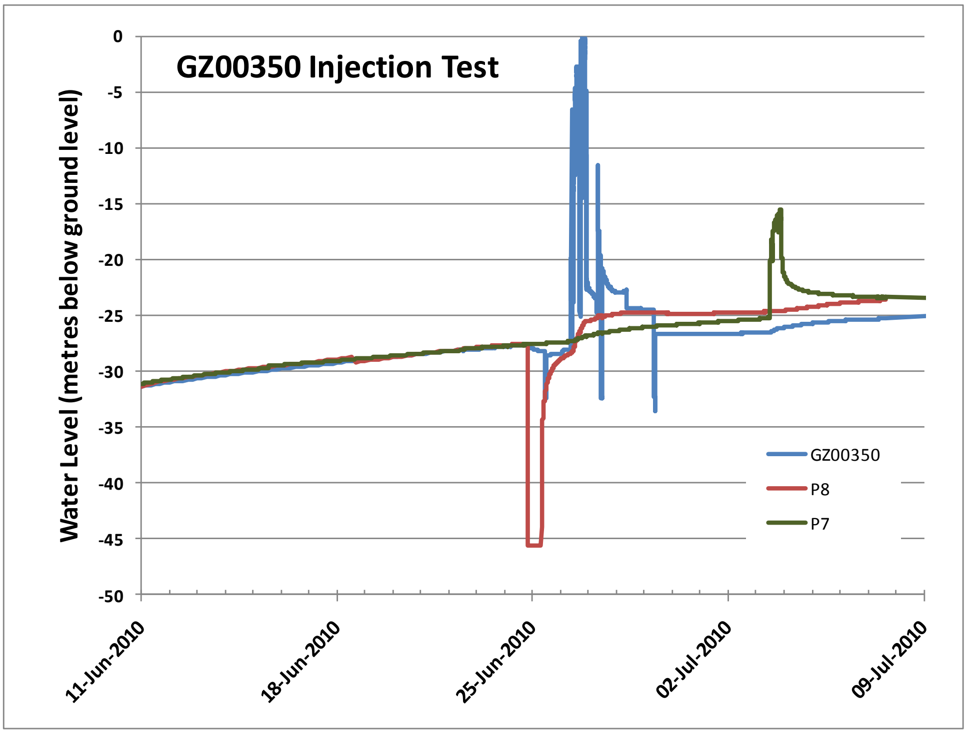


FIGURE 2.15 WATER LEVEL RESPONSE TO INJECTION OF 610 M³ IN GZ00350

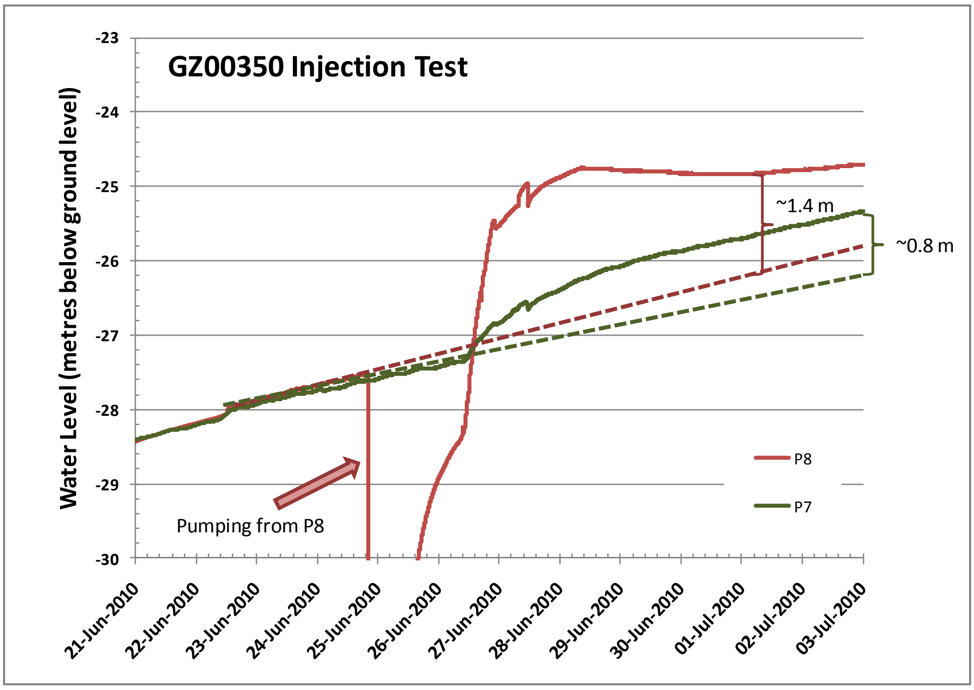


FIGURE 2.16 WATER LEVEL RESPONSE IN P7 & P8 TO INJECTION IN GZ00350

A rough assessment of the storage capacity of the quartzites of this area is:

- P7 water level response: ~760 m^3/m
- P8 water level response: ~440 m^3/m

From previous abstraction data the groundwater yield estimates from the hard-rock parts of the aquifer were estimated to be 800 m^3/m and 500 m^3/m for the P7 and P8 areas respectively (Murray, 2007). These figures included storage and lateral inflow.

2.4.2.3 GZ00344 Injection tests

A 9.5-hour injection test was conducted on the 3rd July 2010 on GZ00344 to establish the injection capacity of this borehole. A total of 317.4 m^3 was artificially recharged at an average injection rate of 9.3 L/s. The water level response to injection is shown in **Figure 2.17**.

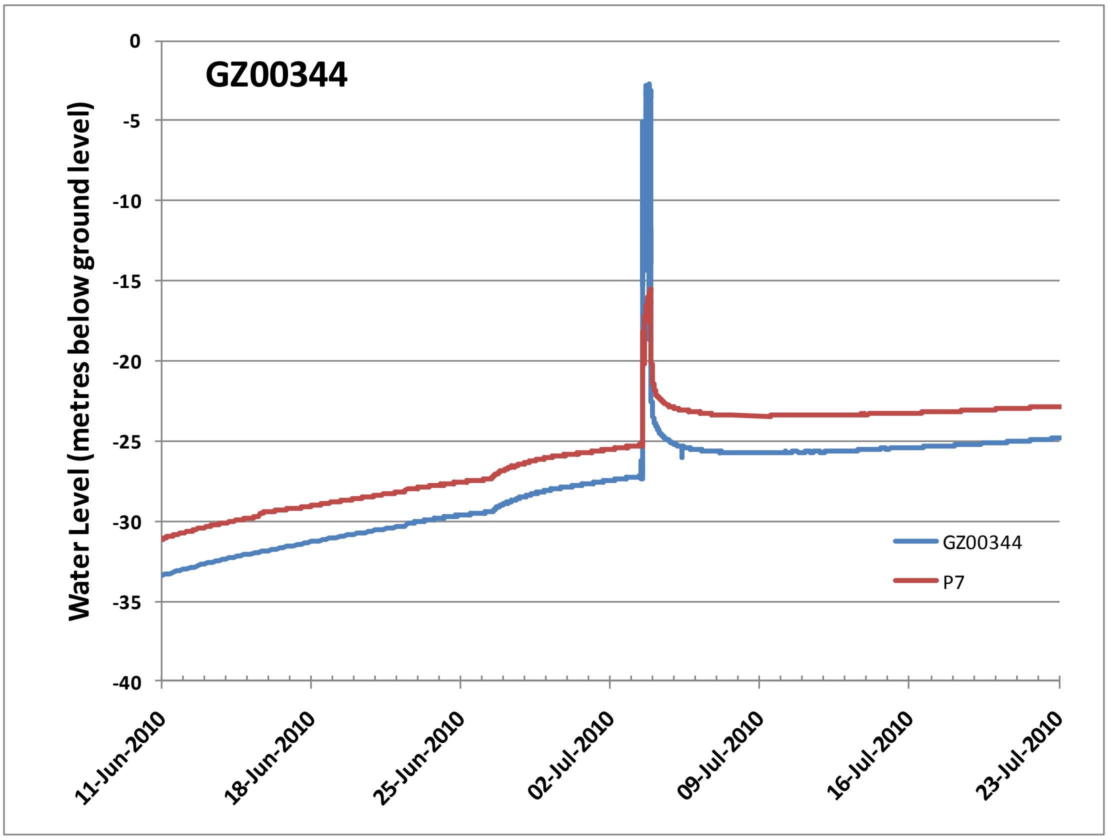


FIGURE 2.17 WATER LEVEL RESPONSE IN P7 TO INJECTION IN GZ00344

The short test showed the limited capacity of the borehole to receive artificially recharged water. This test was followed by 4-day injection test (9-13 August 2010) at an average of 7.4 L/s, where a total of 2 625 m^3 was injected (**Figure 2.18**).

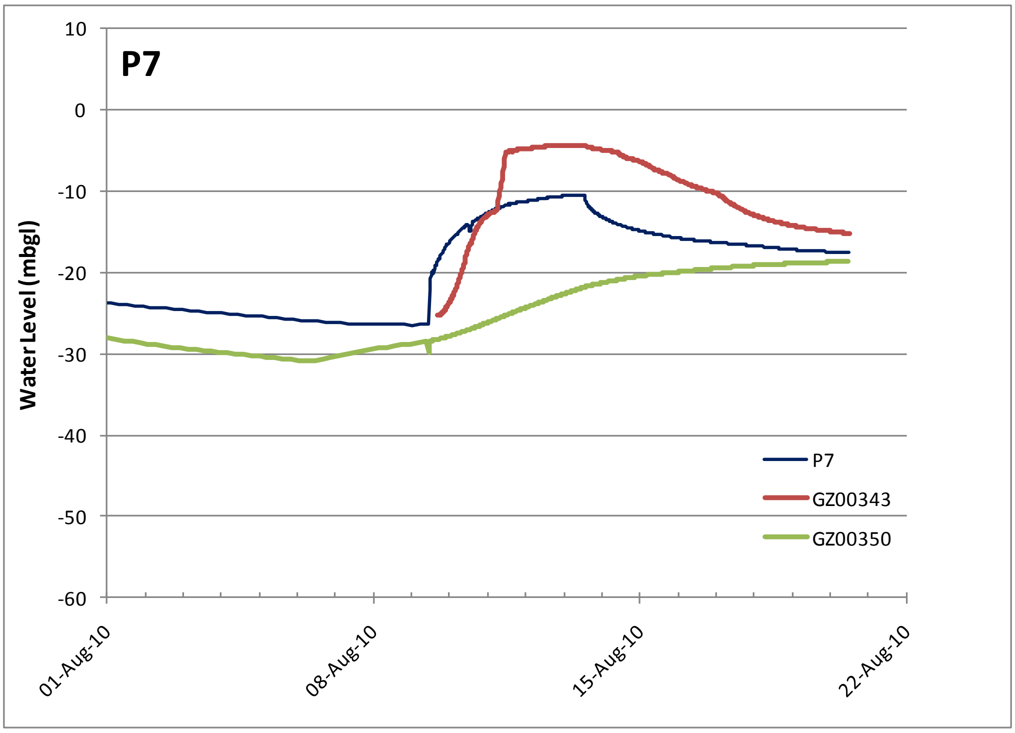


FIGURE 2.18 WATER LEVEL RESPONSE IN SURROUNDING BOREHOLES TO 4-DAYS OF INJECTION AT GZ00344

The injection flow rate was monitored during this test (**Figure 2.19**).

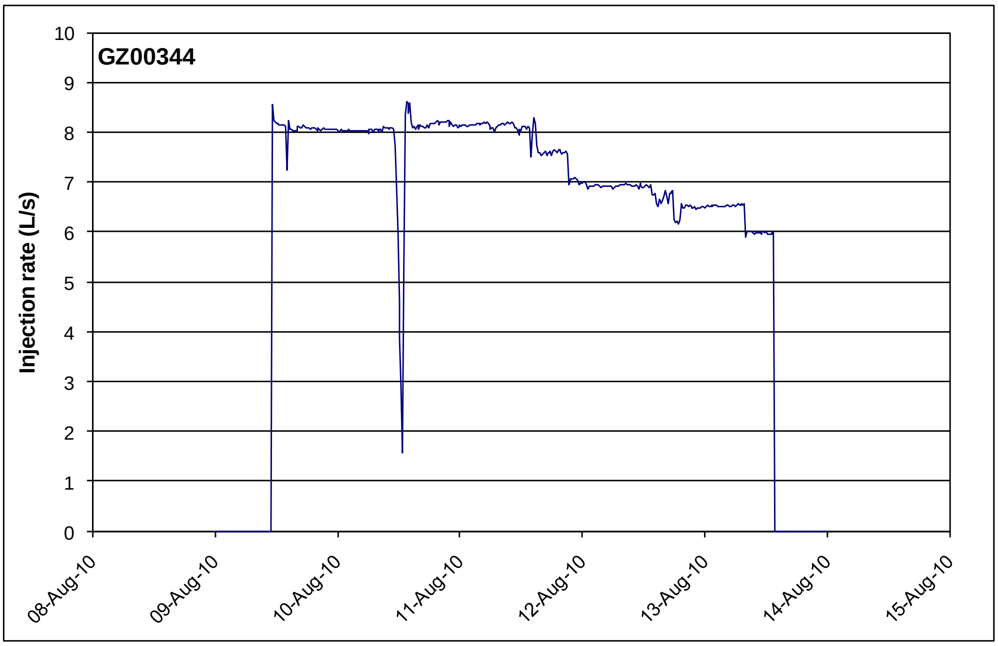


FIGURE 2.19 INJECTION RATE AT GZ00344

The storage capacity of the hard-rock formations from the 4-day injection test is estimated to be about **330 m^3/m**.

2.4.2.4 Conclusions from the P7 compartment injection tests

Although the groundwater compartments are interconnected, the P7 compartment can be considered to be the area that contains the P7 and P8 boreholes. From the 4-day injection test, the storage capacity of the hard-rock formations is in the order of **320 – 760 m^3/m**, with 320 m^3/m probably lying closer to reality. This value is the capacity up to about 5 mbgl. The storage capacity in the alluvium (predominantly) between this level and the rest water level (aquifer full level) of 2 mbgl has not been tested, but the abstraction capacity, which takes both storage and lateral inflow into account is estimated to be 3 200 m^3/m or 10 600 m^3.

At the current pump intake depth of 51 mbgl, the storage capacity is estimated to be:

2 - 5 m:	Predominantly alluvium	~10 000 m^3	3 200 m^3/m (includes lateral inflow)
5 – 51 m:	Predominantly shale and quartzite	~15 000 m^3	330 m^3/m

Assuming the pump intake of P7 is dropped to 1 m above its deepest water strike (i.e. to a depth of 104 m), and that the storage capacity remains even with depth, then the maximum injection capacity of the hard-rock aquifer would be:

5 – 104 m:	Predominantly shale and quartzite	~32 000 m^3	330 m^3/m

If we assume the storage capacity of the alluvium is in the order of 10 000 m^3, then the injection capacity of this area is estimated to be 25 000 m^3 – 42 000 m^3 depending on the level to which the water is initially drawn down. Because there will always be a significant component of lateral inflow after the pump has been shut down, an injection capacity of ~**30 000 m^3** should be designed for.

The key design recommendations are:

- The designed injection rate should be:
  - **8 L/s** (maximum) at GZ00344 (with the longer term injection rate dropping to ~6 L/s)
  - **16 L/s** (maximum) at GZ00350 (with the longer term injection rate dropping to ~10 L/s)
- The design should cater for injection into recharge wells, pits or trenches around P7 and GZ00350 should these boreholes' efficiencies decrease markedly by clogging with time.

2.4.3 Summary of the artificial recharge capacity

The abstraction capacity from the P5 and P7 groundwater compartments are in the order of 100 000 m^3, and the injection capacities about 60 000 m^3 (**Table 2.3**). The total injection capacity of the three boreholes (from the injection tests) is considered to be in the order of 36 – 56 L/s (**Table 2.3**).

TABLE 2.3 ARTIFICIAL RECHARGE CAPACITY

Groundwater compartment	Artificial recharge capacity - maximum (L/s)	Artificial recharge capacity – average (L/s)	Artificial recharge capacity (m^3)	Abstraction capacity (m^3) with current pump in take depths (~50 mbgl)*
P5 compartment	30	20	30 000	46 000
P7 compartment	24	16	30 000	56 000
Total	**54**	**36**	**60 000**	**102 000**

*Based on the summer of 2007 when water levels in the aquifer was heavily drawn down

2.5 Water Quality

2.5.1 Introduction

The groundwater quality of the P5 and P7 compartments are described in Murray (2007). The 2010 injection tests provided further information on the behaviour of the two ions of greatest concern, namely iron and manganese.

The presence of appreciable concentrations of iron and manganese is a general feature of groundwater in the Prince Albert well field area (**Table 2.4**). The data in **Table 2.4** represent "total" iron and manganese, i.e. both dissolved and particulate iron and manganese.

TABLE 2.4 IRON AND MANGANESE CONCENTRATIONS IN GROUNDWATER AT PRINCE ALBERT

Source	Parameter	Minimum	Maximum	Average	Median	N
TMG – Tchando Formation						
SRK1	Fe	10.00	10.00	10.00	10.00	1
	Mn	1.04	1.04	1.04	1.04	1
SRK3	Fe	2.10	2.10	2.10	2.10	1
	Mn	<0.05	<0.05	<0.05	<0.05	1
P1	Fe	0.05	0.05	0.05	0.05	1
	Mn	<0.05	<0.05	<0.05	<0.05	1
Bokkeveld Group – Traka Formation						
P2	Fe	0.20	0.20	0.20	0.20	1
	Mn	n.a.	n.a.	n.a.	n.a.	1
P3	Fe	0.05	0.05	0.05	0.05	1
	Mn	0.08	0.08	0.08	0.08	1
P4	Fe	0.42	0.42	0.42	0.42	1
	Mn	0.14	0.14	0.14	0.14	1
Witteberg Group – Weltevrede Formation						
P5	Fe	0.07	0.12	0.10	0.10	4
	Mn	0.05	0.91	0.35	0.08	4
GZ00346	Fe	0.01	0.03	0.02	0.02	10
	Mn	0.00	0.13	0.04	0.02	10
P6	Fe	0.05	0.65	0.35	0.35	3
	Mn	0.09	0.68	0.29	0.10	3

Source	Parameter	Minimum	Maximum	Average	Median	N
Witteberg Group – Kweekvlei or Witpoort Formations						
P7	Fe	0.08	10.00	3.01	1.27	5
	Mn	0.17	1.10	0.51	0.39	5
GZ00343	Fe	<0.01	<0.01	<0.01	<0.01	1
	Mn	0.08	0.08	0.08	0.08	1
GZ00344	Fe	0.01	0.04	0.02	0.02	10
	Mn	0.52	1.28	1.11	1.15	10
GZ00350	Fe	0.01	0.11	0.04	0.04	9
	Mn	0.20	0.43	0.36	0.40	9
P8	Fe	0.09	5.20	2.98	3.66	3
	Mn	0.21	0.30	0.26	0.26	3
Spring/surface water						
Furrow	Fe	0.03	0.26	0.12	0.10	4
	Mn	<0.01	<0.01	-	-	4

Sampling groundwater for iron and manganese needs special precautions to obtain values representative of the iron and manganese concentrations in the aquifer. Apart from the fact that boreholes may have steel casings that contribute to the presence of dissolved and particulate iron in the groundwater, redox conditions in the aquifer determine the solubility of the two elements. During pumping, redox conditions change as aeration takes place and water bodies in the aquifer are displaced. In addition, deposits that settled out in the borehole, the well screen, or gravel pack if present, are mobilised when pumping starts and can yield very high concentrations which are not representative of the conditions in the aquifer. The 10 mg/L found at the SRK1 borehole or borehole P7 (**Table 2.4**) may be a case in point. Furthermore, dissolved iron (and manganese) may also precipitate in the sample bottle after sampling and yield an incorrect analytical result. For this reason filtration of an aliquot at the time of sampling is recommended with submission of both filtered and unfiltered samples to the laboratory. Generally the samples are not filtered in the field and at best a total iron concentration is obtained when acidifying an aliquot in the laboratory.

The table shows that certain formations (e.g. those of the Witteberg Group) possibly yield higher manganese levels while in other cases both iron and manganese may be low. The spring water in the furrow does not have manganese levels above the detection limit (0.01 mg/L). Hence, the manganese found in the groundwater both before and after injection has to originate from the aquifer matrix.

2.5.2 Injection test

In an attempt to relate the presence of iron and manganese in the groundwater with the abstraction-injection regime at borehole P5 a series of samples were procured from the borehole before and after injection. After the brief injection test on the 19th June 2010 and the longer test from the 3rd – 9th August 2010, abstraction resumed (in September) and one sample was obtained on the 25th October during abstraction. The analytical results for iron and manganese are shown in **Table 2.5** and presented graphically in **Figure 2.20**.

TABLE 2.5 BOREHOLE P5 IRON AND MANGANESE CONCENTRATIONS BEFORE AND AFTER THE INJECTION RUN

Date	Fe total mg/L	Fe dissolved mg/L	Mn total mg/L	Mn dissolved mg/L
27-Jul-10	0.85	<0.01	0.08	0.08*
01-Aug-10	0.34	0.09	0.04	0.04*
20-Aug-10	0.17	0.02	0.03	0.03*
09-Sep-10	0.03	0.03	0.82	0.82
25-Oct-10	0.03	0.01	0.73	0.72

*Note: levels estimated due to contamination from sample bottles

For this series of tests borehole P5 was pumped for 30 minutes and the samples filtered at the well head. Both the filtered and unfiltered samples were submitted to the laboratory. In the initial stages a large part of the iron was in particulate form which was removed by filtration. The dissolved iron never exceeded 0.1 mg/L. Each subsequent time samples were procured the quantity of particulate iron decreased and from 9 September 2010 the dissolved iron virtually equalled the total iron. The important conclusion is that particulate iron collects in the borehole and this takes several hours' pumping to be removed. It is concluded that the "dissolved" iron closely represents the iron concentration in the aquifer. These data are also in agreement with the historic data for borehole P5 (**Table 2.5**).

The analytical data showed that manganese was higher in some of the filtered samples than in the unfiltered ones. As that is impossible the problem was traced to the use of 50 mL plastic bottles which were tinted brown. These were new bottles procured from the pharmacy and tests confirmed that the samples leached manganese from the brown bottles. As the values for "total" and "dissolved" manganese were practically the same it can be safely assumed that the manganese was present in solution. This has implications for the removal process as the levels are well above the limits acceptable for drinking water.

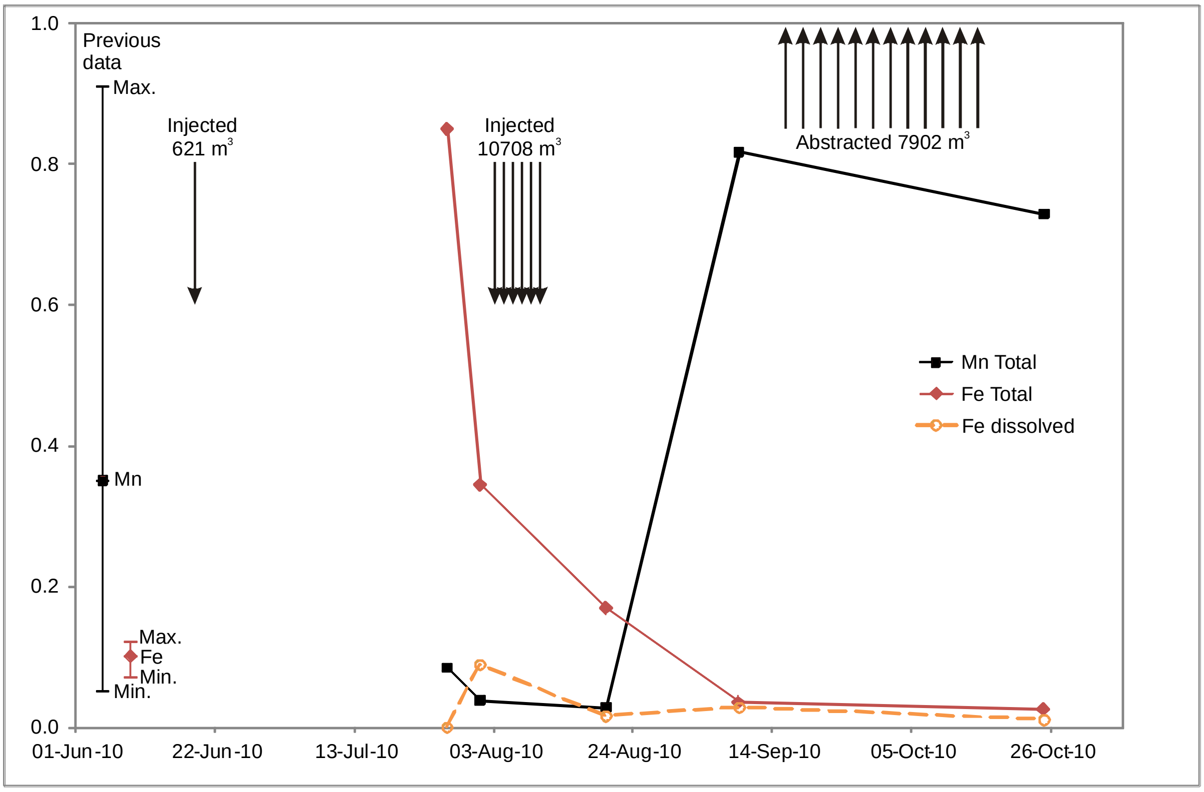


FIGURE 2.20 BOREHOLE P5 TOTAL IRON AND MANGANESE CONCENTRATIONS BEFORE AND AFTER INJECTION TESTS

On the left of **Figure 2.20**, the graph shows the historical average values for iron and manganese in 2005 to 2007. Whereas the total iron remained in the order of 0.1 mg/L the manganese concentration varied over a wide range reaching a maximum of 0.9 mg/L. The initial brief injection test, which was aimed at testing the infrastructure, took place on 19 June 2010 when 621 m^3 was injected in just over 9 hours. At that stage the water level was more than 40 m below surface and it is possible that particulate iron was dislodged by the injected water. It is possible that changes in redox conditions also affected the iron and manganese solubility in the aquifer. Until the second sampling on 1 August, i.e. just before the main injection test (represented by the arrows pointing downwards) commenced on 3 August, the (particulate) iron concentration decreased significantly. After the injection run, dissolved iron decreased and remained very low during the subsequent abstraction period (represented by the arrows pointing upwards) from 14 September onwards.

The cause of the high manganese concentrations from September onwards is not evident at this stage but it could be due to changes in the pH-Eh relationships in the aquifer. The historical data for borehole P5 (**Table 2.5** and **Figure 2.20**) also showed such a high value and an inspection of the raw data shows that this represented dissolved manganese. More detailed information is required to link these high concentrations to redox changes in the aquifer. It will also be necessary to continue monitoring the manganese concentration in order to establish whether it will eventually return to background levels as in August 2010.

It will be necessary to repeat the observations during a subsequent injection run in order to confirm whether the increase in dissolved manganese can be related to the effect of the injection on the pH-Eh relationships in the aquifer and perhaps the effect of the rising water table during injection. The rising water table will allow the dissolution of salts, including manganese compounds precipitated in the unsaturated zone as the water level declined.

2.5.3 Equilibrium diagrams for minerals

Activity diagrams using pH and Eh as variables and assumed concentrations for other constituents serve to illustrate the ranges of pH and Eh over which the various species of iron and manganese can be expected. Two examples of such diagrams are shown below (**Figure 2.21)**.

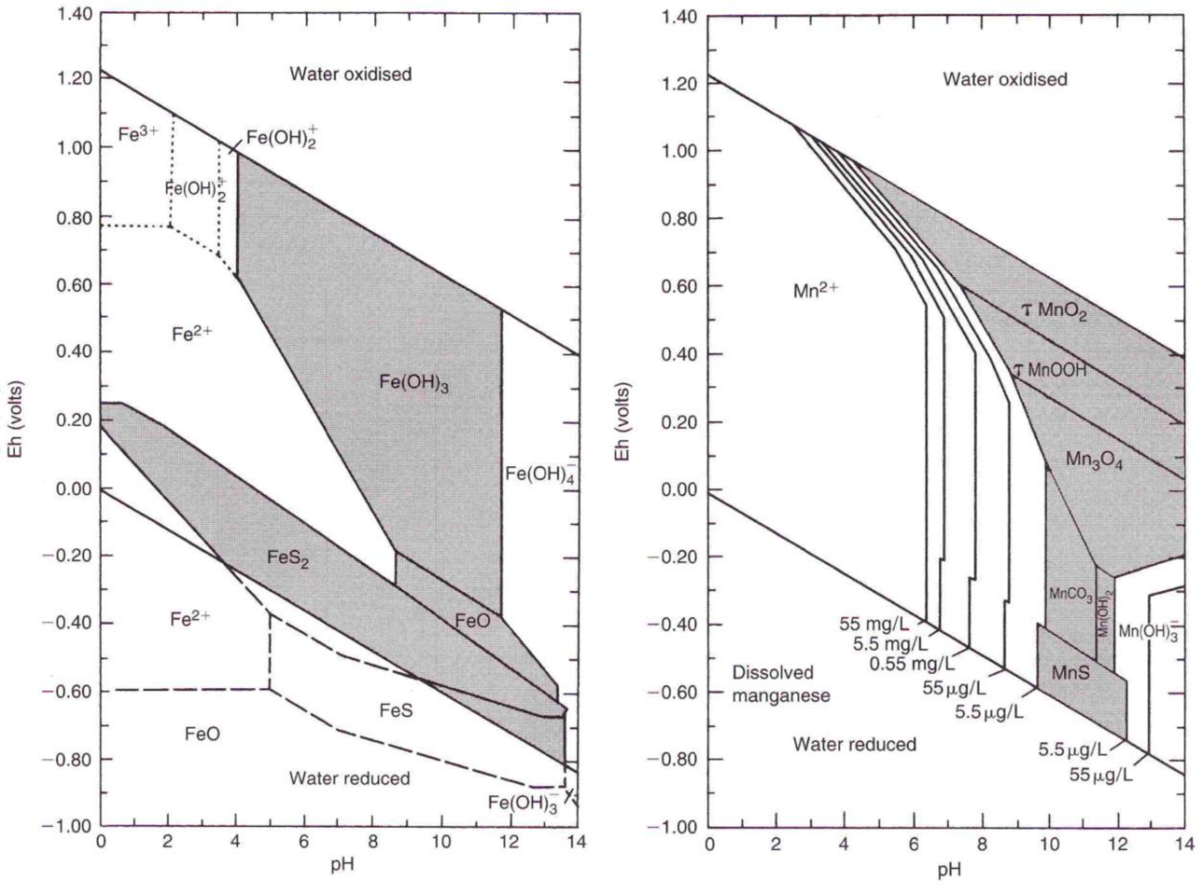


FIGURE 2.21 STABILITY FIELDS FOR SOLID AND DISSOLVED SPECIES OF IRON AND MANGANESE WITHIN THE INDICATED PH AND EH RANGES *(AFTER HEM, 1970)*

Each boundary in the diagrams is governed by a chemical equation describing the reaction for the conversion of the various compounds. These boundaries are affected by the concentrations of the ions and other compounds and the diagrams need to be compiled specifically for the specific groundwater composition, taking the solid phases and partial pressures of dissolved gases into account. The diagrams above are only included for the purpose of illustrating the complexity of the systems and to some extent to show that dissolved Mn^{2+} is stable over a wide pH-Eh range whereas for Fe^{2+} the ranges are limited. This affects the removal possibilities of iron and manganese from groundwater.

2.5.4 Conclusions

The historical data for the Prince Albert well field boreholes confirm that iron and manganese occur in the aquifer matrix and these constituents need special consideration in the aquifer management regime.

The injection test provided some insight into the water quality changes that may occur during and after injection of aerated surface water into the aquifer. However, due to the natural conditions in the aquifer and the large volumes of water involved, a short term pilot test will mainly provide data on the hydraulic feasibility of the technique. A reliable evaluation of hydrochemical interactions needs longer term injection runs with detailed monitoring.

The fact that high (total) iron concentrations were observed at a certain stage points towards the possibility that particulate iron was present in the aquifer after the water level dropped during abstraction. The introduction of oxygen would assist in the precipitation of oxidised iron. Some of these precipitates may have been mobilised by the short term injection test and appeared as particulate iron during the first sampling. At each subsequent sampling the proportion of particulate iron decreased as this was removed from the borehole and its surroundings.

The dissolved iron increased slightly before the main injection run, possibly due to reducing conditions existing in the aquifer at that stage, which allowed Fe^{2+} to enter into solution. Longer term injection would seem to introduce sufficient oxygen into the aquifer to create oxidising conditions limiting the dissolution of iron. This would seem to be in agreement with experience elsewhere in the world where up to five times the injected volume could be recovered before the iron concentration started increasing again.

Manganese chemistry is more complex and the response to injection of oxygenated surface water needs further study. Initially the manganese concentration was very low after the main injection test but subsequently increased rapidly. As the stability diagram indicates, manganese (Mn^{2+}) remains soluble under a wide range of redox and pH conditions. These parameters will need to be measured during operation to find the optimum conditions to maintain low values for dissolved manganese.

2.5.5 Recommendations

Recommendations regarding water quality monitoring are:

- Detailed planning of monitoring for longer scale injection runs should be carried out to provide the information that could not be obtained during the brief pilot injection tests.
- Provided the borehole is open and accessible, down-the-hole logging should be carried out to establish the redox conditions in the deeper parts of the profile. Logging should be carried out before injection (before a sampling run) and after injection (also before a sampling run). If possible it should also be done three to six months after injection or when the iron or manganese concentration increases.
- Regular sampling and analysis of iron and manganese is essential. Before injection, monthly samples are needed for establishing background levels. After injection, sample weekly for two weeks, then monthly.
- Measurement of dissolved oxygen and oxidation-reduction potential is essential at each sampling in addition to the down-the-hole logging.

Sampling should take place after 30 minutes’ pumping using clear or white plastic bottles (NOT tinted, e.g. brown). Samples should be filtered at sampling and filtered and unfiltered samples should be submitted for analysis.

2.6 Engineering Issues

The primary purpose of implementing artificial recharge in Prince Albert is to increase the water reserves in storage that will be available for use during the summer peak periods. In other words, the goal is to maximise the use of the aquifer storage to store excess water from the wet periods for use during the dry season when most water is consumed.

As part of the installation of the new pipeline in the furrow, four 100 mm diameter take off points were installed along the pipeline corresponding to the following borehole locations:

- GZ00347
- P5
- P6
- Midway between P7 & P8

This was done in anticipation of the artificial recharge project, thus it is now relatively simple to convey the water from the source water pipeline to the recharge borehole locations.

A test rig (**Figure 2.22**) was designed and assembled for injection testing. The rig consists of the following components:

- Sample tap - for sampling water quality
- Strainer to protect the water meter from being blocked or damaged by debris in the water
- Water meter with pulse output for logging
- Pressure gauge
- Air valve - for removing air from the source water
- Butterfly valve - for shutting off and controlling flow
- Layflat injection hose – to minimise cascading and to avoid potential cavitation in the pipeline
- Fabricated legs to support the test rig

During the initial tests, the simple strainer located upstream of the water meter fulfilled its function of protecting the water meter. However, due to the large number of leaves in the source water, the strainer often became blocked within a short period of time and would have to be cleaned, which disrupted the testing. Each time the strainer was cleaned, the water had to be isolated at the valve at the main pipeline. For the subsequent tests the following components were added to the rig:

- A large surface area, in line strainer was positioned before the test rig to trap the leaves. This strainer has a flushing valve and can be flushed without having to shut off the flow to the test rig.
- A ball valve upstream of the new strainer for easily isolating the flow if required.
- An additional pressure gauge was installed upstream of the strainer to monitor the pressure difference across the strainer (as an indicator of when the strainer needed clearing).

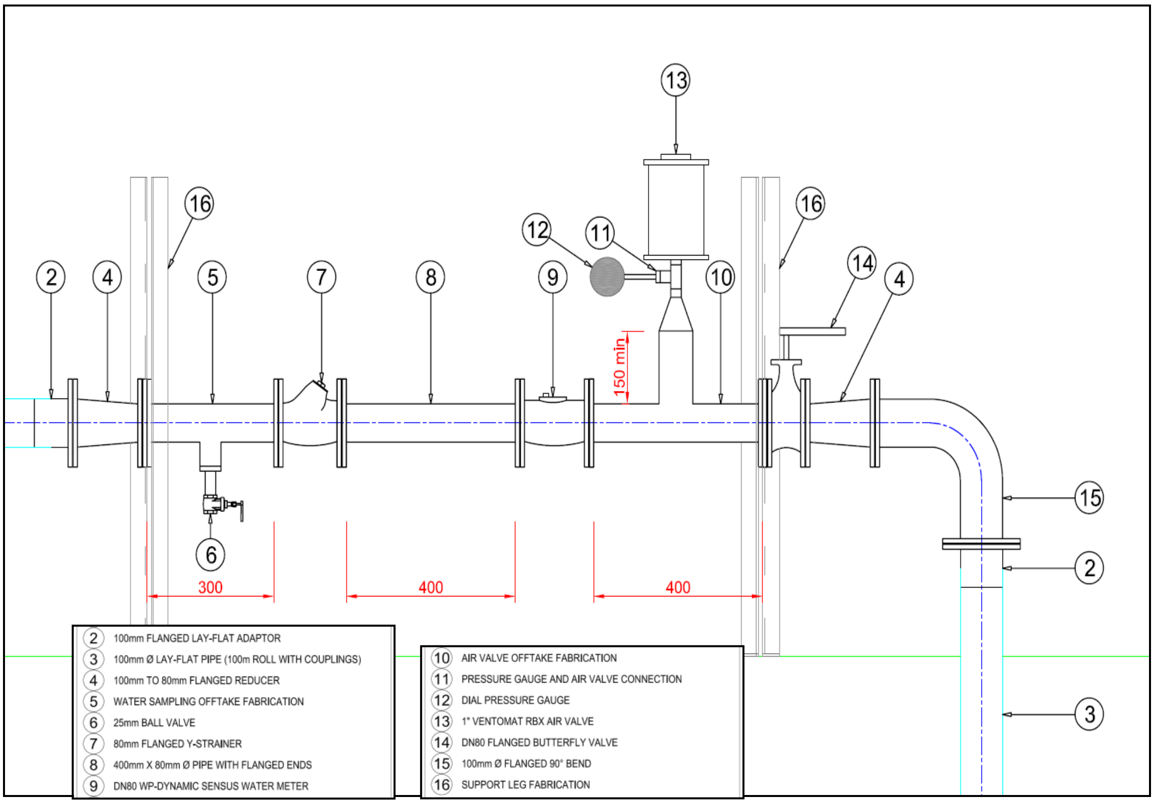


FIGURE 2.22 TEST RIG COMPONENTS USED FOR INJECTION TESTS

The design of the works proposed for the permanent installation needed for injection has taken these points into account in the following way:

- Two large-surface area intake screens have been specified, one at the intake of the furrow from the river and the second at the point that the furrow enters the pipeline. This will greatly reduce the amount of leaves entering the pipeline and would also significantly reduce the frequency of the blockages at the municipal meter.
- The pipework and fittings specified for the injection infrastructure includes a large surface inline strainer and the additional valve and pressure gauge for easy monitoring of strainer clogging.

Due to the potential for borehole clogging, allowance has been made for the construction of recharge wells of 5 m deep and 1.25 m in diameter in the area adjacent to two of the three injection boreholes sites (BH5 and BH7). The injection wells would be used in place of the injection borehole and would use the same injection pipework. As a result no additional pipework has been allowed for during implementation.

Table 2.6 describes the pipeline infrastructure required at each of the three injection boreholes.

TABLE 2.6: RECHARGE BOREHOLES AND ASSOCIATED PIPE INFRASTRUCTURE

DWA Borehole Number	Associated abstraction Borehole	Pipe Length	Max Flow (l/s)	Pipe size (mm dia)	Pipe Type	Comment	Downhole pipe depth required (m)	Downhole injection pipe diameter (mm)
GZ00346	BH5	75	30	110	uPVC		50	102
GZ00344	BH7	12+36	8	63	HDPe	12 m 110 mm uPVC shared	50	50
GZ00350	BH8	12+64	16	90	uPVC	12 m 110 mm uPVC shared	50	76

The infrastructure for re-abstracting the water from the aquifer and supplying the water to the water treatment works is already in place.

A twelve month programme for implementation is envisaged with the water use license taking seven months and a seven month period required for design, tender and construction. A one year mentoring period has been allowed for before final handover.

	Month 1	Month 2	Month 3	Month 4	Month 5	Month 6	Month 7	Month 8	Month 9	Month 10	Month 11	Month 12	Month 13	Month 14	Month 15	Month 16	Month 17	Month 18	Month 19	Month 20	Month 21	Month 22	Month 23	Month 24	Month 25
DWA water use license application																									
Design, tender and construction																									
Training & commisioning																									
Management mentoring																									
Final Handover																									

2.7 Environmental Issues

A detailed study of environmental issues was conducted by Dr S Milton of Sukaroo, and is captured in Murray (2007).

Under the 2010 NEMA regulations (DEA 2010) no environmental authorization is required. The pipelines conveying the water from the existing pipeline to the recharge boreholes are within 32 m of the watercourse, but all the pipelines are less than 1000 m in length and so do not exceed the limits of section 9 of Regulation 544.

No storage facilities are being constructed therefore section 12 of Regulation 544 does not apply.

2.8 Legal and Regulatory Issues

The current Registered Use for the water sources utilised by Prince Albert Municipality are listed in **Table 2.7**

TABLE 2.7 PRINCE ALBERT MUNICIPALITY (PAM) REGISTERED WATER USE FROM THE WARMS DATABASE

Source	Volume in m^3 per annum	Percentage allocated to PAM	Volume allocated to PAM (m^3/annum)
Groundwater	229 000	100%	229 000
Dorps Rivier*	**1 350 480**	**12.60%**	**170 700**
		TOTAL	**399 700**

*The Kweekvallei Irrigation Board has a total authorised abstraction from the Dorps Rivier of 1 350 480 m^3 per annum of which 12.64% is allocated to PAM. The actual volume of water abstracted from the Dorps Rivier is not known due to fouling of the meter with leaves.

Without artificial recharge and pumping according to the recommended pumping rates (Murray, 2007), a total of 488 700 m^3 per annum would be abstracted from groundwater. The additional abstraction required to re-abstract the recharged water from P5 and P7 areas is 60 000 m^3 per annum giving a total of 548 700 m^3 per annum, more than double the current registered groundwater use.

Typically an artificial recharge project would include three activities that require authorisation:

1. Abstracting the source water from a water resource (DW760 NWA Section 21a) – this is covered under the allocation of the Kweekvallei Irrigation Board as no additional water is to be taken from the resource

2. Storing of water (DW762 NWA Section 21b) - Required

3. Re-abstracting the water from the aquifer (DW760 NWA Section 21a) – Required.

In summary, the Prince Albert Municipality must:

- Apply for a license for the storing of 60 000 m^3 of water in the aquifer compartments of P5 and P7. Two key issues will need to be addressed in the license application documentation. The first is that the applicant will have to prove that the water is being stored in the aquifer and that it is not leaking away in the period between the injection and the re-abstraction. Secondly the applicant will have to demonstrate that the injectant water does not contain waste.
- Ensure that the current registered use is properly authorised and licensed where required.
- Ensure that the additional abstraction required for re-abstracting the artificially recharged water is properly authorised and licensed where required.

2.9 Economics

The construction of the scheme was costed based upon 2010 costs (**Table 2.8**). The total cost of supplying an effective 60 Ml of water storage is R860 093 or R14.33 per m^3. This is significantly less than the cost of alternative water storage, using comparative costs based on the *Cost Benchmarks, Typical Unit Costs for Water Services Development Projects*, published by DWAF in 2003 and escalated to current values.

The cost of providing the same volume of storage with concrete reservoirs would be about seventy times the cost or R60m rand.

It is unlikely that providing the same volume of storage in the form of a surface earth dam would be feasible considering evaporation, the land availability and environmental issues. However, assuming it was feasible, the dam costs alone would amount to approximately R4 million or four and a half times the cost of the aquifer storage.

TABLE 2.8 PROVISIONAL COST ESTIMATE

Estimated Value: Direct Costs	**Rands**
Fabrication and installation of filter screens at stream source and pipe inlet	12 230
Borehole GZ00346 (associated with abstraction borehole BH5)	
Connect to bulk supply pipeline, lay pipe, construct recharge apron, injection infrastructure, monitoring facilities and fencing.	79 712
Recharge Well (5 m deep)	19 337
Monitoring equipment	10 000
Boreholes GZ00344 & GZ00350 (associated with abstraction boreholes BH7 & BH8)	
Connect to bulk supply pipeline, lay pipe, construct recharge apron, injection infrastructure, monitoring facilities and fencing.	123 712
Recharge Well (5 m deep)	19 337
Monitoring equipment	5 000
P&G's (20%)	51 420
Contingencies (10%)	30 852
Sub Total Direct Costs	**351 600**
Estimated Value: Indirect Costs	
Professional Fees (20%)	70 320
Water Use Licences	49 104
Environmental authorisation & monitoring	-
Survey & servitude registration	20 460
Occupational health & safety requirements	20 051
Construction Monitoring	77 748
Management & operation training	47 800
O&M mentoring and GW monitoring start-up (1 year)	98 200
Disbursements (5%)	19 184
Sub Total	**754 468**
VAT	**105 625**
TOTAL	**860 093**

2.10 Management and technical capacity

The day to day operation of the scheme is not difficult and should be easily performed by the normal operational staff of the municipality. The more difficult aspects of the scheme operation are in the overall management of the aquifer, the management of pumping schedules to maximise the benefits of the scheme, and the monitoring required to generate the data that can be used to make informed management decisions. The management and technical tasks, their frequency and the capacity required for the tasks are listed in **Table 2.9**. The competency needed for the tasks is described and it is assumed that the municipality will have the resources to perform much of this in-house but will probably need to contract external skilled resources for some of the tasks needing specialist knowledge.

TABLE 2.9 MANAGEMENT AND OPERATIONAL TASKS

Task Description	Frequency	Responsibility / required competency
Ensure that all management and operation tasks are appropriately resourced and ensure each person understands the tasks to be performed and has the tools and resources needed to do the work. Monitor reports and regularly assess performance of all parties involved (both internal and external)	Ongoing	Municipal water manager
Develop operating rules with the aim of having the aquifer water levels at an optimum level at the onset of the recharge period to ensure maximum benefit from AR	Prior to project handover	Hydrogeologist
Ensure that the scheme is operated according to the operating rules designed to allow for maximum benefit from AR	Ongoing	Municipal water manager
Ensure ongoing groundwater management, manage consultation and communication between specialists, municipal staff and the community	Ongoing	Municipal water manager
Check all infrastructure components and pipework for leaks and damage, and repair where required	Prior to injection event commencing	Municipal scheme operator
Monitor weather conditions and the injectant water quality (turbidity specifically) to ensure no turbid water is injected into the boreholes	During injection events as often as required.	Municipal scheme operator
Check and clear screens at furrow inlet, pipe inlet and strainers	During injection event, twice per day or more often as required.	Municipal scheme operator
Design monitoring system, identify monitoring boreholes and set monitoring criteria	Prior to injection event commencing	Hydrogeologist
Design water quality monitoring system, identify monitoring points, sampling schedules and determinands to be analysed	Prior to injection event commencing	Water quality specialist
Check all flow meters and data loggers are working correctly	Prior to injection event commencing	Experienced technician
Manual readings & records of water meters, pressure gauges and water levels	During injection event, twice per day or more often as required.	Experienced technician
Collect water samples and conduct field water quality tests	During injection event, based on schedule.	Experienced technician
Download data	After injection event completed	Experienced technician

Task Description	Frequency	Responsibility / required competency
Analyse data and make recommendations on future injection and abstraction schedules	After injection event completed	Hydrogeologist / water quality specialist
Assess injection efficiency and make recommendations	After injection event completed	Hydrogeologist & water quality specialist
Re-habilitate boreholes (flushing, acid cleaning)	As required	Contractor

2.11 Institutional arrangements

The Prince Albert Municipality (PAM) is the owner, manager and operator of the scheme and needs to maintain a number of institutional relationships to successfully manage the scheme.

The Kweekvallei Irrigation Board is the manager of the surface water scheme that supplies the water from the Dorps Rivier to the town. The PAM has an allocation from this water supply and an agreement in place that saw the municipality fund the installation of the pipeline in return for a greater share of the water supplied from this system. The primary recharge period for the scheme is planned to take advantage of the furrow cleaning period which is managed and scheduled by the Kweekvallei Irrigation Board. The relationship between the two institutions is key for the successful operation of the scheme.

Some of the recharge and abstraction boreholes are located on private land and the PAM has agreements in place with the landowner covering access to this infrastructure. Any additional infrastructure must be implemented in terms of this agreement or additional agreements must be put in place to cover new infrastructure. The landowner's use of groundwater must be monitored and managed to ensure that the recharged water is retained in storage for the use of the PAM.

The license to abstract water and the proposed license to store water underground are granted by DWA who have monitoring and reporting requirements that must be fulfilled in terms of the license conditions.

2.12 Project Implementation Stages

Table 2.10 provides a summary of the project implementation stages, the current progress with project implementation and the work still to be completed.

TABLE 2.10 PROJECT IMPLEMENTATION STAGES

Project Phase	Key activities	Status and Progress		Authorisation Requirements
Pre-feasibility phase	Identify the potential AR project and detail the information currently available	✓	Pre feasibility completed by Groundwater Africa in 2007	None
	Assess the potential AR project based on existing information	✓		
	Identify the work required for the feasibility phase and estimate the cost of producing the feasibility study	✓		
	Establish existing water use license conditions and authorisation requirements from DWA and DEA	✓		
Feasibility phase	Undertake a detailed feasibility study including relevant testing (injection/infiltration/borehole pump tests)	✓	Feasibility study completed by Groundwater Africa in 2007 excluded testing. Testing took place in June to August 2010	None. DWA permission to test was obtained. Both DWA and DE&ADP informed of tests. Tests witnessed by DWA.
	Do preliminary design of the infrastructure required	✓	This document	
	Identify the phases of project implementation if phased implementation is proposed	✓	No phasing proposed. If funding constrained, implement GZ00346 (BH5 compartment) first then GZ00344 & GZ00350 (BH 7&8 compartment)	
	Estimate the costs of the project implementation	✓	This document	
	Identify funding sources and ensure the feasibility study complies with all requirements of the proposed funder	□	PAM to identify suitable funding sources. Most probably internal budget or MIG or combination.	
	Compile a detailed programme	✓	Summary programme in this document. Actual programme dates dependent on the confirmation of funding,	

Project Phase	Key activities	Status and Progress		Authorisation Requirements
Implementation phase	Groundwater infrastructure development & testing	✓	All groundwater infrastructure already in place	Apply for DWA water use license for storing water. Verification of the existing authorisation is needed to confirm that it covers the current abstraction (surface and groundwater). No environmental basic assessment triggered by planned activities.
	Engineering detailed design, tender, construction and commissioning of AR infrastructure	□	Most of the infrastructure is in place. Minor works required to link with bulk supply pipeline and equip injection boreholes. The infrastructure is relatively low cost, so no complex tender procedures needed. If done in 2 phases, then the construction costs can be split into 2 quotations of below R200 000 each	
	Set up groundwater and recharge water monitoring system	□	Largely been done, finalise during implementation	
	Compile operation & maintenance procedures and train operators		To be compiled to match the designed infrastructure and monitoring system. To be done to coincide with construction completion	
Operation and maintenance phase	Performance monitoring during production		A one year mentorship period is budgeted for	Compliance monitoring and reporting
	Modified operation & maintenance procedures		To be fine tuned during mentorship period	
	Final monitoring and reporting strategy		On completion of mentorship period	

2.13 Conclusions and recommendations

The conclusions and recommendation are summarized as follows:

1. Artificial recharge should be developed as the "back-up" water security for the high summer demand period. It should be carried out every year if the groundwater levels have not recovered to aquifer full levels during winter when surplus surface water is available for recharge.

2. The estimated cost of the scheme is R860 093 or R14.33 per m^3 for the 60 Ml of additional storage capacity added to the water supply system. It is recommended that the scheme be implemented as a cost effective method of increasing water security for the summer peak demand period.

3. Borehole injection should take place in the following areas:

   a) P5 groundwater compartment: Borehole GZ00346 at a maximum rate of 30 L/s

   b) P7 groundwater compartment:

4. Borehole GZ00344 at a maximum rate of 8 L/s

   a) Borehole GZ00350 at a maximum rate of 16 L/s

   b) After each injection cycle the following should be assessed to establish the performance of the scheme:

   c) Water level response in the injection borehole and surrounding boreholes

   d) EC, iron and manganese response in the closest production boreholes (i.e. P5, P7 & P8).

5. If the efficiency of the injection boreholes decreases markedly (as a result of clogging) the boreholes should be rehabilitated by backflushing (i.e. pumping) and possibly applying acid. This must be assessed after at least two injection runs.

6. The infrastructure design must allow for injection into shallow wells, pits or trenches if clogging of the injection boreholes is severe.

7. The operator of the scheme must be trained in:

   a) Ensuring recharge does not take place if the injectant (river water) is turbid

   b) Cleaning the filters (i.e. removing leaves) during injection

   c) Measuring and recording water levels and flow meter readings

   d) Sampling and ensuring the analysis of the groundwater water quality before and after each season of artificial recharge.

3 BOREHOLE INJECTION IN PLETTENBERG BAY

3.1 Introduction

An artificial recharge assessment was initially conducted in 2007 (Murray, 2007) with the idea of injecting water from the Keurbooms River into boreholes in Kwanokuthula. The study recommended an approach to assessing the full capacity of the scheme. This report essentially updates the previous report by Murray,2007.

Like a number of coastal towns, Plettenberg Bay's water requirements increase remarkably over the summer, and have an enormous peak demand over the December holidays:

Summer demand: 12 – 13 Ml/day [Peak Week Christmas – New Year: 17-18 Ml/day]

Existing water supply:

- 6.9 Ml/day from surface water (drought) / 8.6 Ml/day (normal)
- 3.4 Ml/day from existing boreholes
- 2.8 Ml/day from the Roodefontein Dam (off-channel storage dam – this is the estimated portion - 55% of storage - which is allocated to the municipality)
- Total: 13.1 Ml/day

Existing water supply: Under construction (November 2010):

- 2.0 Ml/day from desalination (during peak demand periods)

Proposed water supply:

- 2.3 Ml/day from artificial recharge (over 5 peak demand months)

Total: 17.4 Ml/day

The target groundwater/artificial recharge capacity is considered to be:

Artificial recharge:	**3.9 Ml/day over 3 months or 350 Ml**
Abstraction:	**5.1 Ml/day over 5 months or 780 Ml** (natural groundwater & artificially recharged water)

Note: A 10% loss on injection volumes have been assumed, i.e. total injection target is 4.3 Ml/day

To date, the Kwanokuthula Aquifer has been underutilized; however, its considerable storage potential makes it a viable option for large-scale use in summer coupled with artificial recharge in winter. In order to finalise the feasibility study, a period of large-scale abstraction followed by large-scale injection, followed by large-scale abstraction is needed. The project status is outlined in **Table 3.1**.

TABLE 3.1 PROJECT STATUS

Phase	Description	Key tasks	Target completion date
Phase1	Pre-feasibility study	1. Initial indication of scheme viability 2. Identifying key issues	Complete (Murray, 2007)
Phase 2	Feasibility study – Injection testing	1. Assess borehole injection capacity 2. Assess aquifer storage and recovery potential 3. Apply for water use license (sub-surface storage)	1. Injection capacity: Complete (2010) 2. Abstraction: Nov 2010 - April 2011 3. Injection using existing boreholes (GWA5 & Bh6): Jul – Sep 2011 4. Abstraction: Nov 2011 - April 2012 5. License: Dec 2011
Phase 3a	Design & construction	1. Drill injection & monitoring boreholes 2. Design & construct treatment & conveyance infrastructure to get water to injection boreholes. 3. Operation & maintenance procedures	1. Drilling: Jul 2012 2. Design & construction: Jul 2013 3. O&M procedures: Jul 2013
Phase 4	Production & Post-project support	Performance monitoring during production Modified operation & maintenance procedures Final monitoring and reporting strategy	Completion: Jul 2013 Monitoring & final operation procedures: Jun 2014.

Note on Phase 3: The scheme can be up-scaled as the need arises i.e. the injection target of 4.3 Ml/d could be achieved in two or more stages.

Phase 2 of the artificial recharge plan involves three stages that need to be done prior to finalizing the design of the scheme. These include:

1) Large-scale abstraction. Five months of continuous abstraction is needed to assess the storage capacity of the aquifer and to create the space for artificial recharge.

2) A short period of rest followed by three months of artificial recharge.

3) A short period of rest followed by large-scale abstraction.

The planned abstraction – injection – abstraction cycle is shown in **Figure 3.1**.

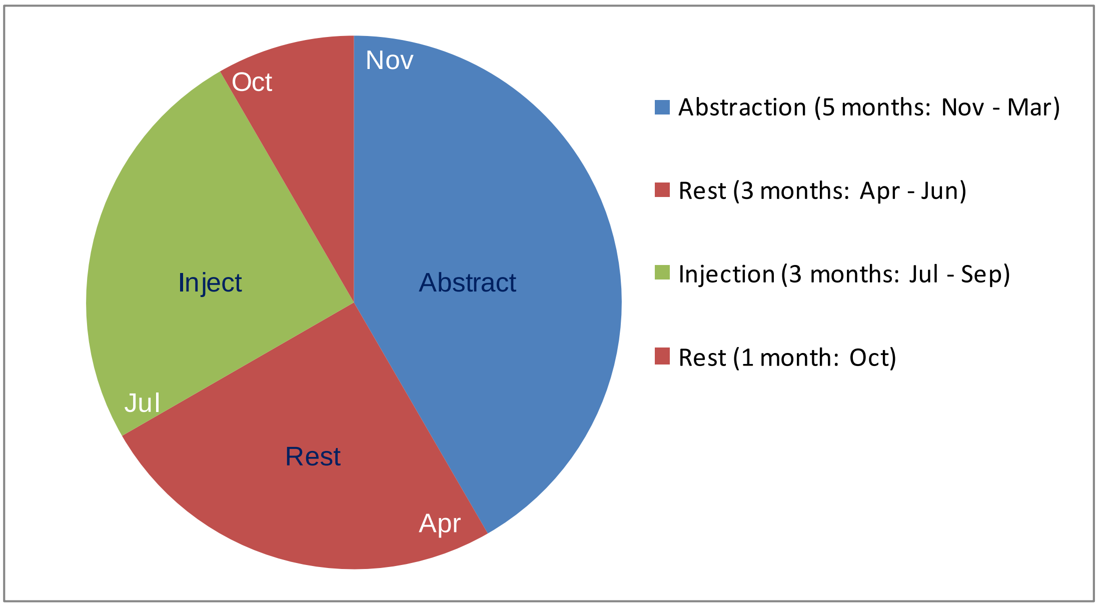


FIGURE 3.1 PLANNED ARTIFICIAL RECHARGE CYCLE

At this stage no dedicated injection boreholes exist, although GWA 1 and Bh 6 could be used in the mean time for a trial injection test (the combined injection rate would, however, only be ~20 L/s or 1.7 Ml/day, which is significantly short of the 50 L/s or 4.3 Ml/day injection target). The location of Plettenberg Bay's abstraction boreholes and reservoirs in the Kwanokuthula area are shown **in Figures 3.2** and **3.3**.

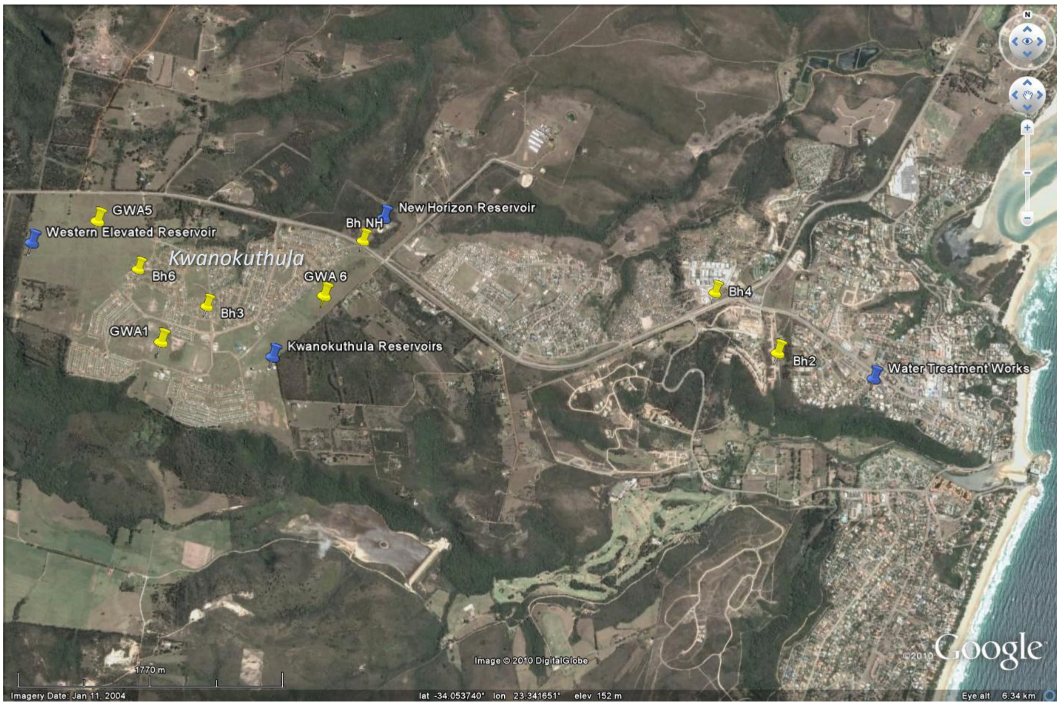


FIGURE 3.2 PLETTENBERG BAY & KWANOKUTHULA BOREHOLES (GOOLE EARTH, 2010)

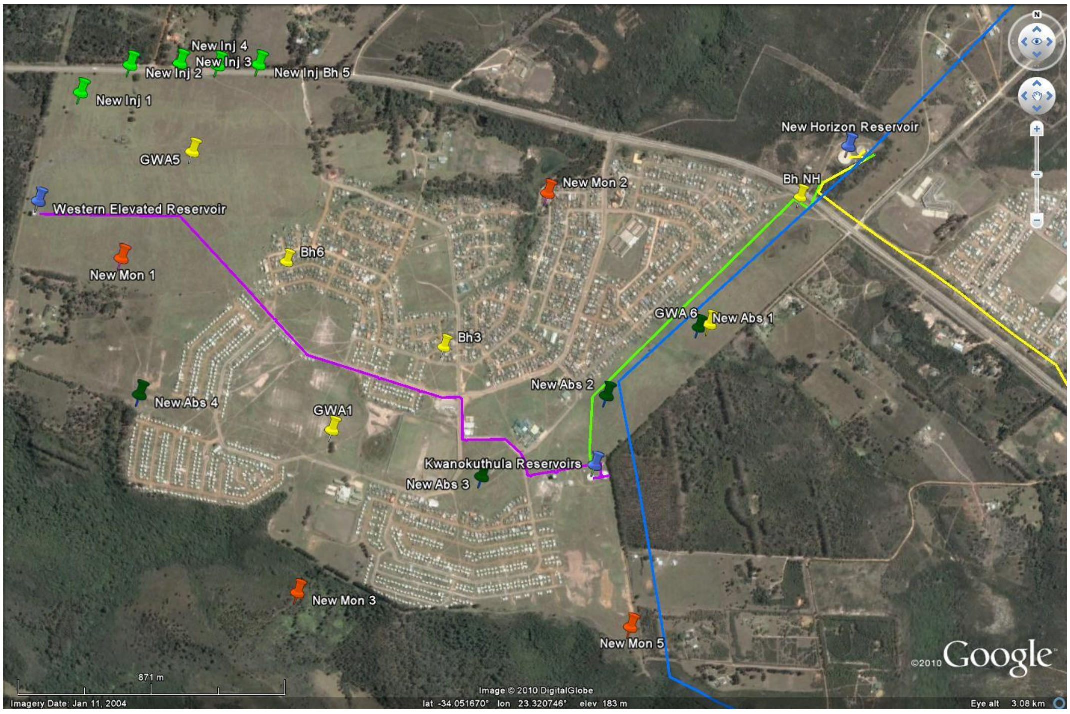


FIGURE 3.3 KWANOKUTHULA'S EXISTING AND PLANNED BOREHOLES (GOOGEL EARTH, 2010)

(Existing abstraction = yellow; planned abstraction = dark green; planned injection = bright green; planned monitoring = orange)

3.2 The need for artificial recharge

3.2.1 The need for additional summer supplies

As a seasonal holiday town, Plettenberg bay experiences a large variation in the monthly water demand. While there is a distinct peak over December and January, the general pattern is a six month period of higher demand and six months of a lower demand (**Figure 3.4**). Artificial recharge is considered as an option to augment the summer demand.

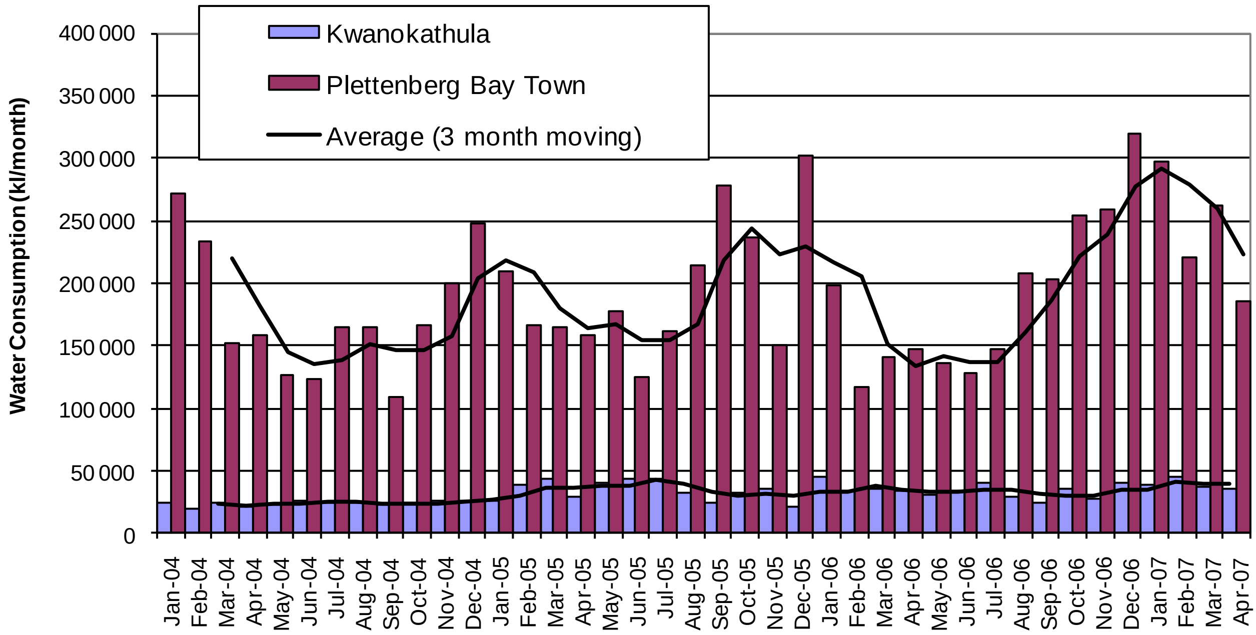


FIGURE 3.4 Monthly variation in water demand for Plettenberg Bay town and Kwanokathula

3.2.2 How groundwater abstraction with artificial recharge could meet the summer requirements

The water requirements over the summer months is in the order of 150 000 m^3/month more than the winter requirements. This equates to ~5 Ml/day or ~58 L/s of continuous supply. It is estimated that this can be supplied over a period of five months through artificial recharge plus natural groundwater abstraction. The existing and proposed supply system is outlined in **Table 3.2**.

Natural groundwater together with artificial recharge should meet the additional 5 Ml/day required for the summer months.

TABLE 3.2 PROPOSED BOREHOLE ABSTRACTION PLAN WITH ARTIFICIAL RECHARGE

a) Abstraction at higher rates

Abstraction Boreholes	Production Yield (L/s)	Daily Yield (Ml/day)	5 Month Yield (Ml)	Surplus (+) or Defecit (-) (Ml)
Target groundwater yield (Kwanokuthula only)	**53.7**	**4.6**	**705.2**	**0**
Existing boreholes				
Bh3	10	0.9	131.3	
Bh 6	6	0.5	78.8	
NH	4	0.3	52.5	
GWA1	8	0.7	105.1	
GWA5	5	0.4	65.7	
GWA6	8	0.7	105.1	
Total	*41*	*3.5*	*538.4*	**-166.8**
New abstraction boreholes with artificial recharge				
2 New abstraction boreholes (7 L/s each)	14	1.2	183.9	
Total yield with AR	***55***	***4.8***	***722.3***	**17.1**
Artificial recharge requirements	**Injection Rate (L/s)**	**Daily Injection (Ml/day)**	**3 Month Injected Volume (Ml)**	
Kwanokuthula				
3 Injection Bhs (10 L/s each)	30	2.6	235.9	
Losses (10%)		0.3	23.6	
Additional available volume	***30***	***2.3***	***212.3***	**28.4**

b) Abstraction at lower rates

Abstraction Boreholes	Production Yield (L/s)	Daily Yield (Ml/day)	5 Month Yield (Ml)	Surplus (+) or Defecit (-) (Ml)
Target groundwater yield (Kwanokuthula only)	**53.7**	**4.6**	**705.2**	**0**
Existing boreholes				
Bh3	10	0.9	131.3	
Bh 6	6	0.5	78.8	
NH	4	0.3	52.5	
GWA1	5	0.4	65.7	
GWA5	5	0.4	65.7	
GWA6	5	0.4	65.7	
Total	*35*	*3.0*	*459.6*	**-245.6**
New abstraction boreholes with artificial recharge				
4 New abstraction boreholes (5 L/s each)	20	1.7	262.7	
Total yield with AR	***55***	***4.8***	***722.3***	**17.1**
Artificial recharge requirements	**Injection Rate (L/s)**	**Daily Injection (Ml/day)**	**3 Month Injected Volume (Ml)**	
Kwanokuthula				
4 Injection Bhs (10 L/s each)	40	3.5	314.5	
Losses (10%)		0.3	31.4	
Additional available volume	***40***	***3.1***	***283.0***	**20.4**

Note: GWA5 had not been test pumped at the time of writing this report

The yields presented in **Table 3.2** are provisional estimates. Boreholes GWA1 & 6 can both supply 8 L/s, but it may be best to pump them both at yields of 5 L/s because of the high iron concentrations present in these boreholes (minimising the abstraction rates may minimise this problem). Likewise, the injection borehole capacities are not known, although indications from the injection test on Bh 6 show that 10 L/s is a reasonable estimate.

The existing Airport borehole and Bh 4 have a combined yield of 4.3 L/s or 0.4 Ml/day. This brings the total groundwater potential with artificial recharge to ~5.2 Ml/day over the five peak demand months.

The above estimates for the Kwanokuthula Aquifer also assume that natural recharge (inflow) to the aquifer is ~540 Ml/annum (~17 L/s). This needs to be tested by monitoring continuous abstraction followed by a rest period (and will only be known with a high degree of confidence after monitoring both rainfall and a few seasons of groundwater abstraction and levels). This 17 L/s estimate is based on recharge and throughflow estimates by CGS (1992 and 1997) and Murray (2007) natural recharge estimates. Should these estimates be conservative, artificial recharge would only be necessary if more water is required than the targeted 5 Ml/day. If, however, they are on the high-side, the artificial recharge requirements will need to be higher in order to make up the deficit.

3.3 The quantity and reliability of the source water

The source water for artificial recharge is surplus winter water from the Keurbooms River. There are two options for artificial recharge:

1. Inject fully treated water from the Water Treatment Works (preferable)
2. Treat water to minimum injection standards prior to injection.

Option1: Fully treated drinking water

There is sufficient capacity at the WTW to cater for a winter injection flow rate of 30 – 50 L/s. **Figure 3.5** shows the volume of water treated per month and the spare capacity per month based on an assumed monthly operating capacity of 410 000 kL/month (410 ML/month). The maximum capacity of the works is 22 000 kL/day (22 ML/day) or 660 000 kL/month (660 ML/month).

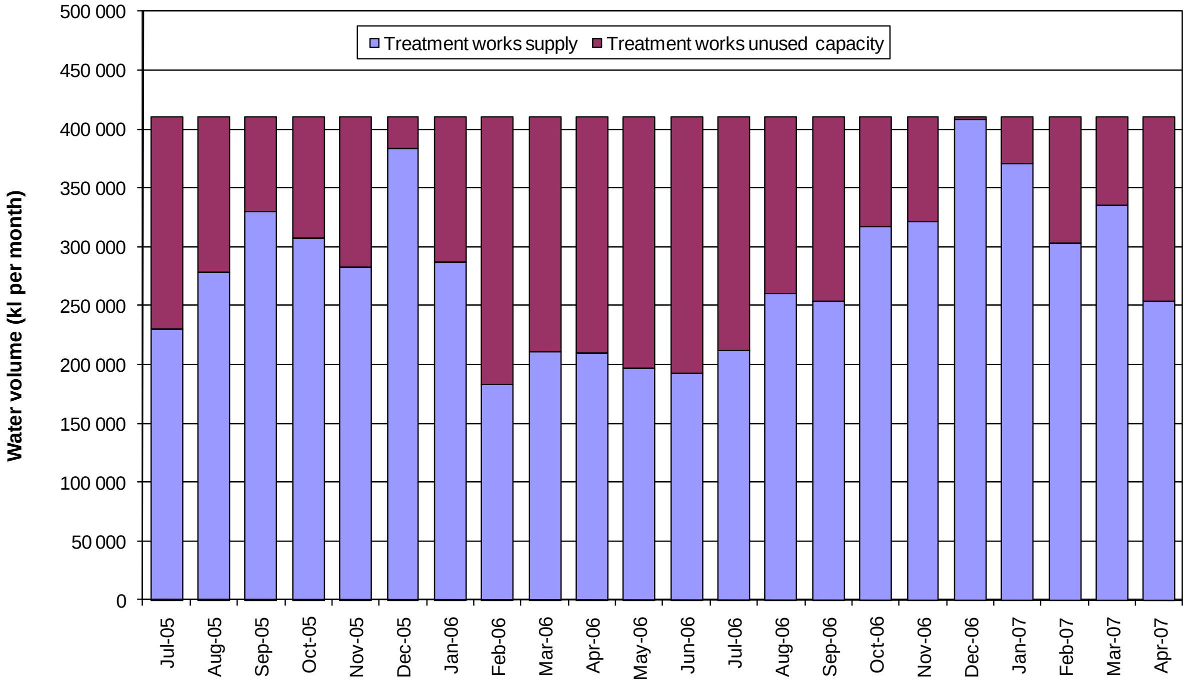
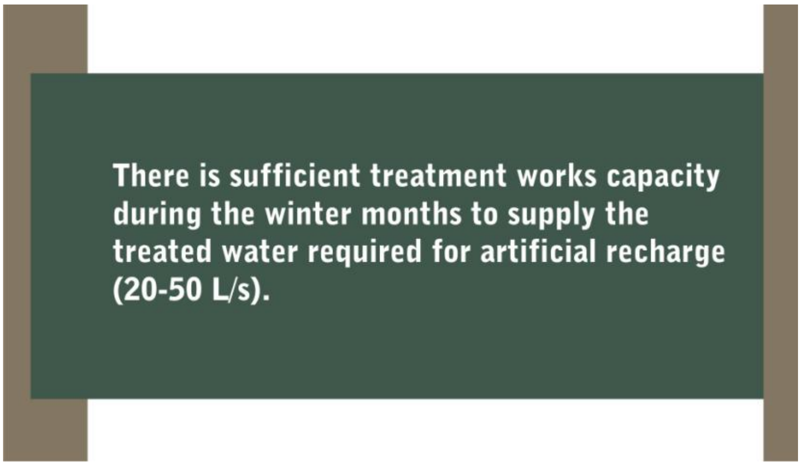


FIGURE 3.5 MONTHLY TREATMENT WORKS SUPPLY AND SPARE CAPACITY BASED ON THE UTILISED TREATMENT WORKS CAPACITY OF 410,000 KL PER MONTH (160 L/S)



Option 2: Raw water treated to injection standards

The Keurbooms River raw water bypass is designed to transfer water to the Roodefontein Dam at a rate of 35 L/s. This happens during off peak demand times when it is not necessary for the treatment plant to receive all the water from the Keurbooms pipeline and when the Roodefontein Dam has the capacity to receive the water.

The water would have to be treated to lower the turbidity, DOC and iron concentrations, increase the pH, and it should be disinfected to ensure microorganisms are not introduced into the aquifer.

35 L/s of untreated water would be available for artificial recharge during the winter, low demand months.

3.4 Aquifer hydraulics

The aquifer consists of fractured quartzites of the Peninsula Formation of the Table Mountain Group. The key issues relating to the storage capacity of the aquifer are:

1. Is the aquifer sufficiently permeable to accept recharged water?
2. What is the storage capacity of the aquifer?
3. Will the recharged water be recoverable? Or put another way, will the recharged water remain in storage until it is needed?

3.4.1 Is the aquifer sufficiently permeable: Will the aquifer receive artificially recharge water?

The ability of the Kwanokathula Aquifer to transmit water is good, with localised transmissivity values reaching up to several hundred m^2/day. **Table 3.3** summarises transmissivity values.

TABLE 3.3 TRANSMISSIVITY VALUES FOR KWANOKATHULA BOREHOLES

Bh No	T-early (m^2/day)	T-late (m^2/day)
BH 3 (Production, 2007)	60	60
BH 6 / GCS 3 (Pumping Test, CGS, 1993)	35	95
BH NH - New Horizon / Hillview (Pumping Test, 1998)	50	500
GWA1C (Pumping Test, 2010)	75	50
GWA6B (Pumping Test, 2010)	1 300	380

Borehole injection test on Bh 6

An injection test was carried out on Bh 6, the most up-gradient production borehole in the wellfield, where a continuous supply of water could be obtained for artificial recharge. The test was limited by the availability of source water, which was domestic water supplied to Kwanokuthula (i.e. fully treated drinking water). The test lasted 9 days and 12 hours and a total of 6 156 m^3 was injected at an average rate of 7.5 L/s. **Photo 3.1** shows the injection borehole and **Figure 3.6** shows the water level response in the injection borehole (the flow meter failed two days before the completion of the test). The fluctuating injection rates and corresponding water levels was a result of the changing availability of water for injection, which in turn was in accordance with the changing water use in Kwanokuthula. The higher flow rates (~9 L/s) were recorded during the night when domestic use was negligible.

PHOTO 3.1 BH 6 CONVERTED TO AN INJECTION BOREHOLE FOR CONDUCTING ARTIFICIAL RECHARGE TESTS

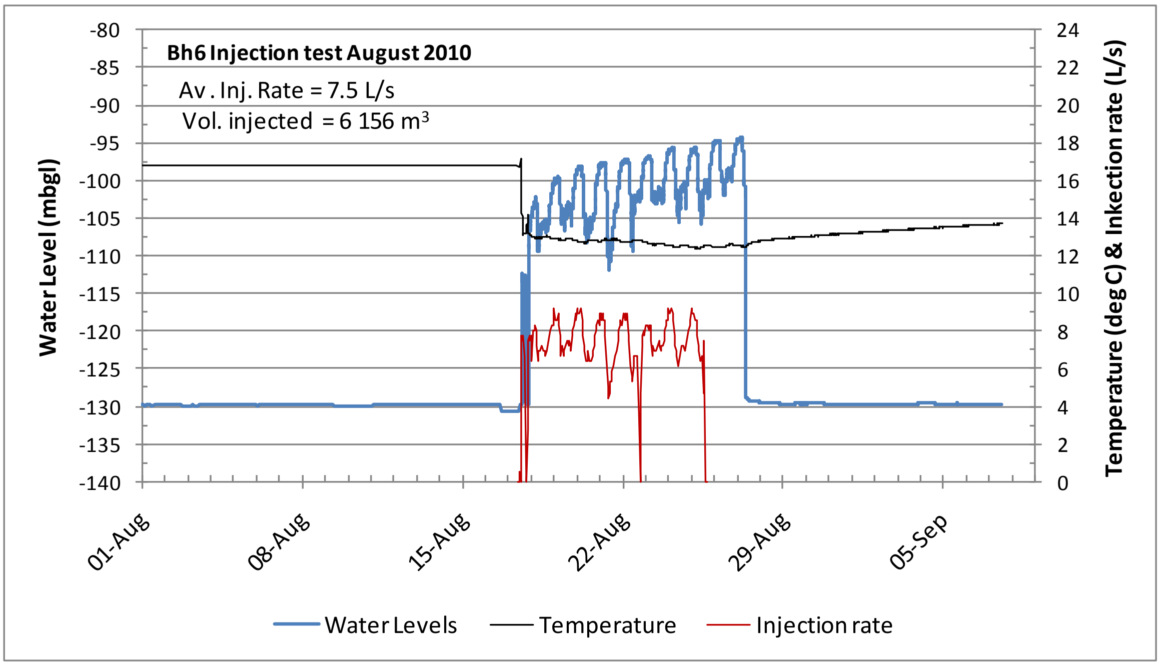


FIGURE 3.6 RESULTS OF THE 12 ½ DAY INJECTION TEST AT BH6

The main conclusion from the injection test is that the aquifer is highly permeable - it easily accepts artificially recharge water. The rapid drop in water levels after injection to virtually the starting water level indicates that the aquifer has a very high storage capacity and/or the water flows rapidly away from the point of injection.

3.4.2 What is the storage capacity of the aquifer?

Groundwater Consulting Services (GCS, 1993) estimated the size of the entire aquifer to be 22 million m^2 and the storage coefficient to be 0.5%. This would imply a storage capacity of 1.1 million m^3 per 10 m of vertical thickness of the aquifer. This refers to the entire TMG aquifer in and around Plettenberg Bay. If we consider the aquifer around Kwanokathula to lie between the shale band to the north and the Enon Formation to the south (1.5 km wide), and to extend over a length of 3 km, we get a storage capacity of 225 000 m^3 per 10 m of aquifer thickness. If we assume a conservative storage coefficient of 0.3% (as opposed to the 0.5% used by GCS) we get 135 000 m^3 per 10 m of aquifer thickness.

The aquifer can easily accept injection water at a rate of ~10 L/s per injection borehole.

If we set the artificial recharge target as 400 Ml, then this should require about 30 m of vertical aquifer thickness.

The aquifer has sufficient storage capacity to store the preliminary upper artificial recharge target of 315 Ml

3.4.3 Will the recharged water be recoverable?

To answer this question it is necessary to consider groundwater levels in the aquifer. The natural groundwater levels in the aquifer are surprisingly deep and flat, which can be observed in the two cross sections (**Figures 3.7 to 3.10**). This supports previous statements that the aquifer is highly permeable.

FIGURE 3.7 LOCATION OF CROSS SECTIONS THROUGH THE AQUIFER (GOOGLE EARTH, 2010)

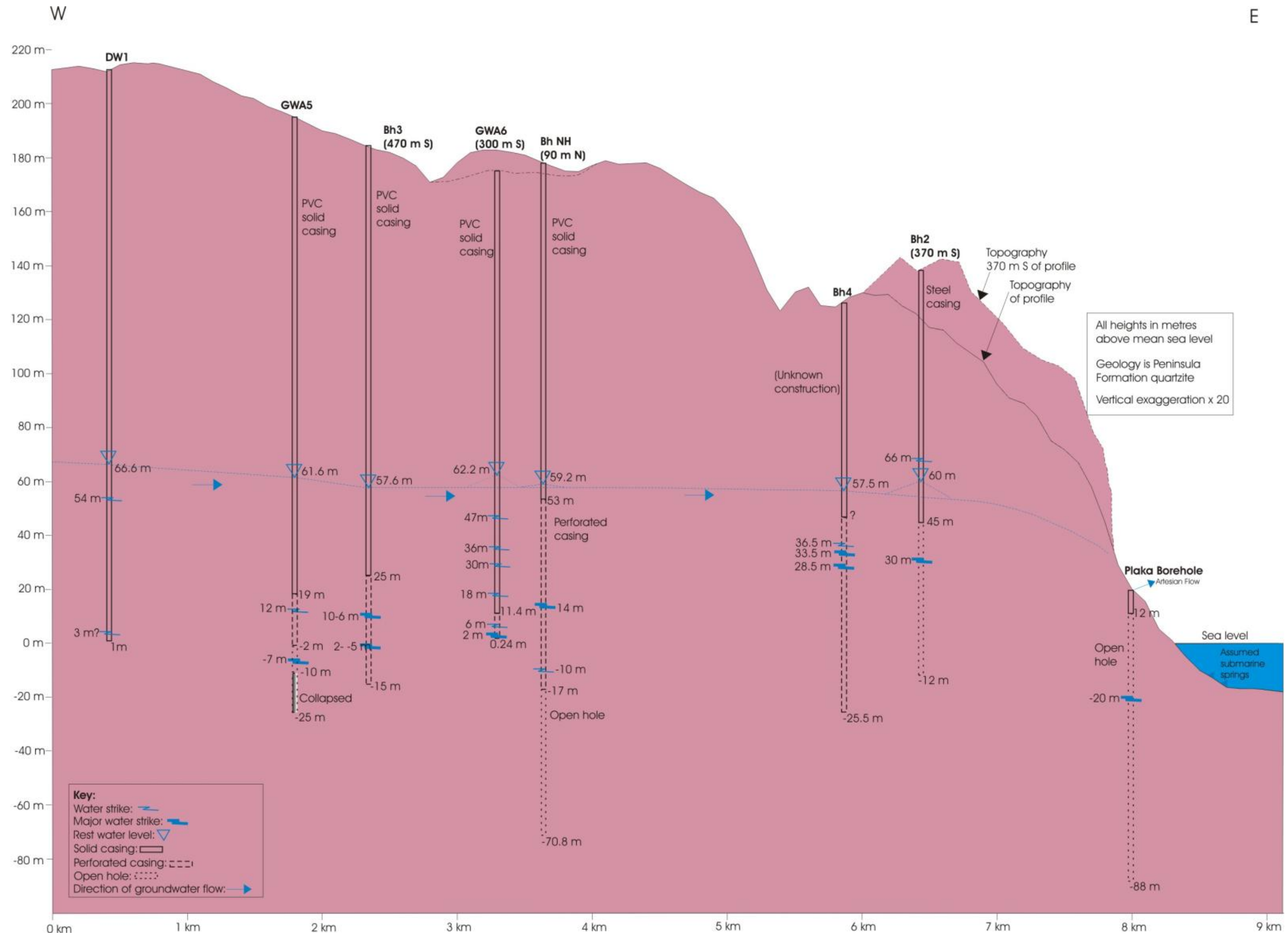


FIGURE 3.8
WEST – EAST CROSS SECTIONS THROUGH THE AQUIFER

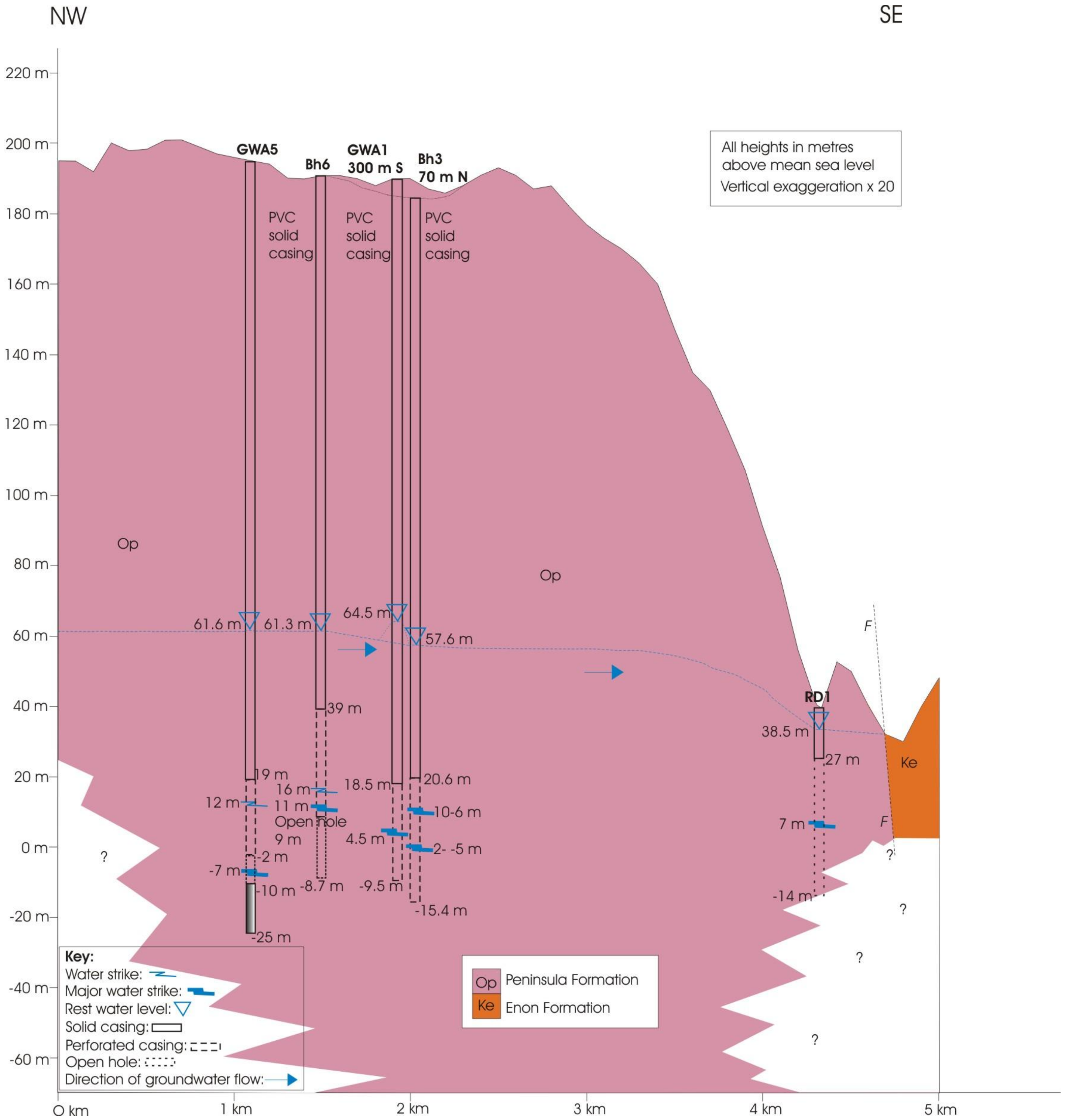


FIGURE 3.9 NW – SE CROSS SECTIONS THROUGH THE AQUIFER

Historic groundwater levels show that the water levels in Kwanokuthula return to around 60 mamsl after periods of abstraction (**Figure 3.10**). This level indicates the level at which the aquifer is in equilibrium, or the level where natural inflow matches natural outflow.

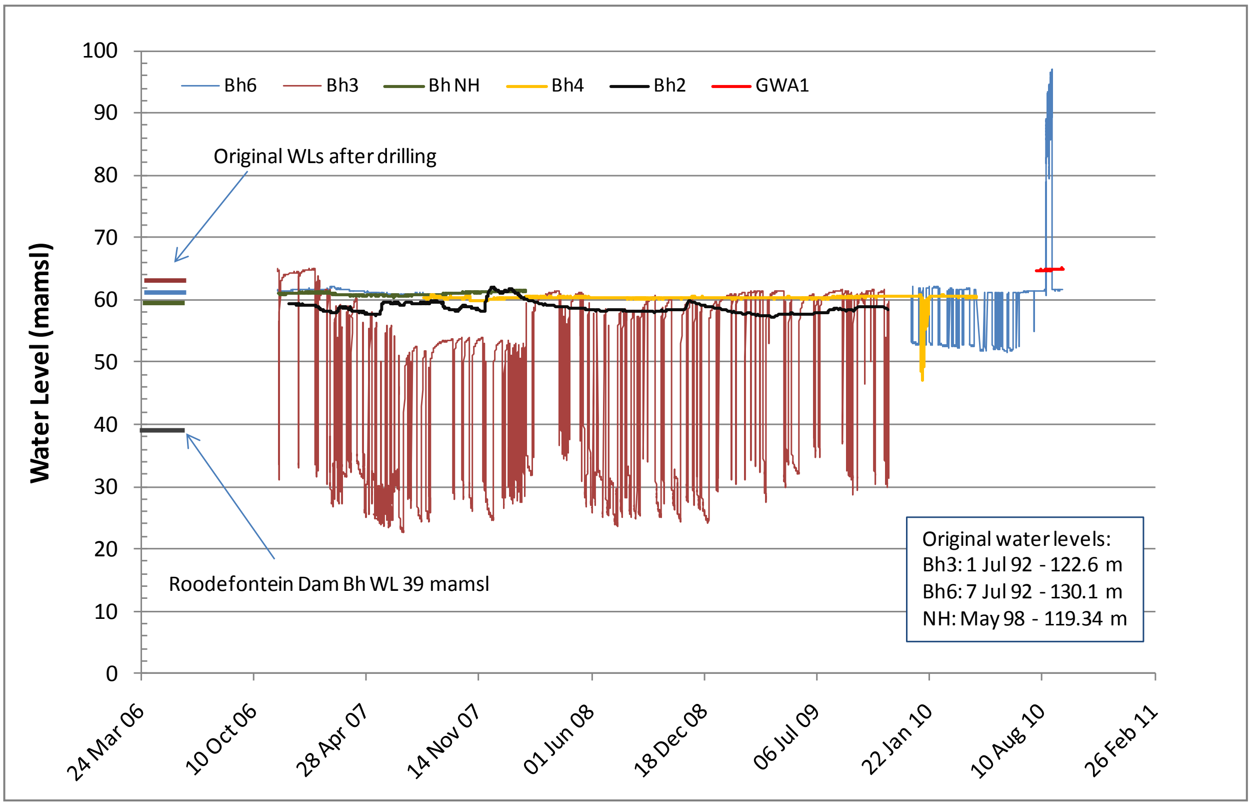


FIGURE 3.10 HISTORIC GROUNDWATER LEVELS

There are two options on recharging the Kwanokuthula Aquifer:

1. Store water above ~60 mamsl (the natural rest water level)
2. Store water below ~60 mamsl

Option 1: Store water above ~60 mamsl

With an unsaturated zone that ranges between 115 – 130 m in Kwanokuthula, there is ample potential storage space above the natural water level. Historic water level data suggests that water will discharge from the aquifer once the levels are raised above 60 mamsl. At this stage it is not known how rapidly this water would discharge from the Kwanokuthula area, i.e. whether water stored above 60 mamsl would be lost before the onset of the summer abstraction period. For this reason it is recommended that Option 2 should be implemented – at least during the initial stages of the project.

Option 2: Store water below ~60 mamsl

It is recommended, at least initially, that aquifer storage be "operated" below 60 mamsl. Once more is known about the flow system in the aquifer, it may become evident that little water is lost once water levels are raised above 60 mamsl and that storage above 60 mamsl is possible. However, at this stage it is best to assume that there will be losses if the water level is raised above 60 mamsl, and that abstraction and artificial recharge should be restricted to below this level.

This option would require lowering the water level in the aquifer first, and then artificially recharging it if the water levels do not naturally rapidly revert back to the "full" levels of 60 mamsl. If the water levels remain below the 60 mamsl mark, it is highly unlikely that artificially recharged water be lost (until the water levels in the aquifer have risen to 60 mamsl).

The aquifer should be "operated" below 60 mamsl (ie the storage in the aquifer below this level should be targeted for use in order to minimize the chance of water losses)

3.5 Water quality

Water quality issues relating to artificial recharge are documented in Murray, 2007. There are two options regarding the source of water for artificial recharge. Both rely on surplus winter flow from the Keurbooms River:

i) Fully treated surface water from the Water Treatment Works

ii) Surface water treated to artificial recharge requirements prior to injection. This would require installing a new treatment plant for the injectant (the water used for artificial recharge).

The key water quality characteristics and treatment requirements are summarized in **Table 3.4**.

TABLE 3.4 TREATMENT REQUIREMENTS FOR ARTIFICIALLY RECHARGED WATER

Parameter	**Injection water quality requirements**	**Fully treated water from the Water Treatment Works**		**Raw water from the Keurbooms River**	
		Water quality characteristics	*Treatment requirements*	*Water quality characteristics*	*Treatment requirements*
Electrical conductivity, mS/m	<60	~20	None	~7-15	None
pH	8.5 – 9	8.5 - 9.2	None	6	Lime addition[1]
Dissolved Organic Carbon (mg/L)	≤ 3	~3	None	~10	Activated carbon[2]
Iron as Fe mg/L	0.1	~0.1	None	~0.5	Flocculation & pH adjustment[3]
Micro-organisms: E. coli /100 mL	0	0	None	100 – 1 800	Ozonation only, due to high DOC[4]

Notes:
1. Alternatively (in line) sodium hydroxide addition is possible (but not preferred)
2. Unless ozone treatment is in place
3. Alternatively ion exchange is possible for Fe removal
4. If DOC ≤ 1 mg/L chlorination for disinfection is possible

In both cases the recovered water should not require further treatment besides chlorination. It is recommended that the first option, the existing fully treated water, be used for artificial recharge. The advantages of this are:

i) The water is superb quality for artificial recharge

ii) The treatment plant can be utilized in winter (when it would normally be under-utilised).

If the second option is preferred the water would require:

i) Increase in pH

ii) Reduction in DOC

iii) Reduction in iron

iv) Reduction in micro-organisms.

3.6 Engineering issues

There are two options on how water could be transferred to the aquifer.

3.6.1 Option 1 – Recharging with treated water

The first option is to recharge with treated water from the water treatment works and will involve the following (and is shown in **Figure 3.11**):

- The excess winter capacity of the treatment works will be utilised to treat Keurbooms for recharge during the 3 winter months of artificial recharge.
- A new 300 mm diameter pipeline has been installed between the water treatment works and the New Horizons reservoir. During the next financial year, the pipeline to Kwanokuthula will also be upgraded, followed by the pipeline from the main Kwanokuthula reservoir to the Western Reservoir. This infrastructure will ensure that there is the capacity to supply 50 L/s from the Western reservoir to the injection boreholes.
- This option includes the following components
  - Drilling and equipping up to 5 injection boreholes
  - Drilling and equipping up to 4 new abstraction boreholes
  - Drilling 6 monitoring boreholes
  - 950 m of injection pipelines and about 2000 m of pipelines from the abstraction boreholes
  - Chlorination only
- Implementing this option if all boreholes are necessary, will cost R12.6 million.

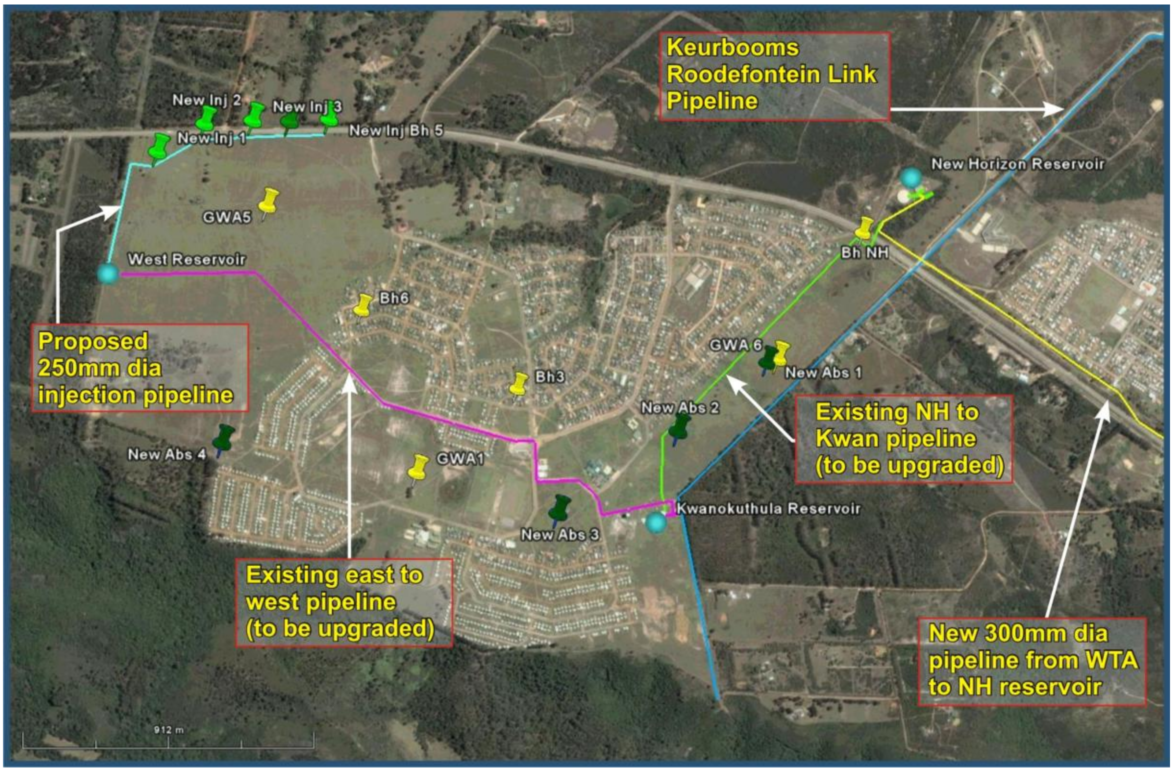


FIGURE 3.11 INFRASTRUCTURE COMPONENTS FOR OPTION 1 – RECHARGING WITH TREATED WATER FROM EXISTING WATER TREATMENT WORKS (GOOGLE EARTH, 2010)

3.6.2 Option 2 – Recharging directly with Keurbooms river water

With the second option (**Figure 3.12**), the following is relevant:

- The pipeline is limited to a flow of 35 L/s which is less than the target for recharge of 50 L/s
- Referring to the section above on water quality, the treatment required to achieve acceptable water quality is comprehensive and includes a number of different components (lime dosing, activated carbon filtration, flocculation & filtration and Ozone treatment). This would require a comprehensive treatment works that has a significant cost (both capital and for operation) and would need to be staffed and managed.
- A pipeline is required to transport the water from the Keurbooms pipeline to the injection borehole locations. The most convenient pipeline route would be adjacent to the N2 but this route is not favoured by the Bitou Municipality due to land access and servitude issues with SANRAL.
- This option includes the following components:
  - Drilling and equipping 3 injection boreholes
  - Drilling and equipping 2 new abstraction boreholes
  - Drilling and equipping 3 monitoring boreholes
  - 2400 m of injection pipelines and 1800 m of pipelines from the abstraction boreholes
  - Water treatment plant
- Implementing this option will cost R23 million.

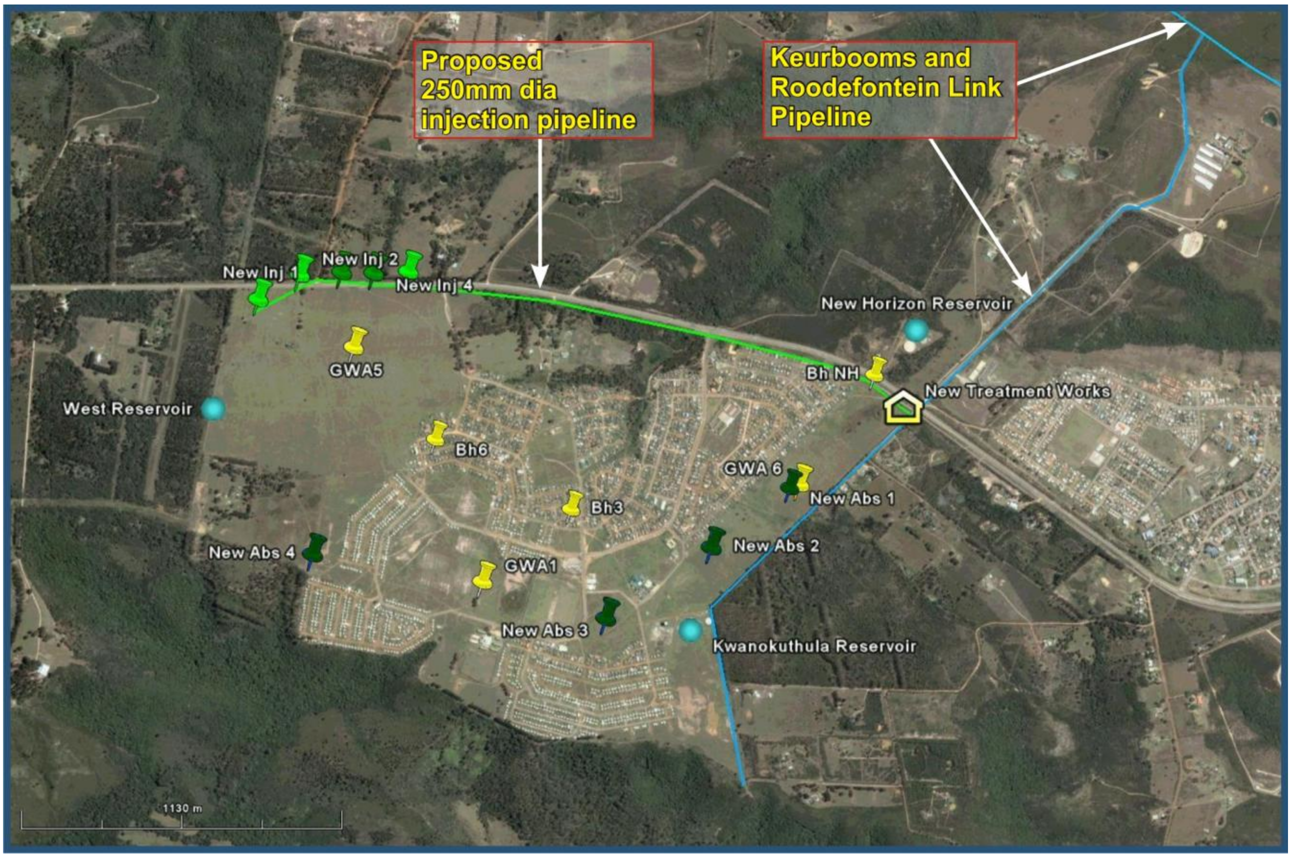


FIGURE 3.12 INFRASTRUCTURE COMPONENTS FOR OPTION 2 – RECHARGING WITH KEURBOOMS RIVER WATER AND TREATING IN NEW TREATMENT WORKS (GOOGLE EARTH, 2010)

3.6.3 Preferred option and project phasing

The preferred option is Option 1, recharging with treated water, based upon the following:

- Option 1 makes maximum use of existing infrastructure
- Option 1 uses the spare capacity of the existing treatment works and does not include the construction of a separate works
- It is the most cost effective to implement and has cheaper running costs

Phasing of Option 1 has been looked at for two reasons. The first reason is financial; it is unlikely that the full project budget would be immediately available. The second reason is to allow for the large scale testing of the artificial recharge concept and the aquifers performance. In other words, can the water levels be lowered (to make storage space available) through large-scale abstraction? Can the water be recharged? And, what volume is retained in the aquifer?

A first phase of the project has been defined and costed recharging 30 L/s rather than 50 L/s and comprising the following components:

- Drilling and equipping 3 injection boreholes
- Drilling and equipping 2 new abstraction boreholes
- Drilling 3 monitoring boreholes
- 715 m of injection pipelines and about 1800 m of pipelines from the abstraction borehole

3.6.4 Supply of the re-abstracted water

During the peak demand times when the water is being re-abstracted, the daily volume of abstracted water is more than the water requirements in Kwanokuthula - this water is needed in Plettenberg Bay town. In order to get the water to the central supply area at the water treatment plant a number of options have been considered (**Figure 3.13**). While the water will not have to be re-treated, it needs to reach the treatment plant from where it can be pumped to a number of reservoirs.

The first option is to pump the water into the Keurbooms-Roodefontein Dam link pipeline, feed the water into that dam and then abstract from the dam with the existing pipeline. The main disadvantage of this option is that 45% of the water will be lost as the Bitou Municipality only has the right to abstract 55% of the water from the dam, even if the Municipality has pumped the water into the dam from other sources.

The second option is to supply the water to New Horizons reservoir site and then gravity feed down to the treatment plant with an existing pipeline. The municipality has recently installed a new 300 mm diameter pipe from the treatment works to New Horizons and the old 200 mm diameter pipeline could be used to transfer the re-abstracted water to the treatment plant.

The third option is to use the Keurbooms-Roodefontein Dam pipeline but in reverse. In other words pump the water back along the pipeline and down to the treatment works using the spare capacity in the older backup Keurbooms pipeline.

Of the three options the second is seen as the most feasible.

Figure 3.13/…

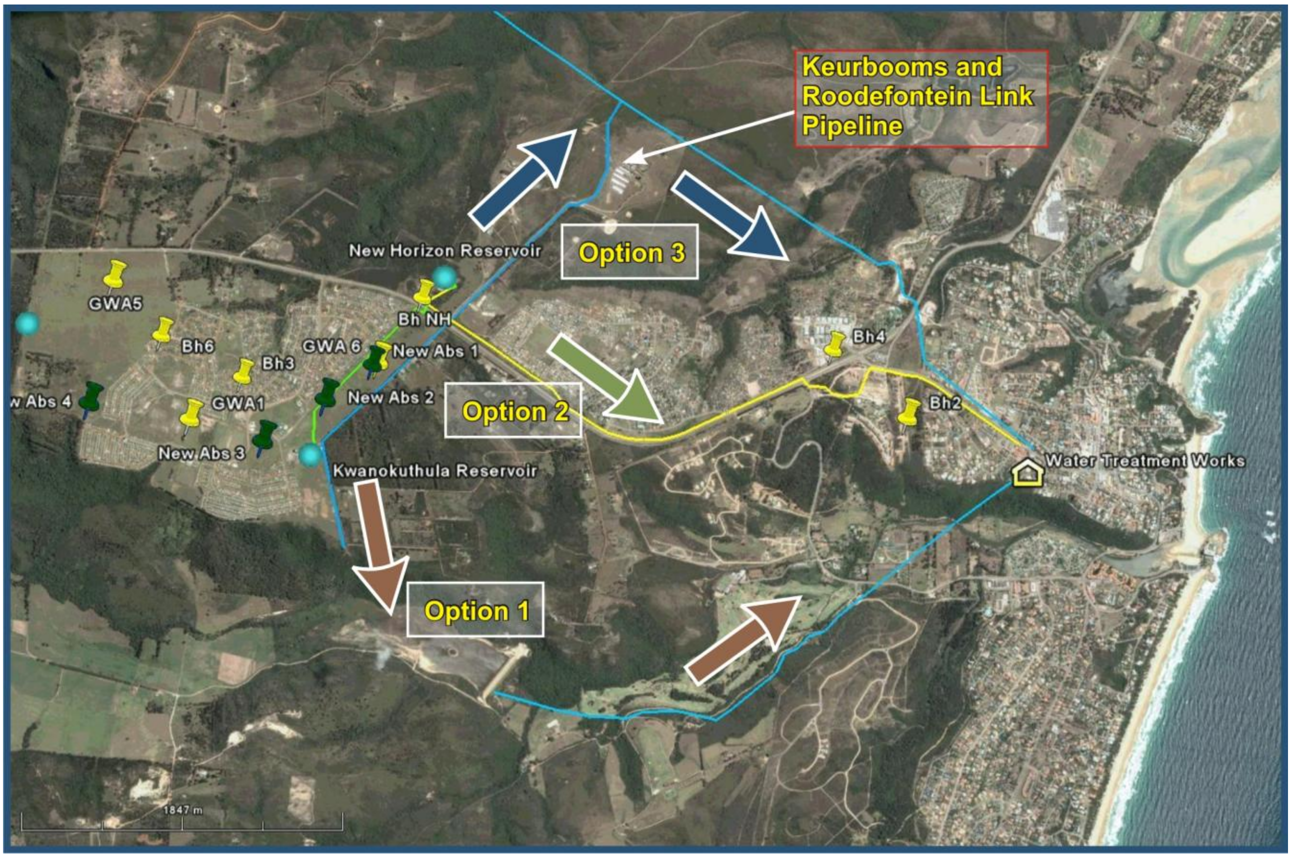


FIGURE 3.13 OPTIONS FOR THE CONVEYANCE OF THE WATER TO THE WATER TREATMENT WORKS AFTER BEING RE-ABSTRACTED (GOOGLE EARTH, 2010).

3.7 Environmental issues

No Activities under the 2010 NEMA regulations (DEA, 2010) are triggered by the planned drilling and conveyance of the water. The new pipelines are all smaller that 360 mm internal diameter, and do not exceed the limits of Section 9 of Regulation 544.

In the case of using the Keurbooms raw water as the source of supply, the treatment works would not trigger a basic assessment due to its size and because the purpose is for treating drinking water.

No storage facilities are being constructed so Section 12 of Regulation 544 does not apply.

In terms of the National Heritage Resources Act (1999), constructing a pipeline longer than 300 m makes it a requirement to inform the responsible heritage authority (Heritage Western Cape) who will stipulate if a heritage impact assessment (HIA) is required.

3.8 Legal and regulatory issues

The Bitou Municipality's Registered Use for groundwater is 362 000 m^3 per annum or 0.99 Ml/day (Murray, 2007). The intention of implementing the artificial recharge scheme is to increase the total groundwater abstraction to 5.2

Ml per day over the five peak months, which is equivalent to an annual abstraction of 780 000 m^3 per annum. This implies a shortfall in the authorization of 418 000 m^3 per annum which will need to be authorized by DWA.

Typically an artificial recharge project would include three activities that require authorisation:

1) Abstraction of the source water from a water resource (DW760 NWA Section 21a) – this is covered under the existing license for abstraction from the Keurbooms River and other sources

2) Storing of water (DW762 NWA Section 21b) - Required

3) Re-abstracting the water from the Aquifer (DW760 NWA Section 21a) – additional authorisation required

In summary, both the storing of 400 000 m^3 and the increase in abstraction to 780 000 m^3 per annum must be authorised by DWA. Two key issues will need to be addressed in the license application documentation for the storing of water. The first is that the applicant will have to prove that the water is being stored in the aquifer and that it is not leaking away in the period between the injection and the re-abstraction. Secondly, it will have to be demonstrated that the injectant water does not contain waste.

3.9 Economics

The costs have been estimated for both source options (**Table 3.6** and **3.7**) as well as for a first phase of Option 1. The costs of the options are compared below (**Table 3.5**).

TABLE 3.5 THE TOTAL CAPITAL COST OF EACH OPTION AND THE COST PER UNIT VOLUMES (ALL COSTS INCLUSIVE OF VAT).

Item	Option description	Capital Cost including VAT (Rand)	Total volume of water stored and recovered (10% losses) (m^3)	Cost of Storage (Rand/m^3)	Volume of water re-abstracted during 5 month peak period (m^3/day)	Capital cost per Ml per day (Million Rand)
Option 1	Recharge with treated water	12 601 125	357 696	35.23	2 333	5.40
Option 2	Recharge with Keurbooms water	23 058 884	214 618	107.44	1 400	16.47
Option 3	First phase of Option 1	8 532 105	214 618	39.75	1 400	6.10

The capital cost of the preferred option (Option 1) compares favourably with alternative water sources development. The cost per ML supplied per day of R5.4 million is just over a third of the current cost of desalination plants which would have a similar duty application of being used only during the peak demand periods. The operational costs of artificial recharge are also expected to be considerably less than desalination.

TABLE 3.6 IMPLEMENTATION COSTS: OPTION 1, TREATED WATER SUPPLY.

Direct Cost: Source Development	**Amount**
Injection boreholes (Drill 7; Equip 5)	R 1 823 682
Abstraction boreholes (Drill 6; Equip 4)	R 1 774 176
Monitoring boreholes (Drill 6)	R 741 456
Borehole Testing	R 450 000
Sub Total Source Development	**R 4 789 314**
Direct Costs: Infrastructure	
Equip injection boreholes (Equip 4)	R 375 037
Equip abstraction boreholes (Equip 4)	R 1 000 000
Equip monitoring boreholes (Drill 6)	R 72 000
Electricity Supply	R 105 000
Recharge Pipeline	
Pipelines 250 mm class 9 - including earthworks	R 611 325
Pipelines 200 mm class 9 - including earthworks	R 63 710
Pipelines 160 mm class 9 - including earthworks	R 43 066
Air valves, isolating valves, chambers	R 184 200
Abstraction Pipelines	
Pipelines 200 mm class 9 - including earthworks	R 27 700
Pipelines 160 mm class 9 - including earthworks	R 247 100
Pipelines 110 mm class 9 - including earthworks	R 168 000
Air valves, isolating valves, chambers	R 94 200
Monitoring equipment	R 48 480
P&G's (20%)	R 607 964
Contingencies (10%)	R 364 778
Sub Total Infrastructure	**R 4 012 559**
Indirect Costs	
Professional Fees (12.5%)	R 501 570
Hydrogeology (borehole siting, drilling and testing supervision, yield analysis & recommendations)	R 688 450
Specialist water quality study	R 32 000
Water Use Licences	R 75 000
Environmental Authorisation & monitoring	R 150 000
Management & operation training	R 33 400
O&M mentoring and GW monitoring start-up (1 year)	R 81 600
Survey & servitude registration	R 75 000
Health & Safety	R 180 000
Construction Monitoring	R 327 500
Disbursements (5%)	R 107 226
Sub Total Indirect Costs	**R 2 251 745**
Sub Total	**R 11 053 619**
VAT	**R 1 547 507**
TOTAL	**R 12 601 125**

TABLE 3.7 IMPLEMENTATION COSTS: OPTION 2, KEURBOOMS RIVER WATER SUPPLY.

Direct Cost: Source Development	Amount
Injection boreholes (Drill 4; Equip 3)	R 1 042 104
Abstraction boreholes (Drill 3; Equip 2)	R 887 088
Monitoring boreholes (Drill 3)	R 370 728
Borehole Testing	R 225 000
Sub Total Source Development	**R 2 524 920**
Direct Costs: Infrastructure	
Equip injection boreholes (Equip 3)	R 281 277
Equip abstraction boreholes (Equip 2)	R 500 000
Equip monitoring boreholes (Drill 3)	R 36 000
Electricity Supply	R 52 500
Recharge Pipeline	
Pipelines 250 mm class 9 - including earthworks	R 1 573 200
Pipelines 200 mm class 9 - including earthworks	R 127 420
Pipelines 160 mm class 9 - including earthworks	R 49 420
Pipelines 110 mm class 9 - including earthworks	R 27 720
Air valves, isolating valves, chambers	R 184 200
Abstraction Pipelines	
Pipelines 200 mm class 9 - including earthworks	R 27 700
Pipelines 160 mm class 9 - including earthworks	R 247 100
Pipelines 110 mm class 9 - including earthworks	R 168 000
Air valves, isolating valves, chambers	R 94 200
Monitoring equipment	R 24 240
Water Treatment Plant	R 9 041 855
P&G's (20%)	R 678 595
Contingencies (10%)	R 1 311 343
Sub Total Infrastructure	**R 14 424 770**
Indirect Costs	
Professional Fees (12.5%)	R 1 803 096
Hydrogeology (borehole siting, drilling and testing supervision, yield analysis & recommendations)	R 378 738
Specialist water quality study	R 32 000
Water Use Licences	R 75 000
Environmental Authorisation & monitoring	R 150 000
Management & operation training	R 33 400
O&M mentoring and GW monitoring start-up (1 year)	R 81 600
Survey & servitude registration	R 75 000
Health & Safety	R 165 000
Construction Monitoring	R 327 500
Disbursements (5%)	R 156 067
Sub Total Indirect Costs	**R 3 277 401**
Sub Total	**R 20 227 091**
VAT	**R 2 831 793**
TOTAL	**R 23 058 884**

3.10 Management and technical capacity

The management requirements of the preferred option are described in this section. The second option which entails recharging with the Keurbooms water would have a number of additional tasks associated with the management and operation of the newly constructed treatment works.

The day-to-day operation of the scheme is not difficult and should be easily performed by the normal operational staff of the municipality. The more difficult aspects of the scheme operation are in the overall management of the aquifer, the management of pumping schedules to maximise the benefits of the scheme and the monitoring required to generate the data that can be used to make informed management decisions. The management and technical tasks, their frequency and the capacity required for the tasks are listed in **Table 3.8**. The competency needed for the tasks is described and it is assumed that the municipality will have the resources to perform much of this in-house but will probably need to contract external skilled resources for some of the tasks needing specialist knowledge.

TABLE 3.8 MANAGEMENT AND OPERATIONAL TASKS

Task Description	Frequency	Responsibility / required competency
Ensure that all management and operation tasks are appropriately resourced and ensure each person understands the tasks to be performed and has the tools and resources needed to do the work. Monitor reports and regularly assess performance of all parties involved (both internal and external)	Ongoing	Municipal water manager
Develop operating rules with the aim of having the aquifer water levels at an optimum level at the onset of the recharge period to ensure maximum benefit from AR	Prior to project handover	Hydrogeologist
Ensure that the scheme is operated according to the operating rules designed to allow for maximum benefit from AR	Ongoing	Municipal water manager
Ensure ongoing groundwater management, manage consultation and communication between specialists, municipal staff and the community	Ongoing	Municipal water manager
Checking of all infrastructure components and pipework for leaks and damage and repair where required	Prior to injection event commencing	Municipal scheme operator
Design monitoring system, identify monitoring boreholes and set monitoring criteria	Prior to injection event commencing	Hydrogeologist
Design water quality monitoring system, identify monitoring points, sampling schedules and determinands to be analysed	Prior to injection event commencing	Water quality specialist
Check all flow meters and data loggers are working correctly	Prior to injection event commencing	Experienced technician
Manual readings & records of water meters, pressure gauges and water levels	Daily or as required during injection event	Experienced technician

Task Description	Frequency	Responsibility / required competency
Collect water samples and conduct field water quality tests	During injection event, based on schedule.	Experienced technician
Download data	After injection event completed	Experienced technician
Analyse data and make recommendations on future injection and abstraction schedules	After injection event completed	Hydrogeologist / water quality specialist
Assess injection efficiency and make recommendations	After injection event completed	Hydrogeologist & water quality specialist
Re-habilitate boreholes (flushing, acid cleaning)	As required	Contractor

3.11 Institutional arrangements

The Bitou Municipality is the owner, manager and operator of the scheme. The municipality is the supplier of the source water and the user of the re-abstracted water.

The license to abstract and the proposed license to store water underground are granted by DWA who have monitoring and reporting requirements that must be fulfilled in terms of the license conditions.

3.12 Implementation Stages

Table 3.9 provides a summary of the project implementation stages, the current progress with project implementation and the work still to be completed.

TABLE 3.9 PROJECT IMPLEMENTATION STAGES

Project Phase	Key activities	Status and Progress		Authorisation Requirements
Pre-feasibility phase	Identify the potential AR project and detail the information currently available	✓	Pre feasibility completed by Groundwater Africa in 2006	None
	Assess the potential AR project based on existing information	✓		
	Identify the work required for the feasibility phase and estimate the cost of producing the feasibility study	✓		
	Establish existing water use license conditions and authorisation requirements from DWA and DEAT	✓		
Feasibility phase	Undertake a detailed feasibility study including relevant testing (injection/infiltration/borehole pump tests)	✓	Feasibility study completed by Groundwater Africa in 2007 excluded testing. Some testing took place in August 2010	None. DWA permission to test obtained. Both DWA and DE&ADP informed of tests. Tests witnessed by DWA.
	Do preliminary design of the infrastructure required	✓	This document	
	Identify the phases of project implementation if phased implementation is proposed	✓	Implementation in two phases is proposed.	
	Estimate the costs of the project implementation	✓	This document	
	Identify funding sources and ensure the feasibility study complies with all requirements of the proposed funder	□	Municipality to identify suitable funding sources. Most probably internal budget or MIG or combination.	
	Compile a detailed programme			
Implementation phase	Groundwater infrastructure development & testing	✓	Partial. Some groundwater infrastructure already in place	DWA water use license for storing water to be applied for. Authorisation for increased groundwater abstraction needed. No environmental basic assessment triggered by planned activities
	Engineering detailed design, tender, construction and commissioning of AR infrastructure	□	Dependent on budget availability from the municipality	
	Set up groundwater and recharge water monitoring system	✓	Partial. Some in place, monitoring boreholes needed, finalise during implementation	
	Compile operation & maintenance procedures and train operators		To be compiled to match the designed infrastructure and monitoring system. To be done to coincide with construction completion	

Project Phase	Key activities		Status and Progress	Authorisation Requirements
Operation and maintenance phase	Performance monitoring during production		A one year mentorship period is budgeted for	Compliance monitoring and reporting
	Modified operation & maintenance procedures		To be fine tuned during mentorship period	
	Final monitoring and reporting strategy		On completion of mentorship period	

3.13 Conclusions and recommendations

The artificial recharge plan should be seen in conjunction with natural groundwater use and as a measure to augment the summer peak demand. Taking this approach, the target volumes should be:

Artificial recharge (Kwanokuthula Aquifer):	2.6 – 3.0 Ml/day over 3 months or 235 - 315 Ml
Abstraction (Kwanokuthula Aquifer):	4.8 Ml/day over 5 months or 720 Ml
Abstraction from Bh4 & Bh A'port:	0.4 Ml/day over 5 months or 60 Ml
Total:	**5.2 Ml/day over 5 months or 780 Ml**

At this stage, artificial recharge is considered a viable sub-surface storage option if the natural groundwater levels can be dropped below the natural rest level of ~60 mamsl. The proposed approach to assess the scheme potential should be:

i) Large-scale abstraction. Five months of continuous abstraction is needed to assess the storage capacity of the aquifer and to create the space for artificial recharge.

ii) A short period of rest followed by three months of artificial recharge

iii) A short period of rest followed by large-scale abstraction.

The estimated cost of the proposed option is R12.6m inclusive of VAT. It is proposed that the project be implemented in phases with the first phase having a capital cost of R8m.

4 DUNE FILTRATION IN SEDGEFIELD

4.1 Introduction

The potential for intercepting Sedgefield's treated waste water was raised by Mr Roger Parsons of Parsons and Associates Specialist Groundwater Consultants and SSI (SSI, 2009). Following a meeting in late 2009 with the Knysna Municipality, Roger Parsons and SSI engineers, it was decided to undertake an initial assessment of this option. Locations of potential boreholes were identified and a numeric model was set up using rough hydraulic parameters that were based on Roger Parsons' best estimates (actual data is not available). The model indicates that the discharged effluent can be intercepted by boreholes located south of the Waste Water Treatment Plant.

4.2 Water requirements & waste water availability

In 2009, Sedgefield's water demand was about 2 000 m^3/day, and the long-term forecasted requirements (for year 2033) are 4 500 m^3/day (SSI , 2009). Sedgefield's main water source is the Karatara River which is augmented in summer with groundwater. The water supply is also being augmented with seawater desalination, contruction of which began in late 2009.

In order to meet the projected 4 500 m^3/day demand, the following sources have been proposed (SSI, 2009):

Surface water:	1 500 m^3/day
Groundwater:	500 m^3/day
Desalination:	1 500 m^3/day
Water re-use:	1 000 m^3/day
	4 500 m^3/day

Both direct and indirect means of polishing water for potable re-use have been suggested by SSI (2009). Direct means would require upgrading the waste water treatment works (WWTW) enabling the delivery of potable water – an option that is very expensive and that requires highly skilled personnel. Indirect means imply allowing the Sedgefield Aquifer to naturally polish the water. Further treatment may be required after such polishing, but the extent of this would only be known after conducting the necessary tests. SSI (2009) identified two artificial recharge sites in the dune sands, south and south-east of the WWTW, where treated waste water could be fed into the aquifer for the purposes of indirect re-use.

While recognizing that artificial recharge already takes place, this report puts forward an option on how and where to intercept this water down-gradient of the WWTW. Should this work, it would not be necessary to convey the treated waste water to new infiltration basins. The location of the WWTW is shown in **Figure 4.1**.

4.3 Hydrogeology

Sedgefield is underlain by highly transmissive, unconsolidated, Quaternary sands (Parsons, 2009), referred to as the Sedgefield Aquifer. The thickness of the sands varies according to topography and may reach in excess of 100 m in places (the dune immediately south of the WWTW reaches about 100 m above the low-lying area immediately north of the WWTW). The WWTW is located at about 60 mamsl. In all likelihood, rocks of the Table Mountain Group underlie the sands (Parsons, 2009). Natural groundwater flow in the sands is from north to south towards the sea along a very gentle hydraulic gradient of about 0.001 – 0.004 (Parsons, 2009).

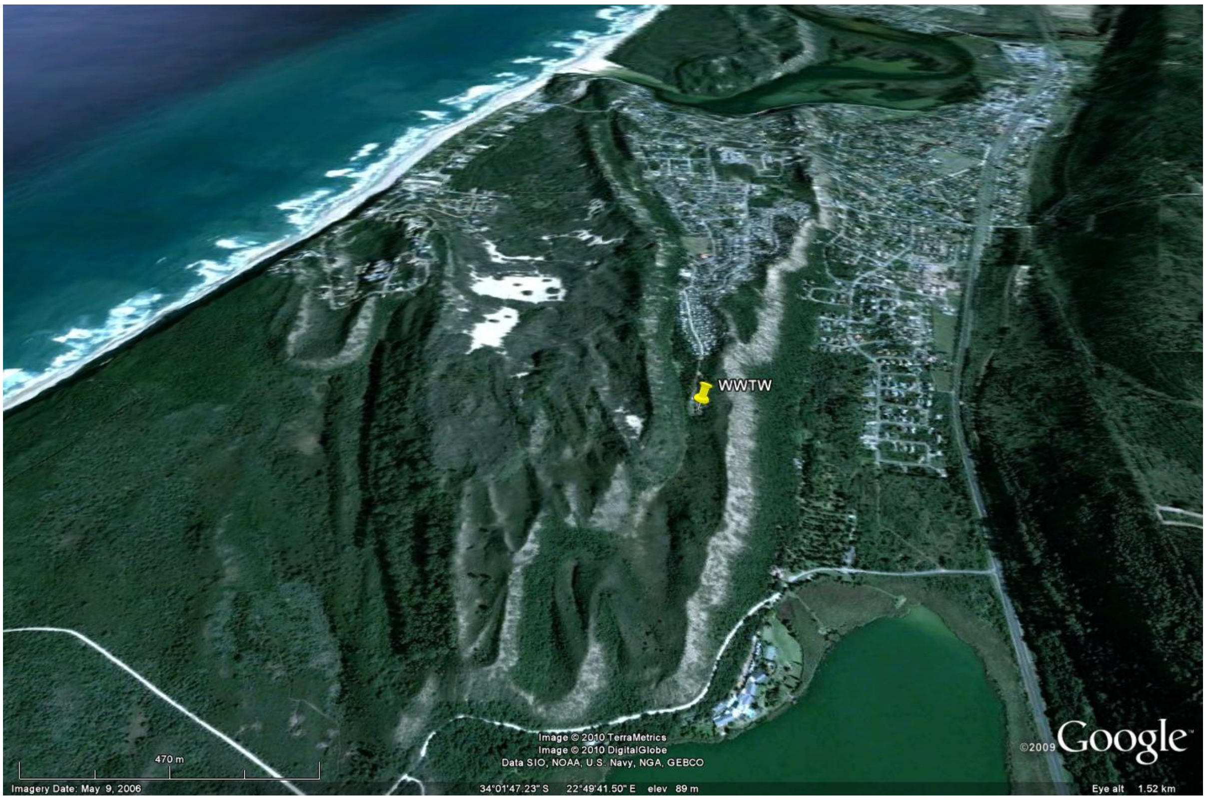


FIGURE 4.1 LOCATION OF THE WWTW IN THE DUNES SE OF CENTRAL SEDGEFIELD. GROENVLEI IS IN THE FOREGROUND WITH NORTH TO THE RIGHT OF THE PHOTO (GOOGLE EARTH, 2010).

4.4 Groundwater flow around the WWTW

Three monitoring boreholes were drilled in and around the WWTW at the time the plant was commissioned in 1997 and their average water level at that time was 2.8 mamsl (**Figure 4.2**). This is the same as the average water level in Groenvlei (DWA monitoring site K4R001, 1980 – 2006). Parsons (2009) shows that groundwater in the Sedgefield Aquifer and Groenvlei are hydraulically well linked; these water levels support this as well as the contention that the sands are highly transmissive.

Groundwater levels from the three monitoring boreholes have risen by 1 m on average since the commissioning of the WWTW (**Figure 4.2**). This can be attributed to infiltration from the WWTW and the two maturation ponds immediately below the WWTW. The location of the three boreholes is shown in **Figure 4.7**.

This fairly small mound (~1 m in height above the natural groundwater levels) can be attributed to the high permeability of the sands. Owing to the ease at which the water can move through the sand, it is probable that the shape of the mound will be evenly distributed around the points of infiltration, which from the borehole water level data appears to be a zone from the WWTW to the second maturation pond.

Because the natural hydraulic gradient is low, the mound can be assumed to move out fairly evenly in all directions, although with time, the water will ultimately make its way down the gentle hydraulic gradient to the sea.

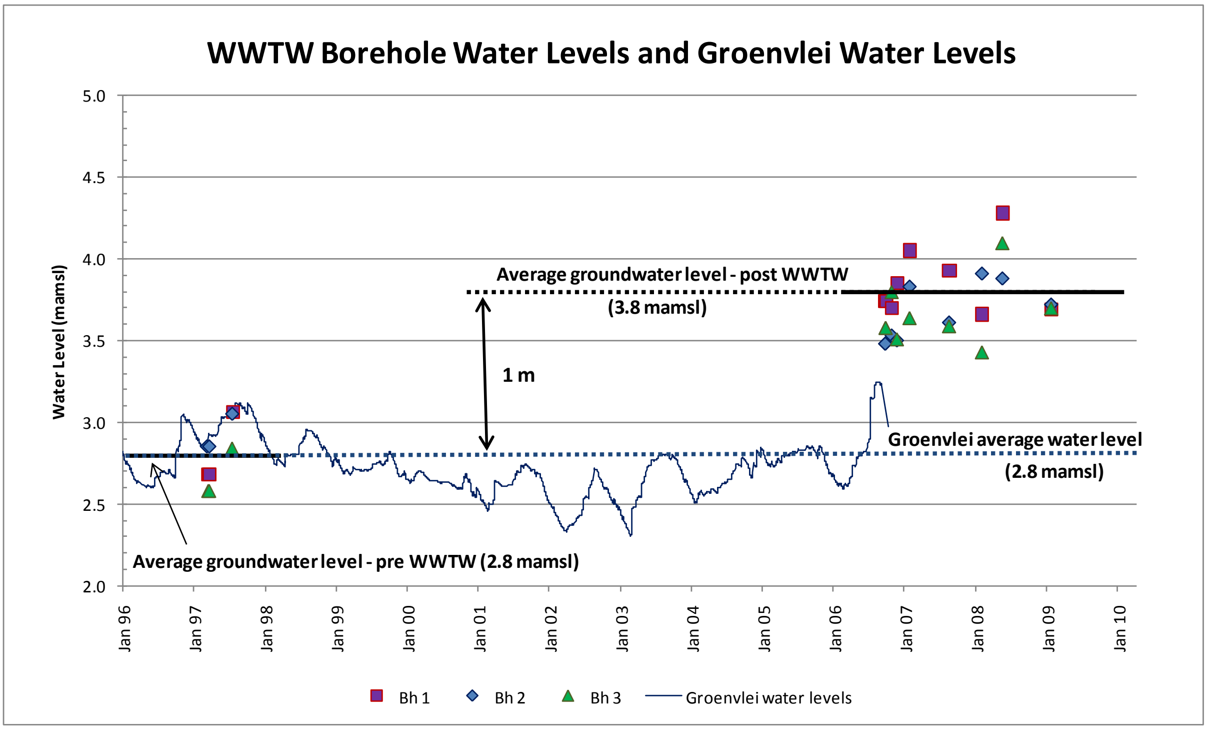


FIGURE 4.2 WWTW WATER LEVELS AND GROENVLEI WATER LEVELS

4.5 Groundwater quality below the WWTW

The change in concentrations and the variability of concentrations of salinity, potassium and nitrate from when the WWTW started operating to more recent times indicate that the water from the three WWTW monitoring boreholes does include water discharged from the WWTW (**Figures 4.3 -4.5**). The water quality confirms that a "mound" does exist below the WWTW and not natural groundwater levels.

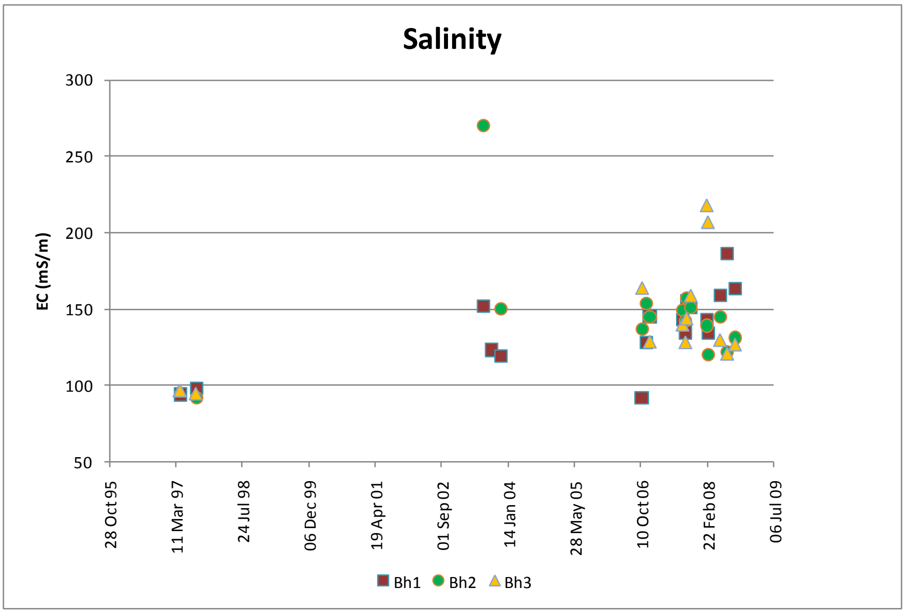


FIGURE 4.3 SALINITY OF BOREHOLE WATER AT THE WWTW

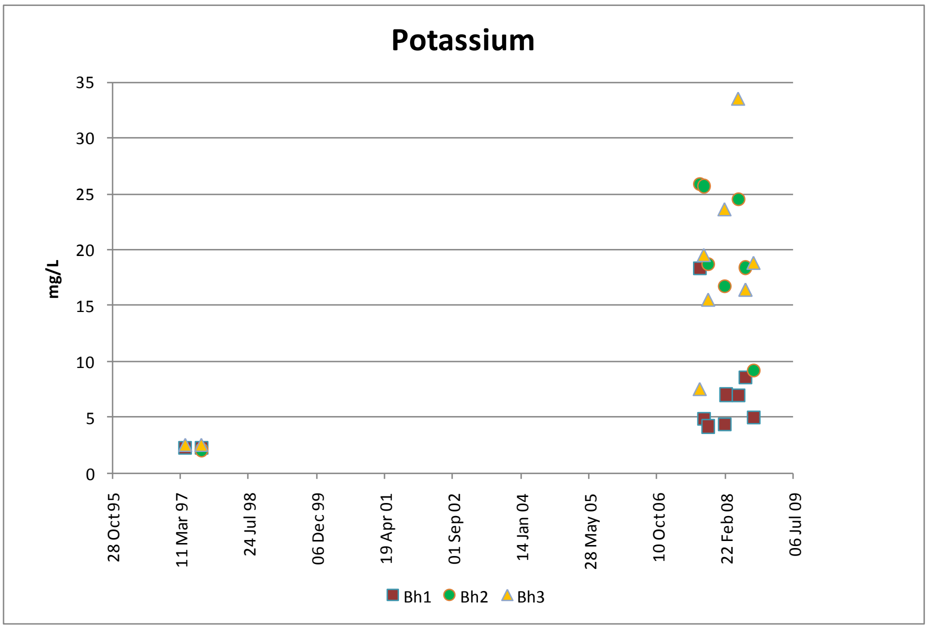


FIGURE 4.4 POTASSIUM CONCENTRATION OF BOREHOLE WATER AT THE WWTW

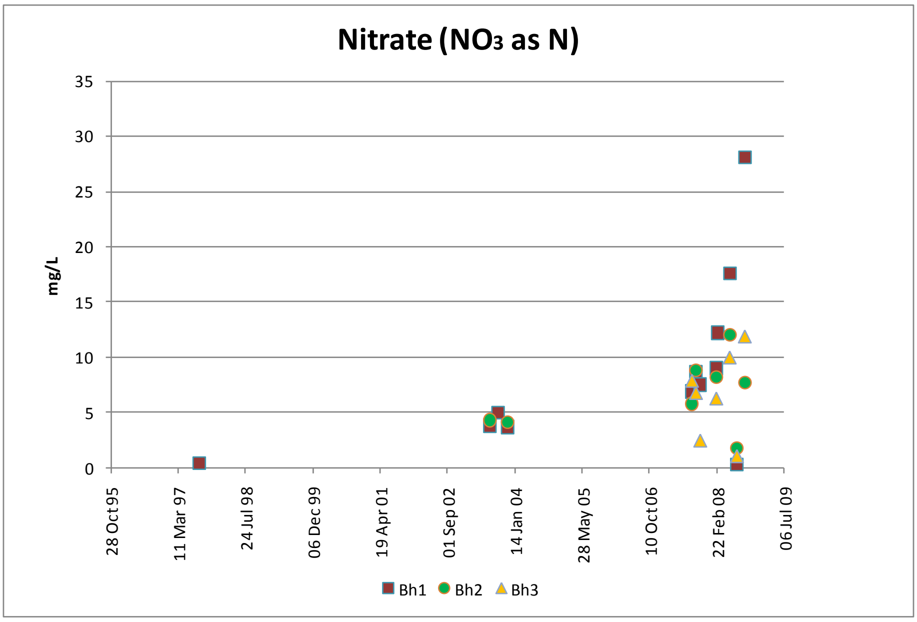


FIGURE 4.5 NITRATE CONCENTRATION OF BOREHOLE WATER AT THE WWTW

4.6 Considerations for intercepting the treated waste water

A key feature when intercepting the infiltrated waste water is that abstraction should not result in drawing water into the wellfield from Groenvlei. Pump intake depths should thus not be too deep and preferably at or slightly below the level of Groenvlei, depending on the distance between the abstraction point and the vlei. From the water levels shown in **Figure 4.2** it is evident that there is little available drawdown as the water level of the mound is only about 1 m above that of the vlei.

The salinity of the vlei's water ranges from 350 – 550 mS/m (DWA monitoring site K4R001) whereas the salinity of the boreholes at the WWTW is generally between 100 – 200 mS/m. (**Figure 4.3**). Irrespective of the location of future abstraction points, salinity and water level monitoring between the wellfield and the vlei would be necessary to ensure flow is not induced from the vlei to the wellfield.

Three options were considered regarding the interception of the treated waste water. These are summarized in **Table 4.1** and **Figure 4.6**.

TABLE 4.1 OPTIONS FOR INTERCEPTING INFILTRATED WASTE WATER

Option	Description	Advantages/ Disadvantages
1a. Wellpoints north of the WWTW	This is the lowest lying area closest to the WWTW and the only site where relatively cheap wellpoints could be installed. They would need to be installed in a line along the base of the dune at about 12 mamsl (higher if possible, with the base of the wellpoints being at the level of Groenvlei). Assuming each wellpoint yields 0.3 – 0.5 L/s it would require ~20 – 30 wellpoints to abstract the current capacity of the WWTW (750 m^3/day), and ~50 – 75 wellpoints to abstract the planned capacity of the WWTW (2 000 m^3/day).	**Advantages**: Wellpoints are cheap, and the capacity of the abstraction system can easily be expanded by adding more wellpoints. Disadvantages: Private land. It may not be physically possible to induce the treated waste water from below the WWTW to the wellpoints.
1b. Boreholes north of the WWTW	These could be placed further up the dune from the wellpoint sites with the base of the boreholes being at or just below the level of Groenvlei. Because this area is close to Groenvlei it would be best to pump at low rates to ensure limited drawdown. Assuming each borehole yields 1 L/s it would require 9 boreholes to abstract the current capacity of the WWTW (750 m^3/day), and 23 boreholes to abstract the planned capacity of the WWTW (2 000 m^3/day)	**Advantages**: Boreholes can be drilled deeper, pumps can be inserted to greater depths and thus water can be induced to flow to boreholes more easily than wellpoints. **Disadvantages**: Private land. It may only be possible to induce the flow of treated waste water to the boreholes by placing the pumps well below the Groenvlei level, and this may also induce flow from the vlei.
2. Boreholes south of the WWTW	Install boreholes in a line south of the maturation ponds, with their base being just below the level of Groenvlei. Assuming each borehole yields 3 – 5 L/s it would require 2 – 3 boreholes to abstract the current capacity of the WWTW (750 m^3/day), and ~5 – 8 boreholes to abstract the planned capacity of the WWTW (2 000 m^3/day).	**Advantages**: Hydrogeologically, this is the most realistic option. The natural groundwater flow is in this direction, and likewise, the flow of infiltrated, treated waste water is in this direction. **Disadvantages**: Private land.
3. Boreholes west of the WWTW	Install boreholes in cluster east of the maturation ponds, with their base being just below the level of Groenvlei. Assuming each borehole yields 3 – 5 L/s it would require 2 – 3 boreholes to abstract the current capacity of the WWTW (750 m^3/day), and ~5 – 8 boreholes to abstract the planned capacity of the WWTW (2 000 m^3/day).	**Advantages**: The boreholes could be located on municipal land, e.g. at the edge of the sports field. **Disadvantages**: The natural groundwater flow is not in this direction, and it may not be possible to induce flow to this area. The water could get contaminated as it flows below the settlement.

FIGURE 4.6 POSSIBLE LOCATIONS OF ABSTRACTION SITES TO INTERCEPT THE TREATED WASTE WATER (GOOGLE EARTH, 2010)

Option 1 would only be possible if the water levels were drawn down considerably in this area, thereby inducing flow from below the WWTW to the boreholes. The natural groundwater flow direction is towards the sea (north to south), thus the hydraulic gradient would have to be reversed in order to induce flow in a northerly direction. Any attempt to do this could also lead to groundwater flow from Groenvlei being induced towards this area, and for this reason, Option 1 was discarded.

Options 2 and 3 were modeled using a simple numeric model. The aim was to establish whether Option 3 is worth pursuing, and if not, how far south from the WWTW the boreholes should be located if Option 2 is the only viable way forward.

4.7 Boreholes south of the WWTW: The only viable option

Dr Ingrid Dennis undertook the task of modeling the aquifer with the limited data that is available. The finite difference numerical model used was MODFLOW. Her report is contained in **Appendix 1**.

Key hydrogeologic assumptions include a sand transmissivity value of 300 m^2/day and a specific capacity of 0.2. The boreholes were located in three clusters and all were given yields of 5 L/s of continuous abstraction (**Figure 4.7**). The three borehole clusters included:

- The W-boreholes, located about 560 m west of the WWTW
- The SA boreholes located about 200 m south of the WWTW

- The SB boreholes located about 350 m south of the WWTW.

FIGURE 4.7 GROUNDWATER ABSTRACTION AREAS: SA, SB AND W (GOOGLE EARTH, 2010)

Seven scenarios were run with abstraction from different borehole clusters and no abstraction at all. The main conclusions are:

- No water (or virtually no water) from the WWTW reaches the western boreholes (W-boreholes).
- It takes the water from the WWTW about 120 days to reach the SA boreholes if they are all pumped at 5 L/s.
- It takes the water from the WWTW about 700 days to reach the SB boreholes if they are all pumped at 5 L/s.

The flow between the WWTW and the W-boreholes was also modeled using a mass transport model to verify whether flow could be taking place in this direction with abstraction from the W-boreholes. From the parameters used in the model, it became apparent that water from the WWTW cannot be induced to flow to the western boreholes.

The key finding of the study is that treated waste water from the WWTW can be intercepted by locating boreholes south of the WWTW. The recommended distance of the abstraction boreholes should be about 350 – 400 m south of the WWTW.

4.1 Engineering issues

The layout of the proposed scheme is shown in **Figure 4.8.** Two different pipeline routes were investigated and the proposed route was chosen to take advantage of the additional capacity in the pumped line from the desalination plant to the main reservoir. Furthermore, the alternative route would require going through environmentally sensitive areas.

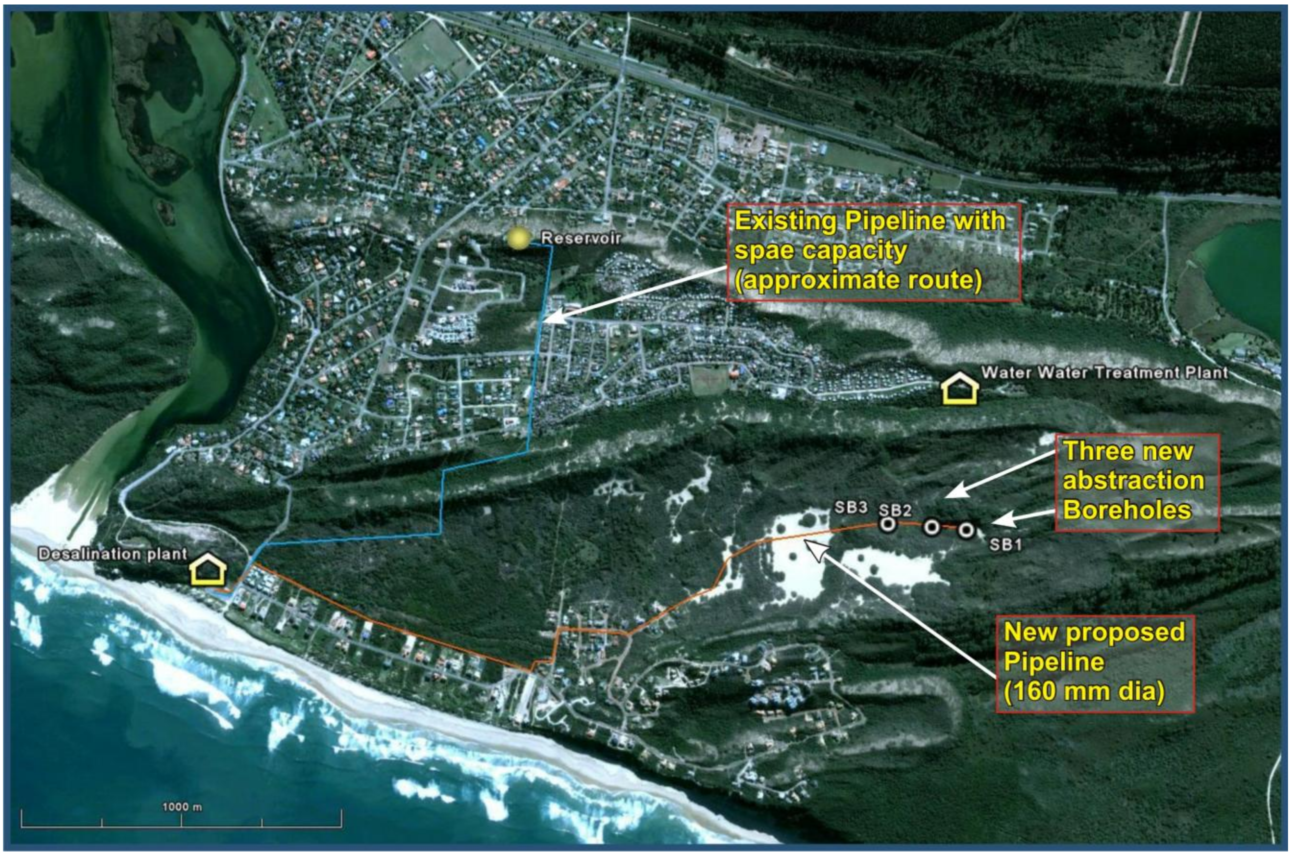


FIGURE 4.8 LAYOUT OF PROPOSED NEW BOREHOLES AND PIPELINE (GOOGLE EARTH, 2010)

4.2 Environmental issues

No Activities under the 2010 NEMA regulations (DEA, 2010) are triggered by the planned drilling and conveyance of the water. The new pipeline is smaller than 360 mm internal diameter, and does not exceed the limits of Section 9 of Regulation 544. However, the boreholes and pipelines are located in a coastal dune system and the DEA&P officials in the area may require an assessment on the impact of the scheme.

In terms of the National Heritage Resources Act (1999), constructing a pipeline longer than 300 m makes it a requirement to inform the responsible heritage authority (Heritage Western Cape) who will stipulate if a heritage impact assessment (HIA) is required. Because of the pipeline location in the dunes and the potential archeological significance of this area, it is likely that a heritage impact assessment (HIA) will be required.

4.3 Legal and regulatory issues

Recharging an aquifer with water containing waste is a controlled activity and needs to be licensed under the National Water Act, Section 21f. However it is not clear whether a license is required as the recharge of the aquifer has been happening since the establishment of the plant and would theoretically be classified as an existing lawful use (which, if not already done so, does need to be registered with DWA). The licensing requirements need to be discussed with the DWA personnel in the area.

Water use authorisation will be required for groundwater abstraction.

4.4 Economics

The cost of drilling, equipping and conveying water from the three boreholes is given in **Table 4.2**.

TABLE 4.2 IMPLEMENTATION COSTS

Direct Cost: Source Development	**Amount (Rand)**
Borehole drilling: 3 Production boreholes	R 510 000
Borehole drilling: 3 Monitoring boreholes	R 510 000
Wellpoint installations (No 2)	R 4 000
Borehole Testing	R 75 000
Water quality testing	R 8 000
Sub Total Source Development	**R 1 107 000**
Direct Costs: Infrastructure	
Pump installation Borehole SB1	R 150 000
Pump installation Borehole SB2	R 150 000
Pump installation Borehole SB3	R 150 000
Electricity (from WWTW)	R 227 500
Pipelines 160 mm class 12 - including earthworks	R 840 140
Pipelines 110 mm class 9 - including earthworks	R 21 840
Pipelines 90 mm class 9 - including earthworks	R 15 070
Air valves, isolating valves, chambers	R 127 800
Chlorination	R 82 500
Monitoring equipment	R 75 000

Direct Cost: Source Development	**Amount (Rand)**
P&G's (20%)	**R 367 970**
Contingencies (10%)	**R 220 782**
Sub Total Infrastructure	**R 2 428 602**
Indirect Costs	
Professional Fees (12.5%)	R 303 575
Hydrogeology (hydrocensus, borehole siting, drilling and testing supervision, yield analysis, recommendations & model upgrade)	R 251 600
Specialist water quality study	R 32 000
Water Use Licences	R 75 000
Environmental Authorisation & monitoring	R 150 000
Management & operation training	R 33 400
O&M mentoring and GW monitoring start-up (1 year)	R 81 600
Survey & servitude registration	R 50 000
Health & Safety	R 150 000
Construction Monitoring	R 262 000
Disbursements (5%)	R 69 459
Sub Total Indirect Costs	R 1 458 634
Total (excl VAT)	**R 4 994 236**
VAT	**R 204 209**
TOTAL	**R 5 198 445**

The total capital cost of the scheme is R5.2 million and the daily yield from the scheme is expected to be 1.3 Ml. This equates to a capital cost of R4 million per Ml per day which compares favourably with alternatives like desalination which cost about R15million per ML per day or nearly four times the cost of the artificial recharge scheme.

4.5 Conclusions and Recommendations

Although there is little data on the hydraulic properties of the Sedgefield Aquifer, it is nonetheless safe to assume that the boreholes located down-gradient of the waste water treatment works (WWTW) would intercept both discharged water that infiltrates from the WWTW and groundwater that naturally flows towards the sea. The numerical model used to locate abstraction boreholes and to estimate travel times of the treated effluent shows that boreholes located about 350 – 400 m south of the WWTW's maturation ponds would allow for a travel time of about 700 days from the WWTW which would be ample time for further treatment in the aquifer. It is expected that only chlorination will be necessary when the water is supplied from the boreholes for domestic consumption.

Assuming three boreholes are drilled and each yield 5 L/s of continuous supply then this scheme would yield 1.3 Ml/day of groundwater that is a blend of artificially recharged treated waste water and natural groundwater. The scheme would cost in the order of R 4.7M including VAT.

In order to properly design the scheme, the following steps are recommended:

Phase 1

1. Establish whether groundwater abstraction from the dune field south of the WWTW would be allowed when considering environmental regulations. If not, the planned scheme cannot be implemented.
2. If yes, inform DWA of the intention to develop this Soil Aquifer Treatment scheme.

Phase 2

1. Obtain recent WWTW data (and update project records): Flow into the plant, discharge water quality and groundwater levels and quality from the three monitoring boreholes.
2. Equip monitoring borehole Bh 3 at the WWTW with a data logger that measures water levels and electrical conductivity. Place the logger just below the water level and opposite the well screen.
3. Conduct a hydrocensus of the area: Obtain groundwater levels and quality from boreholes ~500 m north, east and west of the WWTW and between the WWTW and the coast. Estimate groundwater abstraction from this area.
4. Install two wellpoints or boreholes between the WWTW and Groenvlei and equip them with data loggers that record water levels and salinity. They could be placed 100 - 200 m apart in the caravan park. Place the loggers just below the water level and opposite the well screens.
5. Drill three potential production boreholes about 350 m south of the WWTW (i.e. in the area around the SB boreholes).
6. Drill three monitoring boreholes:
    a. Between the three production boreholes and the WWTW (if practically possible). Aim: Early warning on water quality deterioration from the WWTW.
    b. Between the production boreholes and the sea (in the depression below south of the production boreholes). Aim: Early warning on sea water intrusion.
    c. East of the production boreholes (if practically possible). Aim: Monitoring point on the Groenvlei side of the production boreholes.
7. Test pump the boreholes and establish suitable production yields. Establish whether additional boreholes are required to obtain the target yield.
8. Survey the elevations of all the existing and new boreholes and the new wellpoints.
9. Finalise the design and costing for the scheme.
10. Monitor the water levels and water quality and assess the existing (not pumped) groundwater flow and water quality status.
11. Apply to DWA for a license to implement the scheme.

Phase 3

1. Equip the boreholes and install the conveyance infrastructure.
2. Pump the boreholes and monitor water levels and water quality.

5 THE WAY FORWARD AFTER INJECTION TESTS IN LANGEBAAN

This report was compiled by Dr G Tredoux with input from Dr R Murray.

5.1 Introduction

The Langebaan Road Aquifer System (LRAS) consists of three overlying aquifers. The main productive aquifers of the LRAS are found in the Cenozoic sediments overlying the bedrock. In the well field area these sediments attain a thickness of nearly 80 metres. A clayey aquiclude several metres thick, extending over a large part of the area, effectively separates the sediments into an unconfined and a confined aquifer. Fractures in the bedrock do yield water and thus the bedrock itself can also be considered an aquifer. Water quality in the unconfined aquifer varies while the confined aquifer generally yields good quality water except in the down-gradient areas where salinity increases significantly. The unconfined aquifer is generally intended for the use of the local farmers while the confined aquifer is mainly reserved for town supply. Some farmers do have access rights to the confined aquifer. A production well field for town supply was developed in the confined aquifer in the west near the Langebaan Air Force Base for the use of the West Coast District Municipality (WCDM).

The conceptual groundwater flow in the aquifer is set out in **Figure 5.1** showing a simplified schematic east-west profile of the LRAS in the area between Hopefield in the east, and Langebaan Road and the well field in the west.

Environmental isotope studies indicated a relatively high “age” of up to 20 000 years or more found for the groundwater in the well field area (Weaver & Talma, 2000; 2003; Tredoux & Talma, 2009). This is indicative of a slow exchange of groundwater possibly due to limited recharge and/or discharge. Mathematical simulation modelling by Timmerman (1987) indicated an annual yield of 6.3 x 10^6 m^3/a. However, Timmerman cautioned that abstraction at this rate may lead to saline water intrusion. The Department of Water Affairs (DWA) granted a permit to the WCDM for abstracting 1.46 x 10^6 m^3 annually but this lead to a ~10 m lowering of the piezometric levels in the vicinity of the well field. Although the level was still well above the confined aquifer and despite indications that the well field could sustain the allocated abstraction the Monitoring Committee decided to reduce the abstraction rate by 10%. The concern regarding the sustainability of the abstraction led to the planning and execution of injection tests.

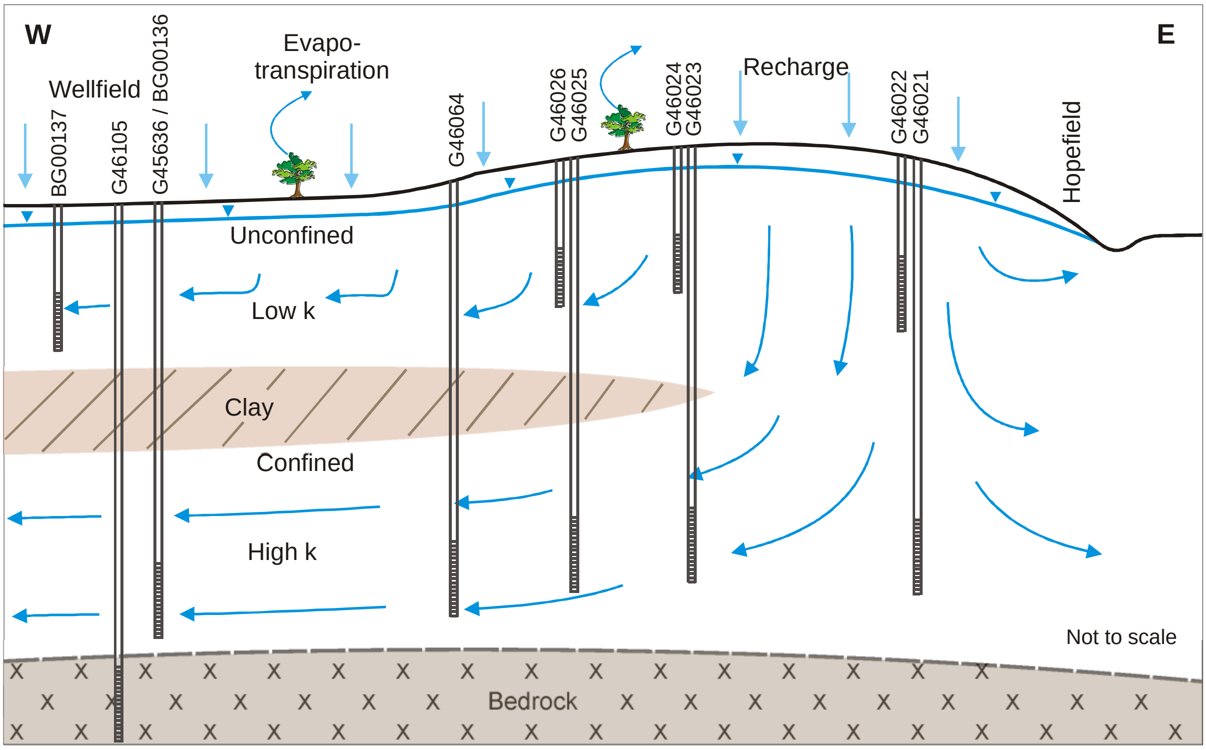


FIGURE 5.1 CONCEPTUAL GROUNDWATER FLOW IN THE LANGEBAAN ROAD AQUIFER SYSTEM

5.2 Langebaan Road pilot injection tests

A borehole injection test was conducted on borehole BG00136 between September and November 2008. The injection rate ranged up to ~30 L/s, but stabilised at ~10 L/s. The total volume injected was 76 636 m^3.

The purpose of the pilot injection tests was twofold, i.e. to:

- determine the optimum injection infrastructure and injection parameters, and
- obtain monitoring data for interpretation of the potential impacts of the envisaged longer term injection programme on the aquifer.

For addressing the second aim, the primary aquifer ecological impacts were interpreted to refer to the chemistry and microbiology in the aquifer and no attempt was made to determine the presence of any potential stygofauna.

The results of the injection runs proved that the confined aquifer could be recharged using a relatively low injection pressure of 100 to 200 kPa. Water quality results were also favourable and bacterial changes seemed to be reversible once injection ceased.

The Langebaan Road Aquifer borehole injection test

Top left: Louise Soltau of the CSIR monitoring injection pressures and water quality

Top right: The injection borehole

Bottom left: A monitoring borehole in the foreground with the injection borehole in the background

Bottom right: Fanus Fourie (DWA) and Louise Soltau at the injection borehole

As a whole the tests were subject to certain limitations as set out below.

Test limitations

For artificial recharge to succeed, the optimum location and mode of recharge needs to be identified. The injection point should generally be located as far up-gradient in the aquifer as possible in order to maximise the storage. This is critical for a confined aquifer as the storativity is very small and the ideal position will be to inject as close to, if not within, the natural recharge area itself where additional water can be stored in the unconfined part. This was not possible at Langebaan Road as budget constraints dictated that the artificial recharge pilot test should be carried out in an area close to existing infrastructure to eliminate the high cost of pipeline construction. Hence the injection borehole was drilled in the WCDM well field and only a short link was needed from the nearest production borehole to the injection borehole. This limitation meant that the ideal injection infrastructure could not be determined for this aquifer.

Overflowing boreholes

Before the WCDM well field came into operation several boreholes in the down-gradient area were free-flowing. Artificial recharge by injection restored the aquifer pressure and within six to eight weeks these down-gradient boreholes were overflowing. Without capping of these boreholes the piezometric head cannot increase sufficiently for utilizing the storage in remote, unconfined areas, unless the injection borehole is located much further up-gradient in the aquifer, i.e. close to the recharge area.

Well field recovery in progress throughout injection

Pilot injection tests were carried out in 2008 and 2009 and in both instances the WCDM discontinued abstraction from the Langebaan Road well field shortly before injection started. Piezometric level recovery in a confined aquifer is a gradual process as the aquifer remains filled with water while changes in pressure are propagated over large distances. Recovery ultimately requires the movement of water in remote areas. Recovery will, therefore, continue for six months or longer. As a result, aquifer recovery continued throughout both injection periods until abstraction was resumed. The recovery process masked the hydraulic response to injection and identification of such a response succeeded only during the second run at certain boreholes near the injection site.

Duration of the tests

The injection test duration was limited to the period that the WCDM could afford to interrupt the groundwater abstraction and to provide treated drinking water for injection not to jeopardize their town water supply requirements. Detecting a piezometric response at a distance of a few km was difficult, as explained above, and the interruption of injection added a further complication.

Microbiological changes

The various populations of microorganisms were affected by the injection of oxygenated water into an anoxic aquifer and it was planned to determine whether the changes could be reproduced during the second run. The injection was, however, interrupted after less than four weeks before the observations could be completed. It would seem that the populations of microorganisms adjusted to the redox conditions and it is possible that the original conditions will be reinstated once injection is discontinued.

5.3 Problems and unresolved issues

Condition of deep exploration and monitoring boreholes

Over the past thirty years a large number of boreholes were drilled into the confined aquifer. A large proportion of the exploration and monitoring boreholes were drilled by Timmerman (1987) also near the present WCDM well field and in the down-gradient area (**Figure 5.2**). For most of these boreholes their present condition is unknown. Boreholes drilled into the confined aquifer present a risk of overflowing when pressures increase. Losses will occur once injection takes place. In other cases, where the head is well below surface, leaking or short circuiting between aquifers may occur when the casings become corroded or damaged, again adding to the losses from the confined aquifer. The extent of this problem is unknown and it only manifested in the form of overflowing boreholes during the pilot injection. Two boreholes drilled into the confined aquifer, G29823 and G29824, which were intended for piezometric level observation were discovered to have collapsed and were filled with sand.

Natural Recharge/Safe yield

The Langebaan Road Aquifer is located in a low rainfall area with a mean annual precipitation of 260 mm/a. The sustainable yield of the aquifer is limited by the natural recharge. Timmerman (1987) studied the LRAS and concluded after a simulation modelling exercise that the maximum long term yield of the confined aquifer was 6.32 x 10^6 m^3/year but cautioned that abstraction at this level could lead to saline water intrusion. He added:

"One of the important input data of a mathematical model is recharge. No reliable recharge studies were available for the investigation area."

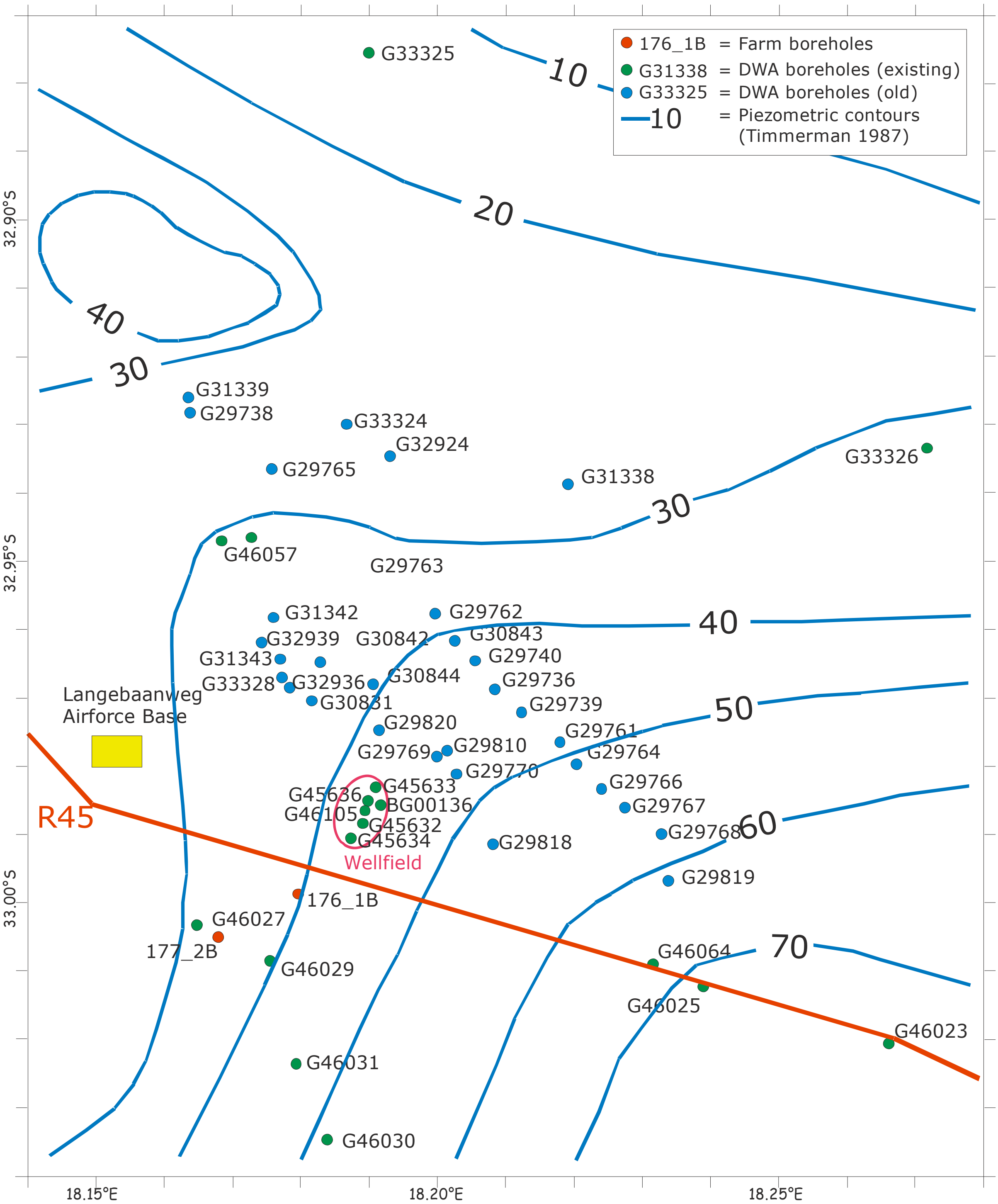


FIGURE 5.2 EXPLORATION AND OTHER BOREHOLES IN THE VICINITY OF THE WCDM WELL FIELD

This key point has not been addressed successfully yet since that time. Abstraction of 1.46 x 10^6 m^3, representing only 25% of the estimated safe yield already seemed to over tax the aquifer at the Langebaan Road well field. Although water levels were seemingly continuing to drop it may be possible that an over cautious approach was followed. This was necessary to allay fears that leakage from the upper aquifer would jeopardize the farmers' water supply. In the meantime it has been proven beyond any doubt that interaction between the confined and unconfined aquifers over most of the area is insignificant or non-existent. This means that now it could be attempted to further stress the aquifer by increasing abstraction from the well field over the short term but only to the extent that the head in the confined aquifer remained above the top of the clay aquiclude. This stress test will need to be subject to detailed observations over a wide area followed by evaluation of the piezometric response, similar to a pumping test.

Delineation of natural recharge area

The expected natural recharge area for the confined aquifer is only vaguely known from the work of Timmerman 20-30 years ago. It will be necessary to study all the relevant DWA reports of that period and work through the logs/details of all boreholes on the south-eastern side near Hopefield to find out where the clay layer is absent, patchy, or sufficiently permeable to allow recharge. It has to be in an area where the topography is sufficiently elevated to allow for a head to develop as the water needs to be conveyed through several km of sand below the confining layer to reach the Langebaan Road area. Hence the groundwater age in the order of 20k years seems conceivable.

There is also a slim chance for recharge from an area just north-west of Hopefield to reach the WCDM well field area. However, what may be complicating the issue is the large number of boreholes Timmerman drilled in the area north-east and north-west of the present well field. It is considered plausible that at least some of the deep boreholes may be short-circuiting the two aquifers causing water from the confined aquifer to be lost into the overlying unconfined aquifer.

It is intriguing that many boreholes were drilled down-gradient (both by Timmerman and Van Kleef/Van Wyk) in an attempt to define the hydrogeological processes in that area (also with regard to bedrock faults). The key area where natural recharge should take place was largely ignored apart from the row of monitoring boreholes installed in the direction of Hopefield along the R45 by DWA on recommendation by Weaver & Fraser (1998).

Recharge rate determination

Once the recharge area is delineated more accurately rainfall and water level monitoring points need to be established in that area to determine the recharge rate to the confined part of the LRAS. Mr Eddy van Wyk has several rain gauges and loggers in pairs of boreholes in the area, one set of which is in the direction of Hopefield but in an area where the aquifer is still confined. Mr van Wyk needs to be consulted to determine what can be deduced from these data as it will guide the efforts to set up the monitoring equipment further afield in the "recharge area".

Understanding the flow regime

Overall: a much better understanding of the aquifer flow regime is required before MAR can be attempted successfully. A detailed study of the water level responses at all the monitored boreholes is needed to refine the aquifer hydraulic response and flow regime. In parallel an interpretation of the hydrochemistry in the confined aquifer is needed which may entail more extensive groundwater sampling to further the understanding of these relationships.

Groundwater flow modelling

A preliminary model developed by Nebo Jovanovic (Tredoux & Engelbrecht, 2009) needs to be improved/redesigned using the results of the flow regime interpretation. As it is presently set up, it simulates the confining layer as an aquitard which may in fact be an aquiclude.

5.4 Successful artificial recharge of the aquifer

At this stage, and without yet fully understanding the natural recharge process and areas, it is considered to be possible to recharge the confined aquifer, either by borehole injection or basin recharge.

The best areas for pilot testing borehole injection would appear to be hydraulically up-gradient, close to the natural recharge area. The injection sites should be located along the main water supply pipeline from Withoogte to the coast, for example, near borehole pairs G46023 & G46024, and G46021 & G46022, in order to have easy access to the recharge water source and to minimise infrastructure costs. In these areas the effect of abstraction from the WCDM well field should be negligible and no interruption of abstraction will be needed during injection.

Alternatively, once the recharge area is well conceptualised basin recharge in the natural recharge area should be considered.

5.5 The way forward

The way forward is captured in the points below:

1. *Delineate recharge area(s)*

Delineating the recharge area(s) of the confined part of the Langebaan Road Aquifer System more precisely has top priority. All possible approaches should be followed including reinterpretation of the existing borehole logs and confirmation drilling where needed. Subsequently, monitoring equipment will need to be set up for monitoring rainfall and water level responses in that area.

Once the recharge area is well conceptualised basin recharge in the natural recharge area should be considered as an alternative. This robust technique will be preferable from an ecological viewpoint and for groundwater storage.

2. *Well field areas detail studies*

Several ^{14}C isotope anomalies were identified in the well field which need detail investigation in order to establish

- the likelihood of different recharge areas
- the possibility and extent of leakage from the bedrock into the confined aquifer
- the potential effect of leakage via abandoned boreholes to affect ^{14}C isotope results

3. Survey of all confined aquifer boreholes

A detailed survey of all boreholes penetrating the confined aquifer, particularly the old DWA boreholes in the area around the WCDM well field and to the north-east and the north-west, is critically important. This needs to be supplemented with camera logging. The condition of the boreholes and their casings needs to be established and remedial measures taken as needed to prevent overflowing, pressure loss, and leakage into the unconfined aquifer. Capping of potentially overflowing boreholes is essential. Plugging or rehabilitation of all leaky or short-circuiting boreholes is needed to prevent water losses. The question is, however, whether all the boreholes can still be found. Hydrochemical logging (including temperature) of suspect open boreholes will assist to provide answers regarding leakage.

4. Resume pilot injection tests

Continuation of injection will require a phased approach starting with extended injection tests in an area close to the recharge area (along the R45 near Hopefield as set out above) with extended hydraulic response monitoring. The hydraulic response to injection should contribute to a better understanding of the recharge area. It will not be influenced by WCDM well field abstraction and this may continue independently. Injection pressures of 150 to 200 kPa are recommended with an ultimate maximum of 250 kPa.

5. Saline intrusion

The potential ingress of saline water from the north-west towards the WCDM well field should continue to be monitored. Down-the-hole hydrochemical logging should take place to determine any potential saline water interface. In the event of saline water ingress the existing injection borehole in the WCDM well field should be utilised for injection to counteract any such phenomenon.

6. *Environmental aspects*

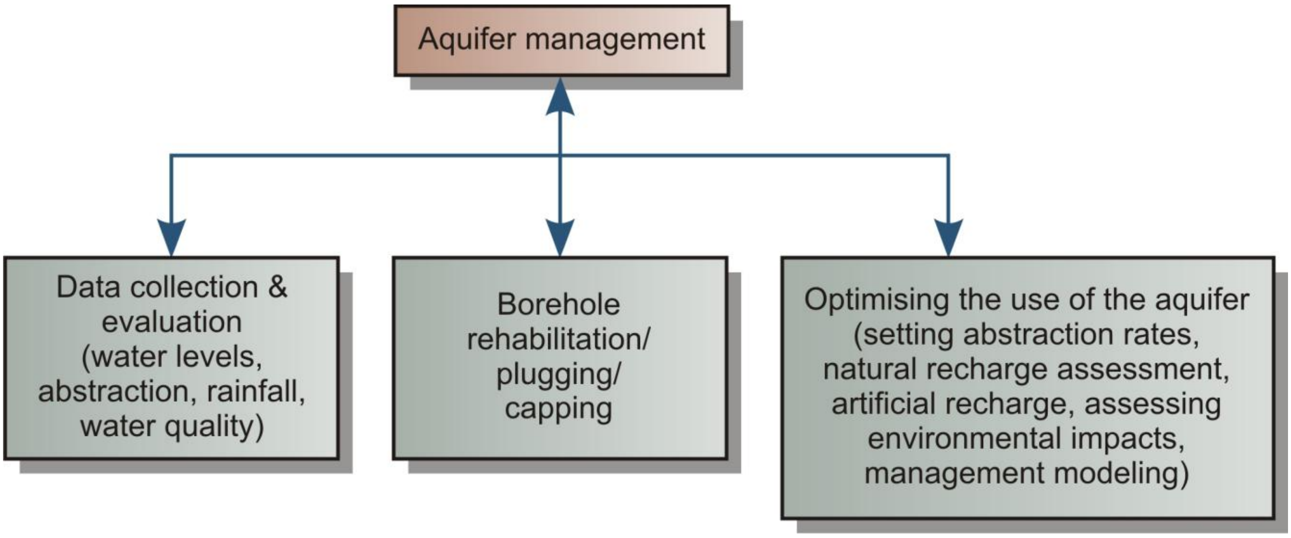
The hypothesis that an increase in piezometric levels and the associated movement of water from the aquifer into the borehole leads to a modification of bacterial populations needs to be tested. This should be done during the winter phase when abstraction ceases. The first sampling should take place just before abstraction stops, then twice monthly for up to four months or until production has resumed.

7. WCDM water level loggers

Calibration and maintenance of the WCDM loggers in the well field area should get a high priority. This management tool is important as real time access to the data is possible at the Withoogte water treatment plant.

8. Project management for optimising the use of the aquifer

Future project administration should be handled via the DWA Regional Office and a three-way contract drawn up to include the funding agency, the local authority end user, and the researchers. A small committee of these three bodies should guide further research on how to optimally utilise the aquifer. In relation to overall management, a possible structure is set out below:



Items that require research:

- Literature study on stygofauna
- Borehole data base – Timmerman reports
- Water level studies
- Hydrochemistry unknown boreholes
- Borehole camera logging
- Hydrochemical logging
- Check temperature anomalies
- Mathematical modelling
- Isotope studies ^{14}C

6 RAINWATER HARVESTING IN HERMANUS

6.1 Introduction

One of the water conservation measures that both individuals and municipalities can easily practice is rainwater harvesting. Too frequently we look for large-scale solutions when localized small-scale solutions may be equally, if not more effective. During the roll-out of the artificial recharge strategy, Hermanus was selected as a suitable location to monitor the groundwater level response of a shallow aquifer to rainfall. The aim was to get an indication of the artificial recharge opportunities at a household level in a residential area built on a sandy aquifer. Hermanus was selected because of the known high number of wellpoint users and ease of access by project staff.

The high cost of municipal water and water restrictions in dry times have seen well points becoming increasingly common in Hermanus, primarily used for irrigation and other non-potable uses. In addition, Hermanus implemented a water conservation and demand management programme in the late 1990's that included encouraging the use of well points and boreholes for garden irrigation and the registration of all groundwater users. (James van der Linde pers comm. 2006). The unconsolidated aquifer consists of 5-10 m deep sands overlaying the sandstones of the Table Mountain Group (TMG). The average groundwater level is about 2.5 m below ground level. **Figure 6.1** provides a schematic cross section of the sand aquifer overlying, and bound at the sea by, the sandstones. The water quality is suitable for garden irrigation with low salts and low iron and in the suburbs of Eastcliff and Northcliff, approximately three out of four houses have well points (see **Photo 1**). Well point yields are estimated between 800 and 2000 litres per hour (Andre Minnaar, *pers. comm.,* 2010).

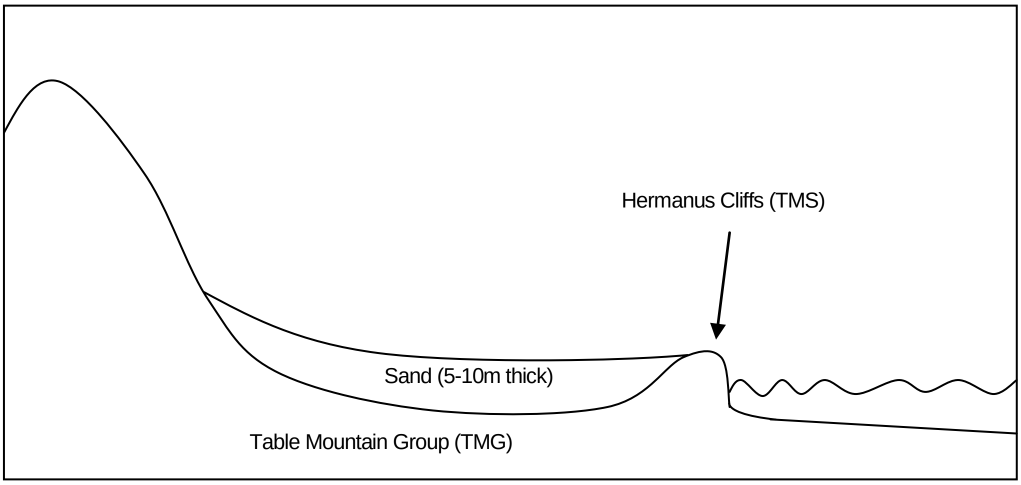


FIGURE 6.1 SCHEMATIC CROSS SECTION SHOWING THE UNCONSOLIDATED SAND AQUIFER

PHOTO 1 PROPERTY WITH A WELLPOINT DISPLAYING THE MUNICIPAL BORE HOLE / BOORGAT SIGNAGE

The project has been divided into 3 phases

- **Phase 1** - Test rainfall groundwater level response
- **Phase 2** - Depending on the results of phase 1, construct an infiltration soakaway for roof water at the test site. Test groundwater level response to this increased infiltration
- **Phase 3** – Depending on the results of phase 2, promote the widespread use of sustainable drainage by both municipal and private households to increase infiltration

The first phase of the project was to monitor the groundwater level through a rainy reason without any intervention in order to identify how the groundwater level responded to rainfall. Two wellpoints (**Photo 2**) with data loggers were installed in the Northcliff property of Mr. Martin de Klerk (**Figure 6.2**).

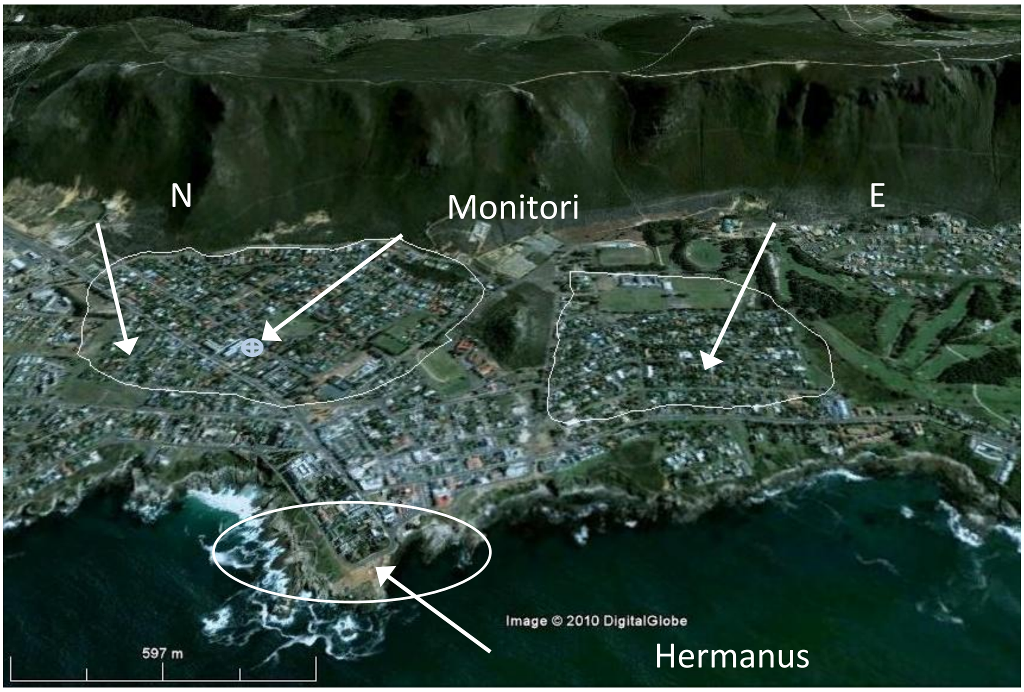


FIGURE 6.2 LOCATION OF MONITORING WELLPOINTS IN NORTHCLIFF (GOOGLE. EARTH, 2010)

PHOTO 2 ONE OF THE TWO WELLPOINTS INSTALLED IN HERMANUS

6.2 Results

The groundwater level response to rainfall is captured in **Figure 6.3**.

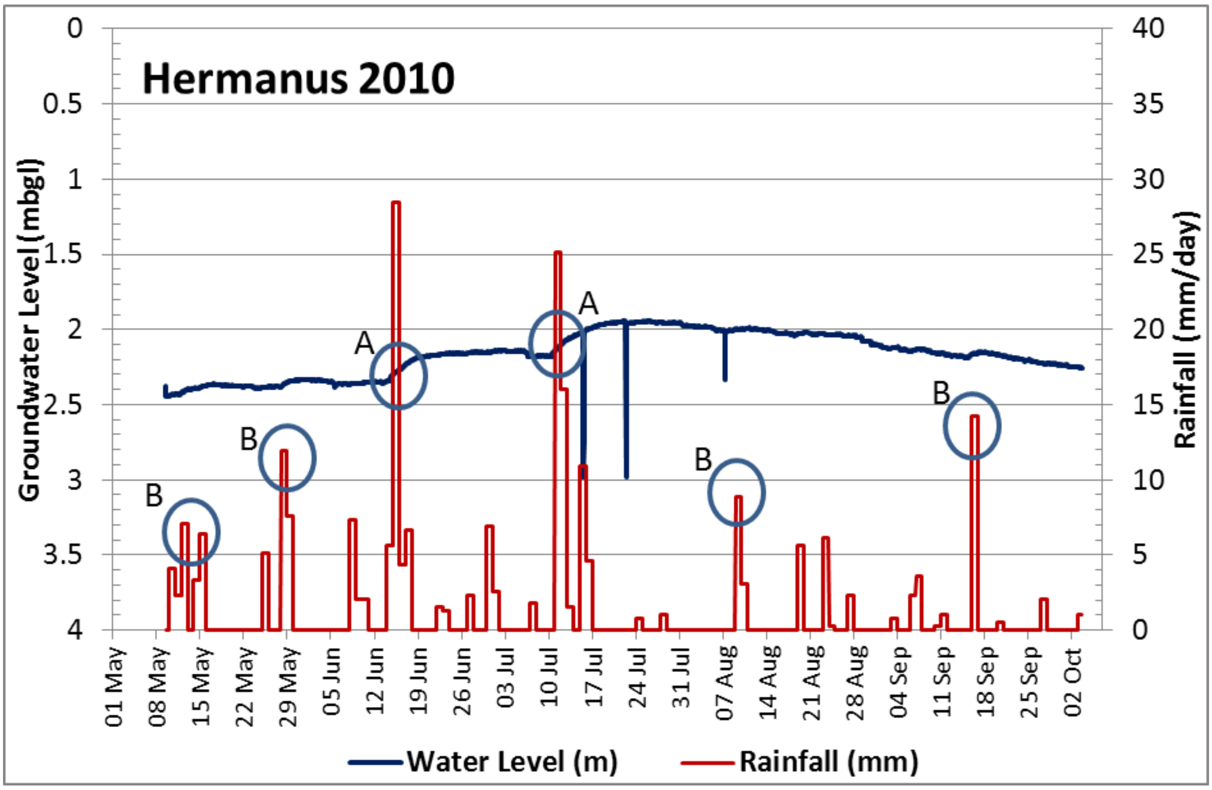


FIGURE 6.3 WATER LEVEL DATA FROM HERMANUS WELL POINT LOGGERS

Monitoring of these boreholes has shown that the groundwater level responds quickly to intense rainfall events above 10 mm (marked “B” in **Figure 6.3**), but the groundwater levels require a substantial rainfall event (in excess of ~20 mm/day) to show a significant rise (>30 cm), as seen in the rainfall events marked “A” in the graph above. The rapid aquifer response indicates that the aquifer is primarily recharged locally from rain infiltrating through the ground and not from seepage from the sandstones of the TMG. Rainfall less than 10 mm will generally be taken up by plants and will increase soil moisture without significantly recharging the aquifer. In between the rainfall events, the water level in the aquifer slowly drops due to abstractions, evapotranspiration from plants and trees with deep root structures, and the natural groundwater flow towards the sea through fractures in the TMG.

Taking the two areas of Northcliff and Eastcliff, it is conservatively estimated that during the dry months 25% of the water used by domestic consumers in this area is derived from wellpoints (**Table 6.1**). While this has limited significance in the total annual water used for the town, it is significant for reducing the summer peak, which is the critical water supply period in Hermanus.

TABLE 6.1 ESTIMATE OF CURRENT WATER USE FROM WELLPOINTS IN THE EASTCLIFF AND NORTHCLIFF AREAS

Suburb	Number of Erven	Estimated Erven with wellpoints (75%)	Monthly wellpoint water use (kl) using average of 10 kl/month/house	Estimated daily wellpoint water use (kl)
Northcliff	370	280	2 800	93
Eastcliff	280	210	2 100	70
Total	**650**	**490**	**4 900**	**163**

Assuming six months use at these volumes, this equals about 30 Ml per annum that is currently supplied from wellpoints.

6.3 Urban Drainage

Increased urbanisation results in an increased area of hard surfaces. In Hermanus, these surfaces drain into stormwater pipes and discharge directly into the sea. This water does not contribute to aquifer recharge and will therefore reduce the long term availability of groundwater. **Photos 3, 4 and 5** show examples of paved areas and road drainage being fed into the stormwater.

It is considered that the continued and increased use of well points and boreholes in the shallow sand aquifer underlying Hermanus is an effective way of meeting the non-potable water demand, however to ensure the longevity of this resource it is important that appropriate water resource management is practiced.

PHOTO 3 EXAMPLE OF IMPERMEABLE PAVING IN HERMANUS

PHOTOS 4 AND 5 STORMWATER INLETS ON THE ROADS IN EASTCLIFF

6.4 Sustainable Drainage

Sustainable Urban Drainage Systems (SUDS) seek to preserve natural hydrological conditions in spite of urbanisation. This includes:

- Dealing with rainwater at source
- Maintaining infiltration rates and minimising the volume of surface run-off.
- Preventing rapid run-off which places significant pressure on storm water drainage systems and can result in flooding.

The highly permeable nature of the Hermanus' surface geology means that natural infiltration will be high, and therefore the natural surface run-off will be low. The construction of roads, driveways and buildings will increase the volume and rate of surface run-off. The appropriate use of SUDS can effectively mitigate the effect of urbanisation and secure the sustainable use of groundwater.

NOTE: Where the risk of contamination of the surface water is high (petrol stations, large car parks, or industrial areas), it is important that appropriate measures are incorporated into the drainage design to prevent pollution of the aquifer. This might typically be the incorporation of petrol interceptors or diverting high risk areas into the foul sewer.

6.5 SUDS Options

The following SUDS options tabulated below (**Table 6.2**) are particularly well suited to Hermanus and can be retrofitted to existing developments and should be prioritised for ALL new development.

TABLE 6.2 SUDS OPTIONS

Option	Description	Particular Requirements	Relative Cost	Hermanus Application
Permeable Paving 	Roads, driveways and paths constructed from permeable materials to allow rainwater to infiltrate into the underlying soil. Generally constructed from blocks with gaps in between laid on a sand and gravel base with geotextile as a separation layer..	Special shaped blocks and permeable tarmac are available, although loosely packed blocks without mortar, or gravel can also be effective where vehicular loads are low.	Moderate / High	Private and municipal paved areas, parking lots, low use roads
Swales 	Shallow grass lined ditch generally at a shallow or no gradient. Sediment is trapped by the grass and water is allowed to infiltrate into the soil. Swale can provide conveyance and storage for conventional storm water drainage.	Must be designed with appropriate conveyance and storage capacity to prevent surface flooding	Low	Road side drains
Concealed Soakaways 	A sub-surface trench or pit typically filled with granular material to create a void and lined with geotextile as a separation layer. Water may be discharged into the basin from roofs driveways or other impermeable surfaces.	Generally should be sited 5 m from buildings, although this can be reduced in highly permeable soils. Base of soakaway should be 1 m above max. groundwater level to maintain unsaturated zone for filtration.	Low / Moderate	Roof rainwater runoff

Option	Description	Particular Requirements	Relative Cost	Hermanus Application
Infiltration Strips 	Trenches that are filled with granular and typically perforated pipe and lined with geotextile as a separation layer. Surface water from the edge of a road or path is filtered and allowed to infiltrate into the soil. Intense rainfall will be conveyed along the pipe to other drainage systems.	Generally requires space to the side of the road, and can be used as an alternative to conventional piped drainage.	Moderate / High	Road side drains and edges of paved areas
Infiltration Basins 	A flat basin designed to capture surface water and enable it to infiltrate into the underlying soil. Water may be discharged into the basin from roofs driveways or other impermeable surfaces.	Requires space for the temporary inundation of surface water. Can be designed to flood only at shallow depths and for short periods. Infiltration basins can therefore be incorporated into the landscaping of household gardens.	Low	Household or municipal stormwater collection point
Wetlands / reed beds 	A constructed wetland habitat which filters and attenuates surface water flows. This will typically include reed beds, and is particularly beneficial as a pre-treatment where there are minor water quality issues.	To establish a healthy wetland system continually damp soil conditions are required, this is unlikely to be possible where natural infiltration rates are high.	Moderate	Municipal stormwater drainage collection point

Additional information associated with the design and Construction of these measures can be found in the CIRIA publications C697 and C698.

6.6 Implementation potential

In order to realise the benefit of SUDS and maximise groundwater recharge, the following actions are recommended in Hermanus:

1. Ongoing monitoring of groundwater levels, rainfall and abstraction at the test site to demonstrate the effectiveness of the roof water soakaway.

2. Introduce a new By-law requiring all new developments to be constructed with infiltration drainage in areas with suitable ground conditions.

3. Promote the benefits of retrofitting SUDS to existing developments, particularly to those who have well points.

4. Identify opportunities to construct new SUDS systems to receive the existing storm water drainage and infiltration.

5. Promote the use of wellpoints for the irrigation of gardens, public open space and sports fields.

It is estimated that over 30% of the Northcliff and Eastcliff suburbs is made up of constructed impermeable surfaces which discharge into stormwater drainage. If one was to implement SUDS in these two suburbs and was able to infiltrate just 10% of the runoff from the total area, the increased recharge would equal four Megalitres per annum or 14% of the current wellpoint water use (**Table 6.3)**.

TABLE 6.3 POTENTIAL INCREASED RAINWATER INFILTRATION AND ARTIFICIAL RECHARGE.

Suburb	Suburb Area (Ha)	Target Infiltration area - 10% of total (Ha)	Annual rainfall (mm)	Harvestable rainfall - estimated 70% (mm)	Annual potential increased recharge (kl)
Northcliff	68	6.8	546	382.2	2 599
Eastcliff	40	4	546	382.2	1 529
Total	**108**	**10.8**			**4 128**

Sustainable drainage is implemented extensively on an international scale, but is not common in South Africa. SUDS should be implemented on all new developments; especially where there is groundwater use and groundwater recharge potential.

7 BOREHOLE INJECTION IN THE VERMAAKS RIVER VALLEY, OUDTSHOORN

7.1 Introduction

The artificial recharge aim would be to opportunistically divert a portion of flood water that rapidly runs off from the upper Vermaaks River catchment into boreholes in order to raise the depressed water levels. Artificial recharge, however, will not take place in the foreseeable future due to institutional issues regarding environmental concerns. Part of the aquifer is in CapeNature's land and they have stated that they do not want artificial recharge to take place in this area. The project has thus been halted. Prior to CapeNature taking this decision, meetings were held with the Oudtshoorn Municipality and CapeNature where the proposed scheme was described. Site visits were held and initially the AR concept seemed to be well-received by all parties. Tests were conducted on two boreholes to establish whether they would be suitable to receive artificially recharged water. The results were positive and are briefly described below.

The location of the Vermaaks wellfield and the proposed trial artificial recharge facility is shown in **Figures 7.1** and **7.2.**

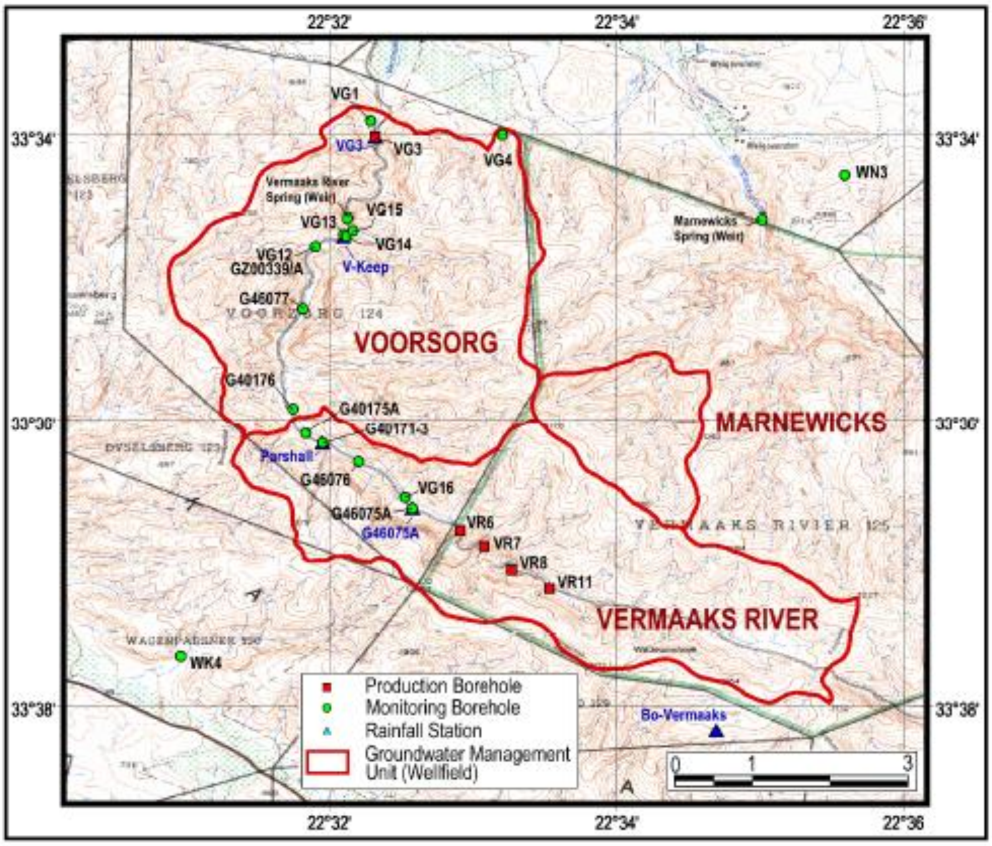


FIGURE 7.1 THE VERMAAKS RIVER GROUNDWATER MANAGEMENT UNIT

FIGURE 7.2 THE PROPOSED INFRASTRUCTURE TO TEST AR IN THE UPPER-MOST PART OF THE VERMAAKS RIVER WELLFIELD (GOOGLE EARTH, 2010)

The intention for artificially recharging Vermaaks River boreholes is to raise the groundwater levels in the upper part of the catchment (where water levels have dropped over the years due to abstraction). The proposed method involves the diversion of runoff (when available) into boreholes to directly recharge the aquifer.

Figure 7.3 shows the water level decline in the upper parts of the wellfield (at borehole VR8).

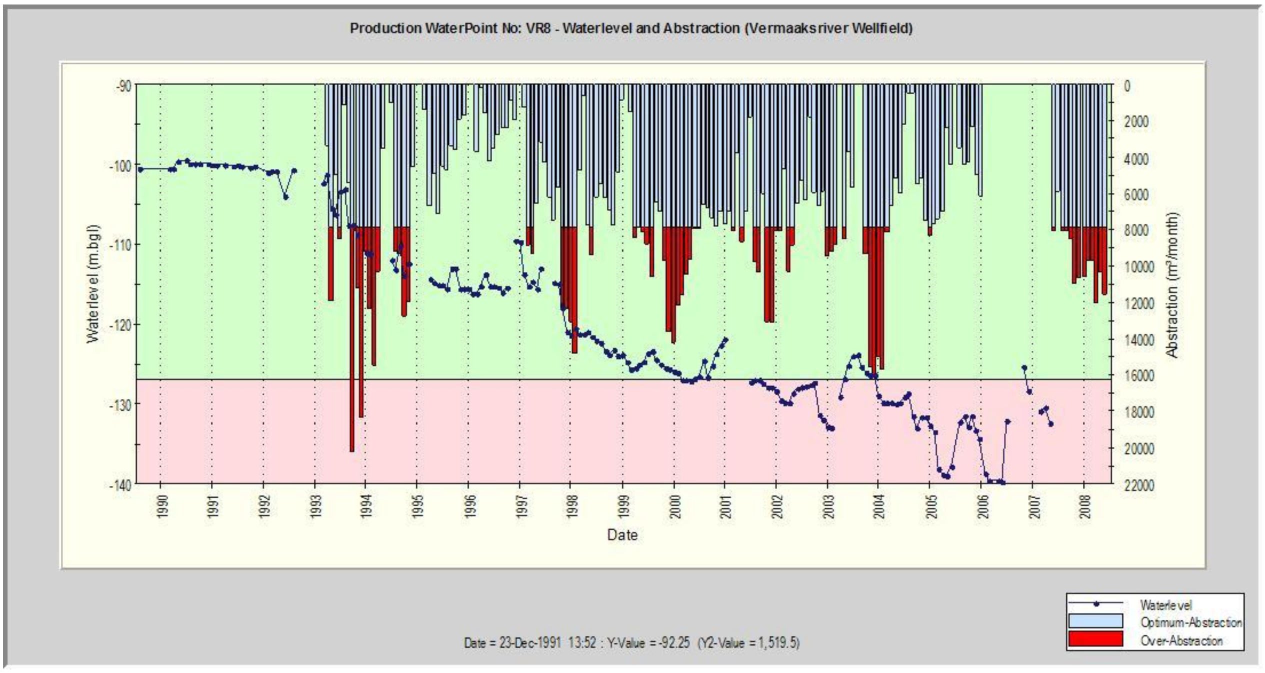


FIGURE 7.3 WATER LEVELS IN BOREHOLE VR8

7.2 Artificial recharge potential

Prior to assessing the feasibility of AR at this site, two key questions needed to be answered in order to proceed:

1. Is it possible to replenish the aquifer by borehole injection?
2. Will CapeNature support this intervention? The key issue being that the project has to be environmentally acceptable.

There are two further questions that would need to be addressed if the project was to have continued:

3. Will the volumes injected make a difference? I.e. is there enough storm runoff that can be injected to significantly raise the groundwater levels?

4. Is the water quality acceptable for borehole injection? The key issue being whether the water is too turbid (after some form of basic filtration) to be allowed to enter the aquifer.

Questions 3 & 4 would have to have been addressed if the go-ahead was given for an initial injection trial period. This would have involved diverting stream flow from above VR to one of boreholes adjacent to VR 11 (shown in **Figure 7.2**). It would be necessary to run the trial test until a few storm events were monitored (approximately one year).

Regarding Question 1, a very brief initial injection tests on the nearby boreholes at VR 11 was conducted on the 14th December 2009 and the results were positive. The aim was to establish if these boreholes are dry/blocked, and if not, whether they could be used as injection boreholes to artificially recharge the aquifer. Water was pumped from VR 11 into the two nearby boreholes in order to establish whether they would accept water or not. The western hole (closer to the road – VR11M-W) took the injected 7 L/s, while VR11M-E (the other hole) did not. From these brief tests, it appears as if VR11M-W can take at least 10 L/s and possibly 20 L/s.

Figure 7.4 shows the water level response to injection. The starting water level in VR 11 was in the order of 174 mbgl. This graph shows that the water level rose very rapidly in the hole, and that after about 30 minutes it began to stabilise when it reached a level of about 48 m below ground level. The water level rise thereafter until the test was completed after 280 minutes was very gradual. After injection had ceased, the water level rapidly dropped off, thereby confirming a borehole that is hydraulically well connected to the aquifer. The same cannot be said about the other borehole (VR Mon East), where the water level had a delayed response to injection, and then did not drop after injection ceases.

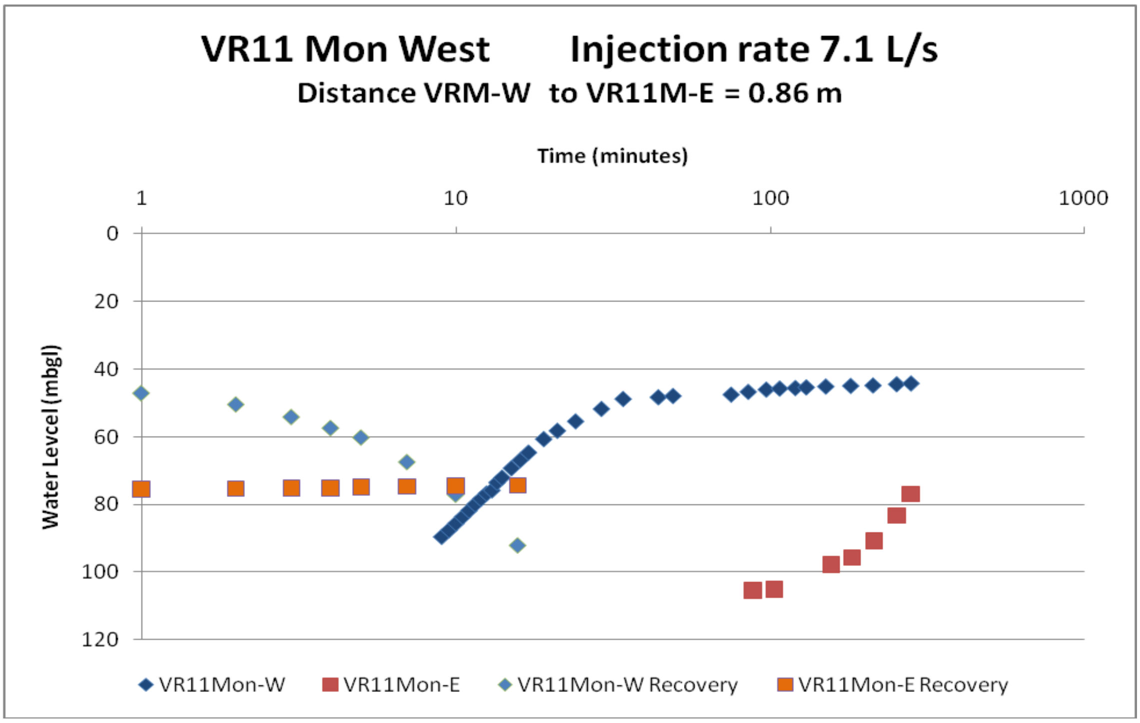


FIGURE 7.4 INJECTION INTO BOREHOLE VR11 MON WEST

In order to confirm the poor capabilities of Bh VR11 Mon East, the same injection test that was done on VR11 Mon West was conducted on this borehole. This was run for 85 minutes and the water level rose to near surface (6.5 mbgl) – see **Figure 7.5**. This, together with the slow drop in water levels after the test, confirmed that the borehole is not suitable for injection, and can be considered virtually a dry or blocked hole.

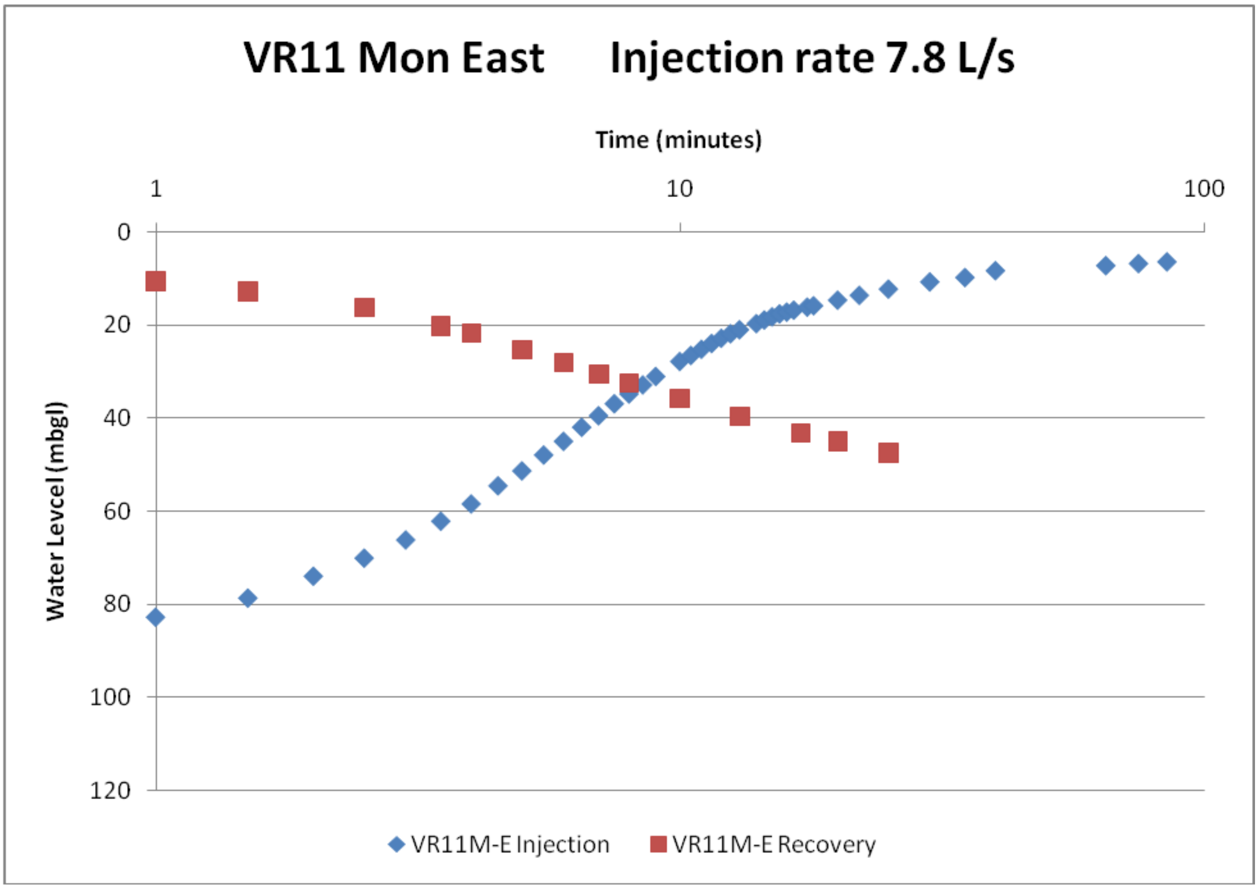


FIGURE 7.5 Injection into borehole VR11 Mon East

From these tests, it is evident that VR11 Mon West can receive 7 L/s. It is possible that it can receive up to 20 L/s, but this could not be tested. For design purposes, it would be best to assume that it can receive 20 L/s as the intention would be to recharge as much water as possible during the short time when flood water is available.

7.3 Conclusion

As stated in the introduction, artificial recharge in the Vermaaks River valley will not take place due to institutional issues which in-turn are based on environmental concerns. The main reasons for the decision to discontinue further artificial recharge investigations relate to CapeNature's points (pers comm. Dr Kas Hamman, CapeNature, 20 January, 2010):

- “Any permanent structures (cement) that would be required in the river, should the initial feasibility phase be successful would not be supported by CapeNature”
- “CapeNature will continue to pursue the matter to have the Vermaaks wellfield permanently shut down…”.

If the test at VR 11 proved very encouraging, it would be necessary to up-scale artificial recharge to have the potential to make a significant difference to groundwater levels in the aquifer. This would entail constructing a few permanent artificial recharge take-off points in the river, which CapeNature would object to. In relation to the second point, it is not worthwhile undertaking the research, and designing and constructing the scheme, if it could be closed down. For these reasons, the project was stopped.

8 ARTIFICIAL RECHARGE OPTIONS IN LEPHALALE

8.1 Introduction

This case study has been taken from the DWA report that was published as part of the roll-out of the artificial recharge strategy (DWA, 2009). It is written as a "to-do-list" that is required to assess the potential of this area, and serves as an example of how artificial recharge can be considered at the inception stages of wellfield development. The full report on the artificial recharge assessment of this area is captured in the DWA report entitled *Hydrogeological Assessment and Aquifer Recharge Potential within the Lephalale (Ellisras) Local Municipality Area, Phase 3: Artificial Recharge and Geochemistry* (DWA, 2010).

The water requirements in the Lephalale area will increase significantly with the planned new coal-fired power stations, and artificial recharge has been considered as one of the options in augmenting the water supply to the area (**Figure 8.1**).

8.2 Artificial recharge options

The groundwater and artificial recharge water supply/ augmentation options are listed below:

1. Maximise deep/ hard-rock aquifer
2. Maximise shallow/ alluvial aquifer
3. Artificially recharge the shallow aquifer with stormwater
4. Artificially recharge the shallow aquifer with treated waste water
5. Artificially recharge the shallow aquifer with untreated dam water
6. Artificially recharge the deep aquifer with treated dam water
7. Artificially recharge the deep aquifer with stormwater
8. Artificially recharge the deep aquifer with water from the shallow aquifer by creating easy flow paths between the two using "linking" boreholes

8.2.1 Maximise deep/ hard-rock aquifer

Aim: Maximise groundwater abstraction from the hard-rock aquifer

Tasks:

1. Identify, drill and test pump the remaining geological structures within an acceptable economic radius of:

    a. The town

    b. The pipeline from the dam that supplies the town (if this option is considered, first establish if it is possible to feed into the existing pipeline – i.e. establish engineering requirements)

2. Delineate the groundwater units/ wellfields and quantify them under conditions of simultaneous usage (i.e. take interference into account). This probably requires pumping from the wellfields for a considerable period and monitoring adjacent areas. The aim would be to establish their sustainable yields based on simultaneous pumping.

3. Establish their “mining” yield. If the deep aquifers are needed to bridge a period of supply until surface water becomes available, then the aim would be to establish how long the deep aquifer can supply the required rate on a continuous basis before the storage has been drained.

4. Linked to point 2 above: Establish whether water is induced from the alluvium if nearby deep, hard-rock boreholes are pumped heavily. Do this by large-scale abstraction from the deep boreholes and assess the drawdown curves and monitor isotopes and water quality. A nearby shallow borehole would also be best for monitoring purposes.

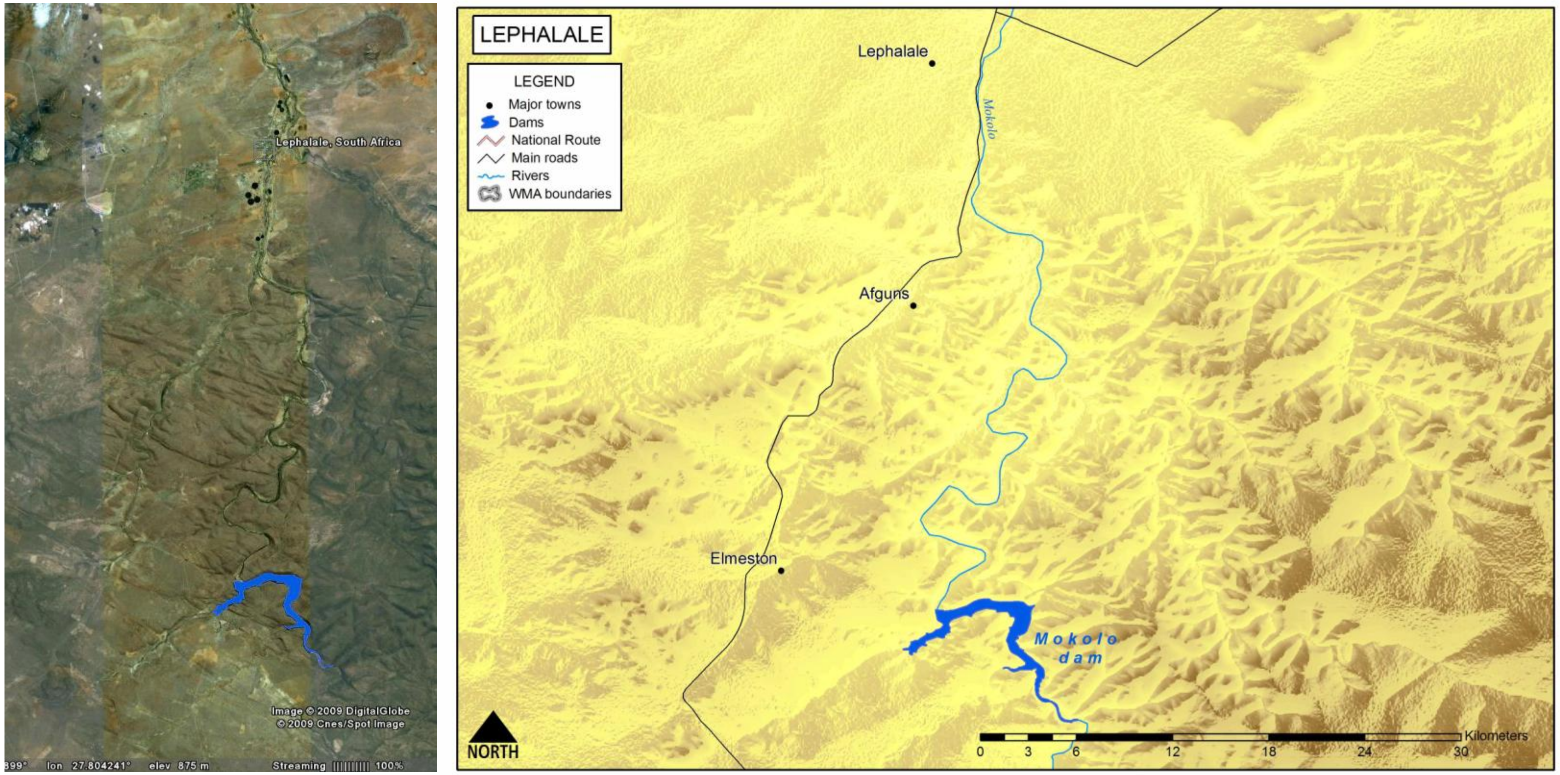


FIGURE 8.1 GOOGLE EARTH AND HILL-SHADE IMAGES OF THE LEPHALALE AREA SHOWING THE MOKOLO DAM UPSTREAM OF THE TOWN (GOOGLE EARTH, 2010)

8.2.2 Maximise shallow/ alluvial aquifer

Aim: Maximise groundwater abstraction from the alluvium

Tasks:

1. Establish the volume in storage by delineating the alluvial aquifer, establishing its thickness, water levels and S-values:

    a. consult the VSA report (undated)

    b. drill a few shallow boreholes to test the thickness of the alluvium, to verify VSA results and to get S-values (i.e. drill nearby monitoring boreholes)

    c. possibly do geophysics to determine thickness in selected areas

2. Establish borehole yields in the alluvium – i.e. test pump a few shallow holes (& monitor both pumped and nearby observation boreholes)

3. Establish throughflow (either using Darcy or a simple numeric model)

4. Determine whether a collector well system (horizontal "well/s" across the river bed) or numerous shallow boreholes parallel to the river are most cost and water supply efficient.

8.2.3 Artificially recharge the shallow aquifer with stormwater

Aim: Recharge the alluvium with stormwater that is collected in the stormwater dam

Key assumption: The alluvium is sufficiently transmissive to yield significant quantities of water (i.e. establish the borehole/collector well yield from the alluvium first)

Tasks:

1. Quantify the stormwater. Obtain an estimate of the volume of water available for recharge and only consider this option if the volume is significant. Establish whether storm runoff is already allocated or used by others.

2. Assess the quality of the stormwater if the abstraction boreholes are close to the stormwater discharge point. If so, this water will have to settle in the dam before being used for recharge. Key factors are turbidity, oils, heavy metals, etc. – the determinants that are found in urban storm runoff. Note that the first storm runoff for the season should go to waste (discharged elsewhere if possible) – to flush all the remnants from the dry season.

3. Enhance canal infiltration by ripping the base and sidewalls of the canal where it crosses the alluvium.

4. Enhance infiltration in the second stormwater dam (the dam shortly before the discharge point) by ripping the base and sidewalls of the dam – i.e. convert to recharge basin

5. Drill a few shallow monitoring boreholes to establish effectiveness of these interventions

6. Check the discharge point – e.g. ensure the stormwater is not being taken by farmers.

7. Determine whether a collector well system or numerous shallow abstraction boreholes parallel to the river should be drilled below the stormwater discharge point.

8. Drill a few shallow monitoring boreholes.

8.2.4 Artificially recharge the shallow aquifer with treated waste water

Aim: Recharge the alluvium with treated waste water

Key assumption: The alluvium is sufficiently transmissive to yield significant quantities of water (i.e. establish the borehole/collector well yield from the alluvium first).

Tasks:

1. Quantify the waste water. Obtain an estimate of the volume of water available for recharge. Establish what portion is already allocated.

2. Assess the quality of the waste water.

3. Establish where waste water is discharged into the river

4. Establish all sites where (deep) alluvial boreholes could be drilled adjacent to the river (consider land ownership, access, etc).

5. Alternatively, establish a site for a collector well.

8.2.5 Artificially recharge the shallow aquifer with untreated dam water

Aim: Recharge the alluvium with surplus untreated dam water:

a. At a point close to town (i.e. where a take-off point can be installed in the existing pipeline that is not far from town)

b. From normal dam releases

Key assumption: The dam managers are willing to release surplus water in a manner that will enhance recharge (this will require selling the concept to them).

Tasks:

1. Establish current dam release practices (when and for how long)

2. Quantify releases – i.e. water available for recharge, and establish existing allocated water (i.e. compulsory releases)

3. Establish pipeline capacity to see whether surplus water can be transferred close to town for recharge; quantify this water source

4. Identify preferred discharge point

5. Cost pipeline diversion

6. If this is not an option, establish if dam managers would be ameanable to large-scale releases in pulses rather than a continuous "trickle". The aim would be to try and get a pulse of dam water to recharge the alluvium close to town (i.e. close to where you'd place your wellfield).

7. Study the DWA "pulse" report

8. Compare the water savings if water is left in the dam (evaporation losses) and trickle releases (evapotranspiration losses and abstraction from upstream farmers) to if it is transferred to the alluvium (evapotranspiration losses and abstraction from all farmers)

8.2.6 Artificially recharge the deep aquifer with treated dam water

Aim: Borehole injection (ASR) with municipal drinking water

Tasks:

1. Establish if there is ever surplus water available and surplus capacity in the water treatment works (WTW)

2. Assess water compatibility (municipal water and deep groundwater)

3. Conduct injection tests

8.2.7 Artificially recharge the deep aquifer with stormwater

Aim: ASR using storm water

Tasks:

1. Establish quantity and quality of stormwater (see Option 3 points 1 & 2)

2. Determine stormwater treatment requirements

3. Assess water compatibility (stormwater and deep groundwater)

4. Conduct injection tests

8.2.8 Artificially recharge deep aquifer with shallow aquifer by creating easy flow paths between the two with "linking" boreholes

Aim: Create conduits for alluvial aquifer to recharge hard-rock aquifer.

Note:

1. This is only an option if water in the alluvium is not drawn into the deep aquifer by large-scale abstraction from deep boreholes (see Option 1, point 4). If so, it is not necessary.
2. Under current (i.e. natural conditions), deep water will probably flow into the alluvium. For this to work, the water level in the deep aquifer must be continuously pumped.
3. Consider long-term environmental implications of this option as this will cause a permanent connection between the shallow and deep aquifer. I do not recommend this option for this reason.

Tasks:

1. Drill a deep borehole on a hard-rock structure where the alluvium is thick and perforate the casing opposite both the alluvium and the deep aquifer.

2. Assess whether the shallow aquifer will feed the deep aquifer:

   a. Analyse chemistry and isotopes of shallow water (prior to puncturing the deep aquifer)

   b. Analyse chemistry and isotopes of deep aquifer from another, nearby deep borehole that only intercepts the deep aquifer.

   c. Test pump the nearby deep borehole and monitor chemistry and isotopes

d. Assess drawdown curve for leakage (in the test pumping of the nearby deep borehole)

3. Test pump the "linking" borehole and establish yield.

4. Repeat in other areas where structures cross the alluvium.

8.3 RECOMMENDATIONS

The recommended process to determine which options to pursue is as follows:

1. Establish which of the above options are realistic.

2. If any of the artificial recharge options are considered to be realistic, initiate an artificial recharge pre-feasibility study as described in the Artificial Recharge Strategy (DWAF, 2007).

9 INFILTRATION BASINS OR TRENCHES AT KENHARDT

9.1 Introduction

Kenhardt relies on groundwater for their water requirements. The main wellfield in the Driekop Aquifer west of town can neither meet the peak summer demand nor the future requirements of the town. Artificial recharge, which was recommended by DWAF over 30 years ago (Nonner, 1979) remains the cheapest option to potentially solve Kendardt's water resource problems. Besides the more expensive groundwater development options from aquifers further afield (some of which have been explored and developed) and recycling domestic waste water, the only remaining alternative is the hugely expensive option to pipe water over ~80 km from the Orange River. This case study has been adapted from the first Water Research Commission investigation into artificial recharge (Murray & Tredoux, 1998).

The artificial recharge plan essentially involves transferring water from the Rooibergdam when available to the Driekop Aquifer up-gradient and within the existing wellfield (**Figure 9.1**). The water would gravitate from the dam down an existing canal to the holding tank on the edge of town from where it would be pumped to the infiltration basins in- and up-slope of the wellfield.

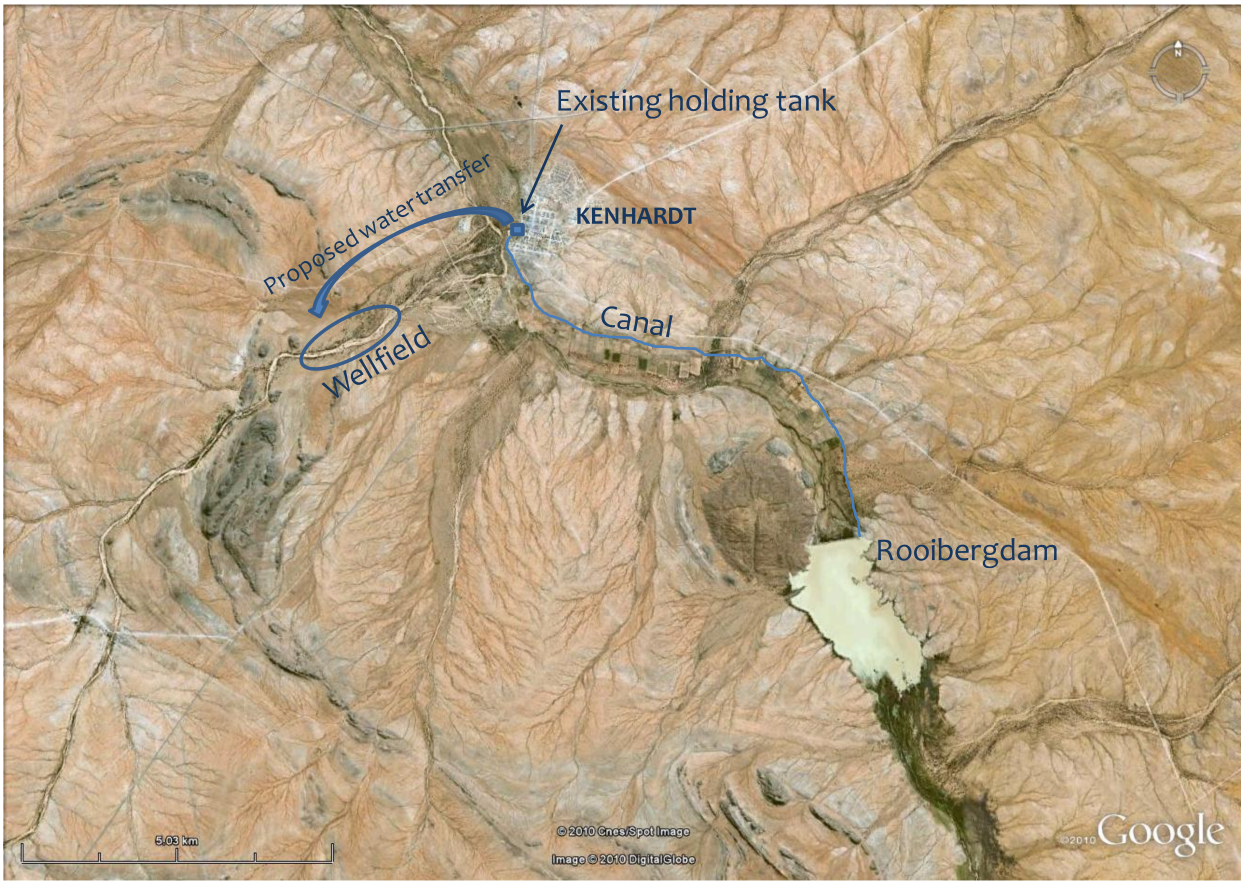


FIGURE 9.1 THE PROPOSED WATER TRANSFER SYSTEM FOR RECHARGING THE DRIEKOP AQUIFER (GOOGLE EARTH, 2010)

9.2 The source water

The Rooibergdam, also referred to as the Rooidam and the Kenhardt Dam, was constructed in 1900. It is a shallow dam that covers an area of approximately 3 km by 1 km when full, but is usually dry for several months of the year. Its capacity dropped from 7.1 Mm^3 after completion in the 1930s to 3.7 Mm^3 in the 1980s. An indication of its reliability can be taken from a 17-year monitoring period from 1980 to 1997 when it was observed that during nine of these 17 years, the dam contained water for at least six months per year; and only in three of these years did it not have any water at all. Although the dam is "unreliable" it appears as if the dam is well-suited for opportunistic artificial recharge.

The three main water quality concerns relate to the amount of suspended solids, the salinity and the bacteriological quality of the water. It is likely that micro-organisms will die off prior to reaching the abstraction boreholes, since the proposed method requires infiltration through 5 - 10 m of alluvium and the abstraction points should be located at least a hundred metres from the infiltration points.

Suspended solids would need to settle out prior to artificial recharge, as they would clog the recharge facility. The turbidity of the water is likely to fluctuate substantially in relation to its standing time in the dam. It is estimated that the turbidity will have to be reduced to below 2 NTU before it can be pumped to the infiltration basins.

The electrical conductivity (EC) fluctuates significantly, however, over an eight-year monitoring period it was observed to generally fall between 25 - 100 mS/m, with an average of 68 mS/m. Like turbidity, EC of the dam water would have to be monitored in order to prevent higher salinity water from entering the recharge facility.

9.3 The Driekop Aquifer

The Driekop aquifer has been described in detail by Nonner (1979) and Van Dyk (1994). The aquifer consists of alluvium with a thickness of up to 11 m; weathered gneiss of the Namaqua Metamorphic Complex, which reaches a thickness of 30 m; and fractured gneiss which have water bearing fractures extending down to a depth of about 50 m. Based on drilling yields, Nonner (1979) describes the alluvium as either being dry or low yielding (5 - 50 m^3/day); the weathered gneiss as being the main water yielding formation - especially where water strikes are located near the top and bottom contacts of the weathered rock; and the unweathered gneiss as giving high yields if the boreholes penetrate fractures (up to ~13 L/s). Although the infiltration capacity of the alluvium would need to be determined, artificially recharging the aquifer should not be a problem – the infiltration contact area would need to be designed according to the permeability of the sands.

The water level in the aquifer prior to development in the late 1970's / early 1980's lay between 3 - 6 metres below ground level (mbgl), which was either within the alluvium or at the contact between the alluvium and the weathered rock. By the late 1990s, the water levels had dropped by ~10 m as a result of groundwater use.

Nonner (1979) estimated the volume of water held in storage as just below 1 000 000 m^3, or twice the projected 2013 demand.

The water in the Driekop Aquifer is fairly saline with EC values ranging from 140 – 190 mS/m, and has elevated fluoride concentrations of 2 – 4 mg/L. By selectively introducing low salinity water from the Rooibergdam, the groundwater quality in the Driekop Aquifer should improve.

9.4 Design of the artificial recharge facility

In terms of recharge efficiency and management requirements it seems as if infiltration basins may be the appropriate artificial recharge method. Other options include borehole injection and a trench system which was proposed by Nonner (1979). Nonner's design consists of an 18 m long, 1 m deep trench filled with gravel. The recharge water would be fed into gently sloping, slotted casing which is located in the middle of the trench. The recharge trenches would be placed about 100 m up slope of the main production boreholes. Besides clogging considerations, the design of the recharge facility will need to be based on an assessment of the hydraulic relationships between the various soil and rock layers. Considerations for assessing these relationships are proposed in the WRC report (Murray & Tredoux, 1998).

10 BOREHOLE INJECTION IN KATHU

Both Khumba Mine (Sishen) and Kathu town (**Figure 10.1**) could benefit from practicing artificial recharge: Khumba Mine could get an easy way of disposing surplus groundwater that is pumped during their mine dewatering process, and Kathu could benefit through the recharging of their aquifer. The only foreseeable drawback involves the possible expansion of Khumba Mine operations, which may result in them requiring groundwater for their own purposes.

Water levels in the wellfield area of the Kathu Aquifer have dropped by over 20 m in 27 years, and there is a general decrease in water levels of about 0.7 m/annum (Murray, 2006). This trend could be stopped or reversed by borehole injection via existing or new high yielding (20-30 L/s) boreholes in and up-gradient of the hydraulic depression. The only available water source is groundwater from Khumba Mine. Although much of this water is used in the mining operation, any surplus could be transferred to the Kathu Aquifer. An artificial recharge pre-feasiblity study has been undertaken for Sishen Iron Ore Company (Pty) Ltd (Murray, 2006).

FIGURE 10.1 KHUMBA MINE WITH THE GREENERY OF KATHU TO THE NORTH-WEST OF THE MINE (GOOGLE EARTH, 2010)

PHOTO 10.1 ONE OF KATHU'S MUNICIPAL BOREHOLES

11 SAND DAM OPPORTUNITIES IN LIMPOPO AND MPUMALANGA

The six examples below show how sand dams could be used to increase the assurance of small-scale irrigation supplies for farmers.

11.1 Mbonisweni Community Garden

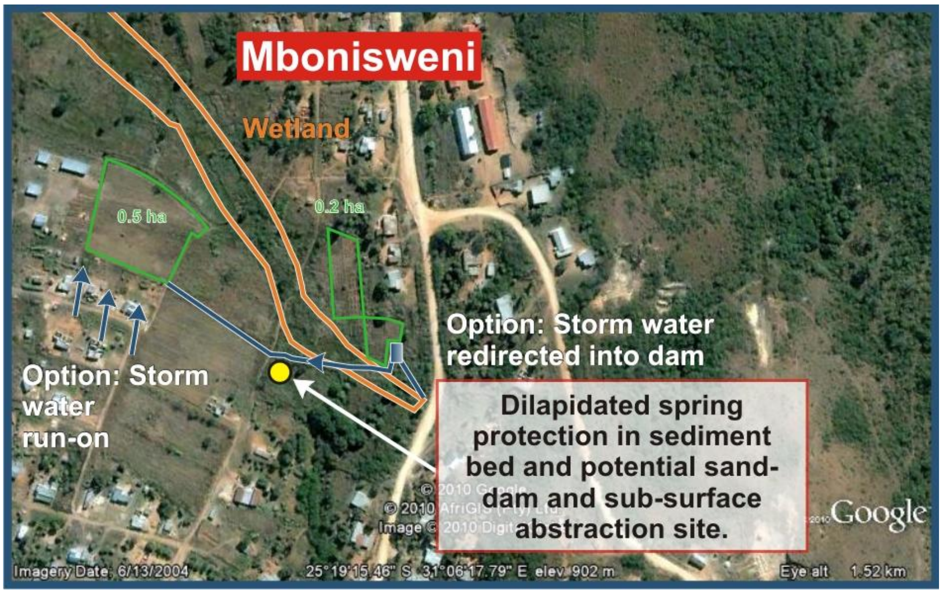
Location: Mpumalanga
Nearest Town: White River
Coordinates: S25^{0}19' 16.2" and E31^{0} 06 10.5
Potential sand dam use: food garden water supply (0.7 ha)

Mbonisweni village, located just 10 km east of White River, is a peri-urban settlement in the former homeland of KaNgwane. Water is a significant constraint for the group of 10 women who share a community garden, particularly during the dry winter months.

The group has access to land and to limited water from a permanent spring located in an adjacent wetland. Ongoing serious siltation from stormwater runoff from the adjacent gravel road progressively filled a small dam that was used to collect spring-water, and as a result the group re-located to a field closer to their homes. The road has since been surfaced, but the small dam and spring remain silted. The existing silted dam could be further developed as a sand dam, with augmented inflow and an improved sub-surface abstraction system.

The existing dilapidated spring protection works (effectively abstracting from a rudimentary well sited in the sediment beds) could be augmented or replaced with a more substantial barrier and engineered sediment abstraction system. While major construction work in the wetland is likely to be environmentally unacceptable, it is possible that a low-impact sand-dam could provide a sustainable and environmentally acceptable solution. Stormflow runoff could be diverted from a nearby tarred road, to augment the natural overland flow to provide a composite solution i.e. a sand dam with increased inflow (water harvesting) diverted from the nearby tarred road (**Figure 11.1**).

FIGURE 11.1 MBONISWENI GARDENING GROUP FIELDS AND POTENTIAL SAND-DAM SITE (GOOGLE EARTH, 2010)



11.2 Mjejane Community Garden

Location: Mpumalanga
Nearest Town: White River
Coordinates: S 25^0 16' 20.25" E 31^0 14' 08.15"
Potential sand dam use: food garden water supply (2 ha) and cattle watering

Mjejane is located about 30 km east of White River town and is a small and relatively isolated, extremely poor rural settlement bordering the Kruger National Park. A community food-gardening initiative comprising 24 members (with 132 dependents) is seriously limited by inadequate water availability all year round. The gardens are located on the right bank of an ephemeral stream. Soils are very sandy as a result of the granitic geology and any impoundment would result in rapid siltation presenting an ideal opportunity for a sand dam.

A conceptual planning study identified a rainwater harvesting option as one solution, and while a sand dam was considered, the construction costs were in excess of available funding at the time (**Figure 11.2**). A functional borehole downstream of the site was also considered to increase supply, but operational costs and risks were not favourable in the context. Even if the water harvesting solution was implemented, the water availability is only sufficient to support 25% of the garden area (totaling 2 ha) in the dry season. Augmentation with a sand dam would therefore allow greater food production in this critically poor location.

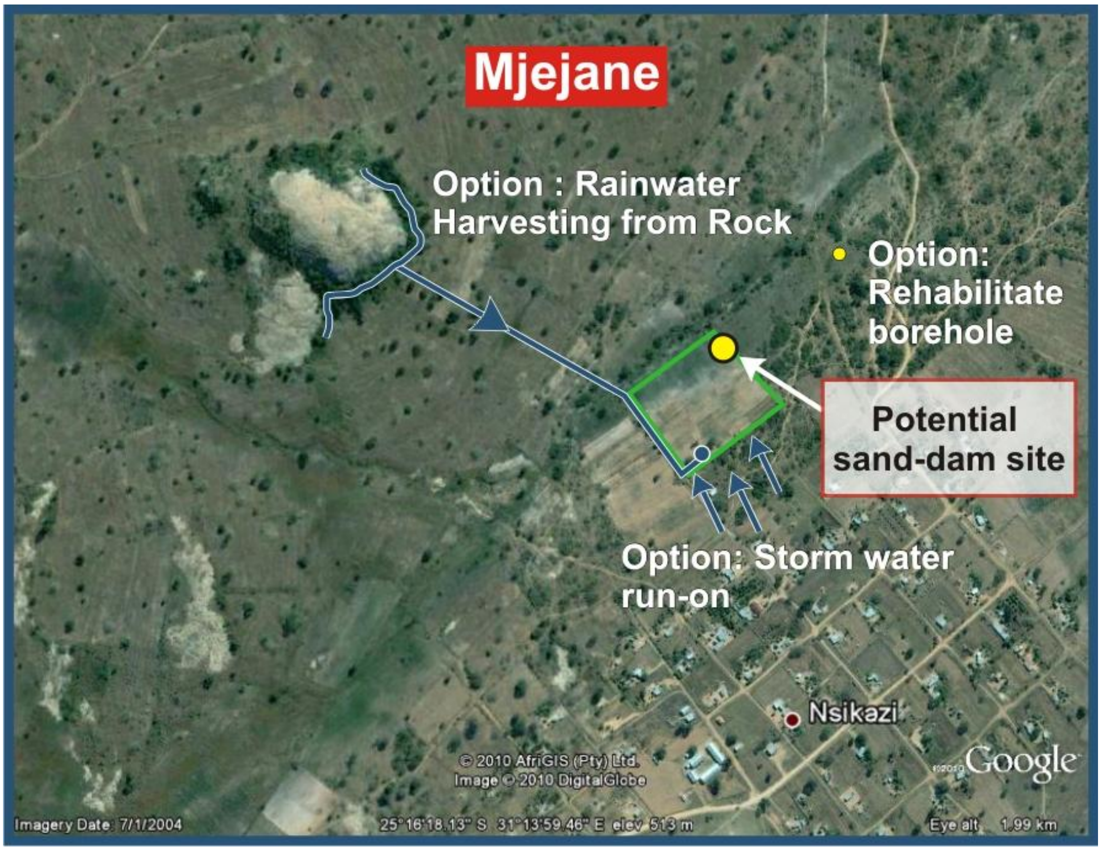


FIGURE 11.2 PROPOSED SAND DAM SITE (GOOGLE EARTH, 2010)

11.3 Luphisi Food Growers and Crafting Group

Location: Mpumalanga
Nearest Town: White River
Coordinates: S 25^{0} 24’ 41.25” E 31^{0} 15’ 57”
Potential sand dam use: food garden water supply (2.2ha) and cattle watering

Luphisi Village is situated next to the Kruger National Park and the Mthethomusha Reserve Park, about 35 km south-east of White River town. The village is more isolated than the distance from the town suggests. There are similarities between Luphisi and Mjejane, in terms of the generally sandy poor soils, underlying granite geology, rock dome outcrops, relative isolation, moderately low rainfall and village size. The area is very traditional and very poor.

The Vukani Group generates income from food gardening and crafting. Their shared 2.2 ha field with a rudimentary fence is located next to a seasonal stream fed by summer rains and a series of minor springs and seeps. The community group explained that the water supply from the adjacent stream had dropped significantly in the last ten years and the stream now only runs after heavy rain. The site presents a good opportunity for a sand dam with rock outcrop on both sides of the river bank in numerous places (**Figure 11.3**).

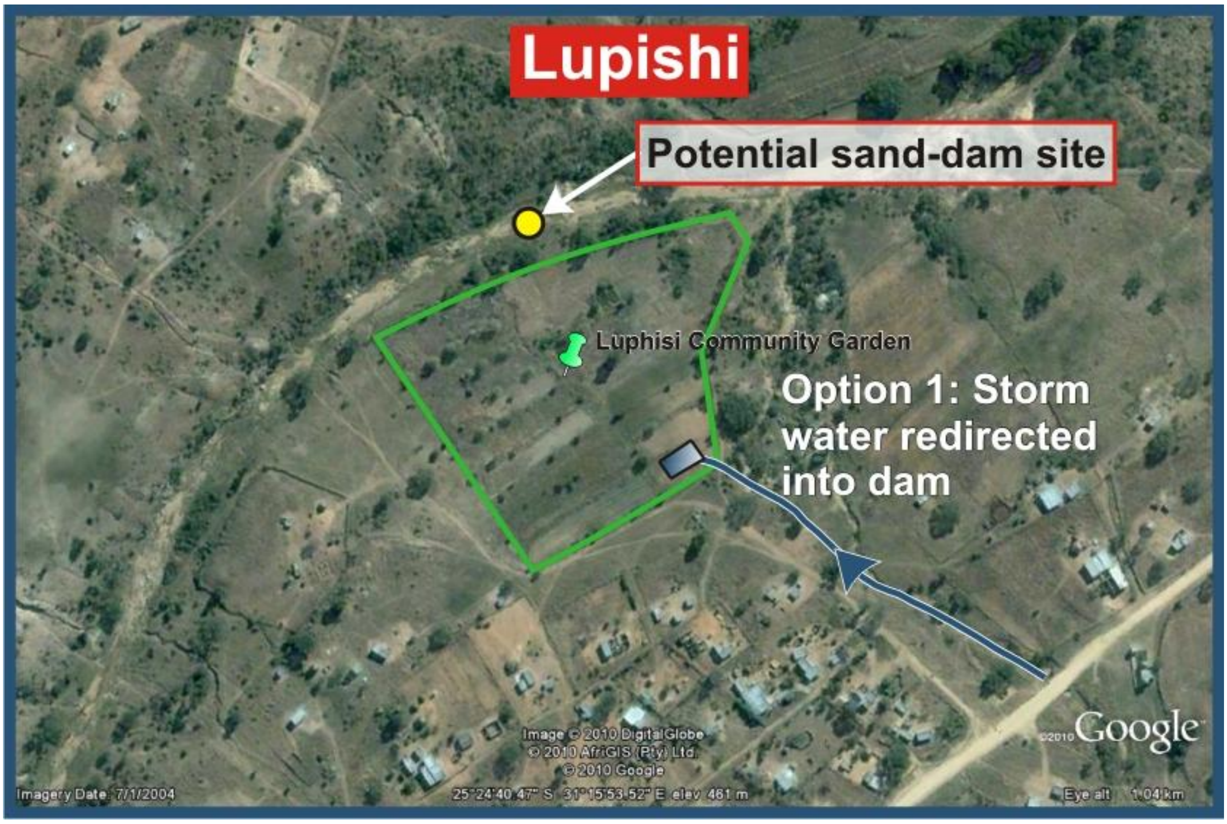


FIGURE 11.3 PROPOSED SAND DAM SITE (GOOGLE EARTH, 2010)

11.4 Grootfontein Irrigation Scheme

Location: Limpopo
Nearest Town: Polokwane
Coordinates: S 24^0 12' 48.84" E 29^0 54' 22.74"
Potential sand dam use: food garden water supply (10 ha)

Grootfontein is a defunct irrigation scheme located 90 km south east of Polokwane alongside the Olifants River. The flood scheme was built in 1954 and is supplied by a stream fed from the adjacent mountains. The 102 ha scheme is unused for a number of reasons such as the nearly dried up flow from the once 'Grootfontein' spring that feeds the stream. Now, only 10 ha of foodplots used by approximately 25 people still receive some water from the run-of-river arrangement thus water supply is limiting for food production in the gardens. The mountain stream has heavy sand loads and while no specific site for a sand-dam was identified, it is highly likely that numerous suitable sites exist along the stream (**Figure 11.4**).

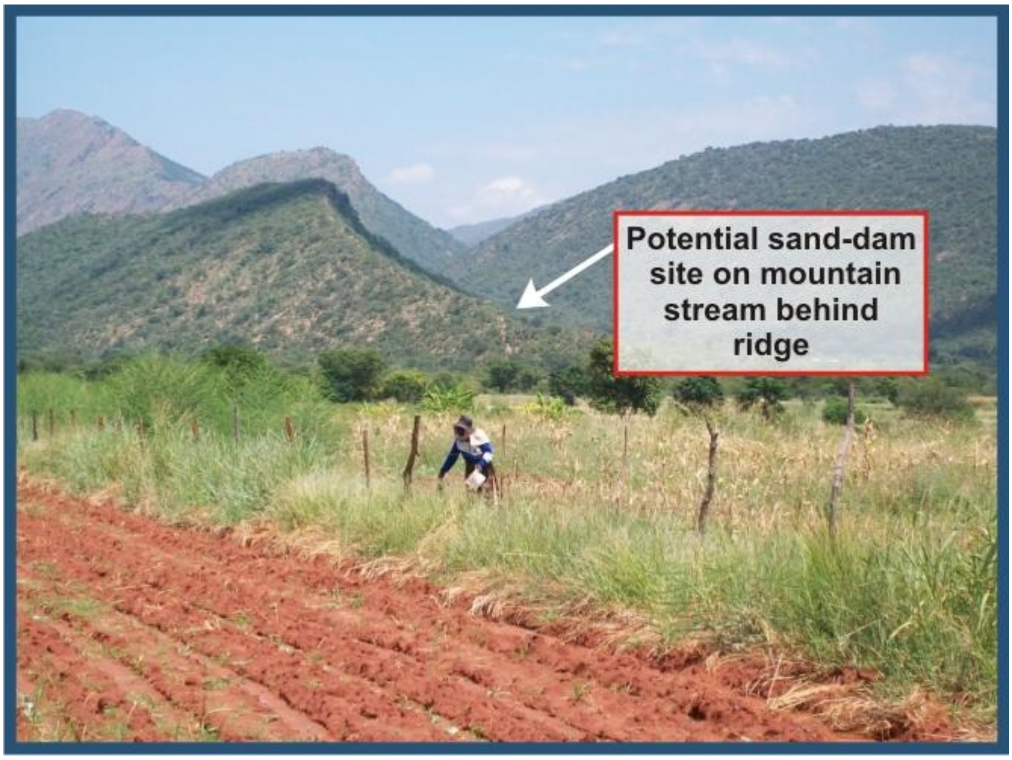


FIGURE 11.4 POTENTIAL SAND DAM SITE

11.5 Letaba - Homu (Giyani area)

Location: Limpopo
Nearest Town: Giyani
Coordinates of fields: S 230 18.272’ E 300 48.766‘
Coordinates of sand-dam site (approx): S 230 17.718’ E 300 47.446’
Potential sand dam use: supplementary irrigation supply and groundwater recharge (approx. 7 - 10 ha of drip irrigation)

Description and Potential Users

A sand-dam, or series of cascading sand-dams could supply gravity water to two emerging small-scale farm enterprises, currently wholly reliant on relatively-expensive, pumped groundwater.

Paul Hlakati (0726024694) is a remarkably energetic individual who previously farmed on the recently-collapsed Homu irrigation scheme until he was removed by Government in 2006 to make way for a ‘commercial partnership’ with R15 Million invested by Government. The scheme collapsed again after two years due to a range of problems, these include lack of planning, appropriate approach, negative profitability and insufficient water pumped from the Sami Dam. Paul Hlakati has since re-established himself separately from the defunct scheme. He has borrowed drip-lines and acquired land from the Tribal Authority, but struggles with borehole pumping costs and maintenance, which a gravity feed from a sand dam would alleviate. One of the other people who could benefit from a sand-dam is a young and newly appointed, progressively thinking Chief. He has just established his own small irrigation field with the intention of promoting food production and small-scale farming as a way out of poverty for the local people. Pumping costs are a major challenge and the practicable gravity supply from a sand-dam (or series of), even if supplementary to the existing boreholes, would lower their risk substantially, thereby increasing their resilience and profitability. The nearby upslope location of the sand-dams will probably re-charge the boreholes upon which these emerging enterprises depend. Both farmers market in Giyani Town, a locality with minimal other economic opportunity.

11.6 Lorraine Village (Northern Drakensberg)

Location: Limpopo – Tzaneen area
Nearest Town: Trichartsdal (Maruleng Local Municipality)
Coordinates of fields and stream for sand-dam: S 240 10.850 ’ E 300 25.242‘
Sand-dam site: numerous possible sites upstream towards mountain
Potential use: small irrigation and market gardening water supply

Description and Potential Users

Lorraine scheme was built in the 1960’s and only remnants of the original concrete-furrow system are still visible. An unusually determined group of people including young men, a local councillor and a Tribal Authority Councillor have taken it upon themselves to rebuild the primary components of the scheme without any external support, given the local poverty and unemployment (reported 95% local unemployment and complete lack of economic opportunities or Government support). This effort comprises a rock-pack diversion weir and small ‘main’ canal to about 40 plots, each surrounded with established mangoes, and cropped in summer with maize and mixed vegetables. Water runs off the northern slopes of the Northern Drakensberg and is sufficient in summer for supplementary irrigation. In winter minimal cropping is possible due to limited water supply. The sand-dam could, with a gravity pipeline, provide water to allow winter home-food production, and some market gardening impacting directly on local food security and hunger.

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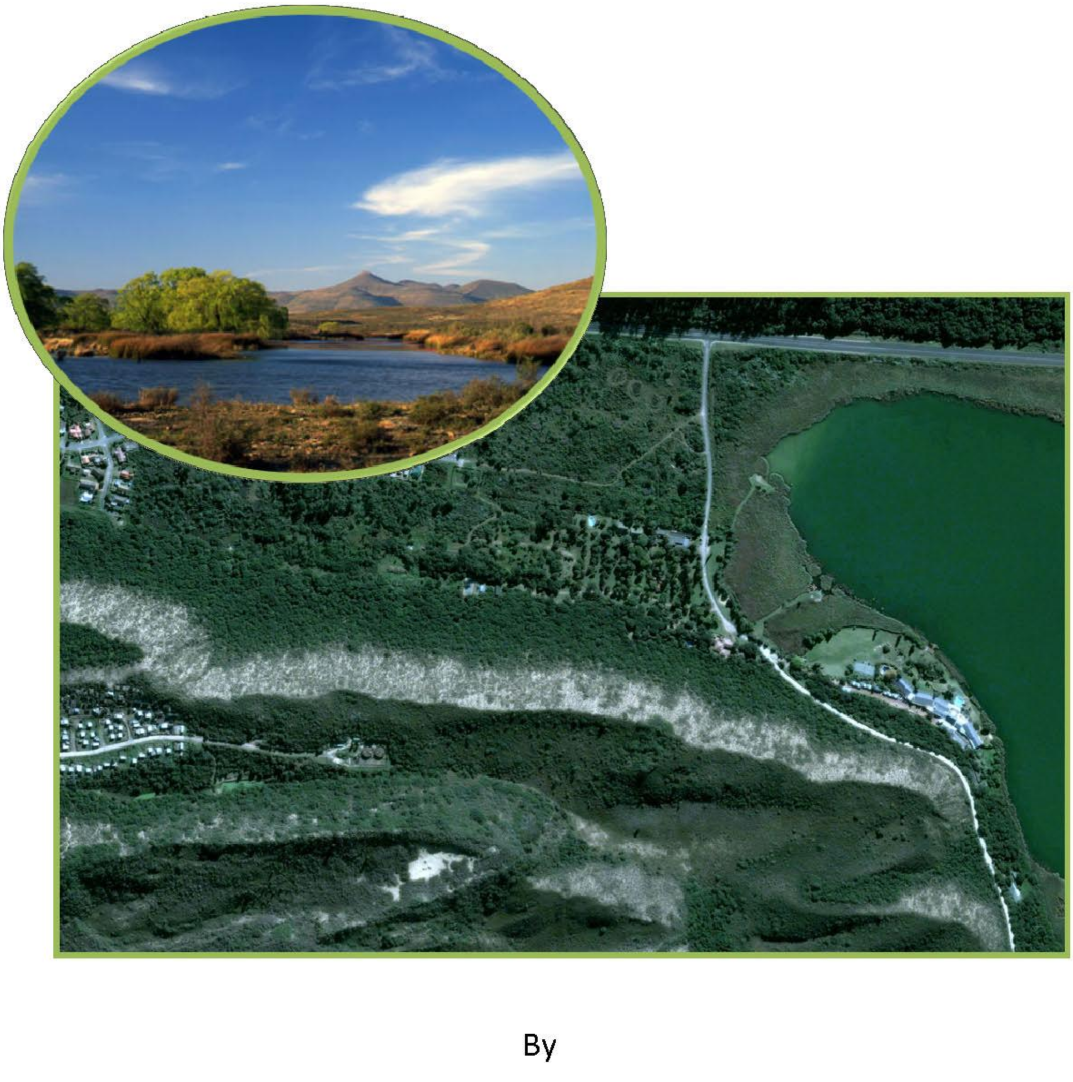
Appendix 1
WASTE WATER TREATMENT PLANT AT SEDGEFIELD NUMERICAL GROUNDWATER MODELLING

WASTE WATER TREATMENT PLANT AT SEDGEFIELD

Numerical Groundwater Modelling



By

Ingrid Dennis

June 2010

1. Numerical Modelling

1.1 Introduction

The simulation of groundwater flow and transport by numerical models is a relatively recent development, dating from the early 1970s. Today numerical models dominate the study of complex groundwater problems. Numerical models basically represent an assembly of many single-cell models. Tremendous advances in computer technology have made them the standard procedure for the solution of groundwater flow and mass transport models.

The numerical model solves both complex and simple problems. Once the numerical model is completed, various scenarios can be realised without undue effort. The dominance of the numerical models has led to the use of 'groundwater model' as a synonym for numerical groundwater models. The basic steps involved in modelling can be summarised as:

- Collecting and interpreting field data: Field data are essential to understand the natural system and to specify the investigated groundwater problem. The numerical model actually develops into a site-specific groundwater model when real field parameters are assigned. The quality of the simulations depends largely on the quality of the input data.
- Calibration & validation: Model calibration and validation are required to overcome the lack of input data, but they also accommodate the simplification of the natural system in the model. In model calibration, simulated values like water levels or concentrations are compared with field measurements. The model input data are altered within ranges, until the simulated and observed values are fitted within a chosen tolerance. Input data and comparison of simulated and measured values can be altered either manually or automatically.
- Modelling scenarios: Alternative scenarios for a given area may be assessed efficiently. When applying numerical models in a predictive sense, limits exist in model application. Predictions of a relative nature are often more useful than those of an absolute nature.

1.2 Assumptions and limitations

In order to develop a model of an aquifer system, certain assumptions have to be made. The following assumptions were made to develop the model:

- The system is initially in equilibrium and therefore in steady state, even though natural conditions have been disturbed.
- The available information on the geology and field tests were considered as correct.
- No abstraction boreholes were included in the initial model, however they were included in some of the scenarios.

- Many aquifer parameters have not been determined in the field and therefore have to be estimated.

It is important to note that a numerical groundwater model is a representation of the real system. It is therefore at most an approximation, and the level of accuracy depends on the quality of the data that is available. This implies that there are always errors associated with groundwater models due to uncertainty in the data and the capability of numerical methods to describe natural physical processes.

1.3 Generation of the finite difference network

In order to investigate the behaviour of aquifer systems in time and space, it is necessary to employ a mathematical model. MODFLOW (Harbaugh and McDonald, 1996) a modular three-dimensional finite difference groundwater flow model which was developed by U.S. Geological Survey is the software used during this investigation. It is an internationally accepted modelling package, which calculates the solution of the groundwater flow equation using the finite difference approach. A professional graphical interface, PMWIN, developed by Chiang and Kinzelbach (1999), was used to create the model and to analyse and display the modelling results.
The mesh constructed for the model consisted of 350 x 3100 cells in the x and y directions respectively. Superimposed are the surface water features. Each of the cells is 10 x 10 m. The coordinates for the modelled area are -18000, -3768300 (lower left corner) to -14500, -3765200 (upper right corner). The finite difference network for the study are is shown in Figure 1.

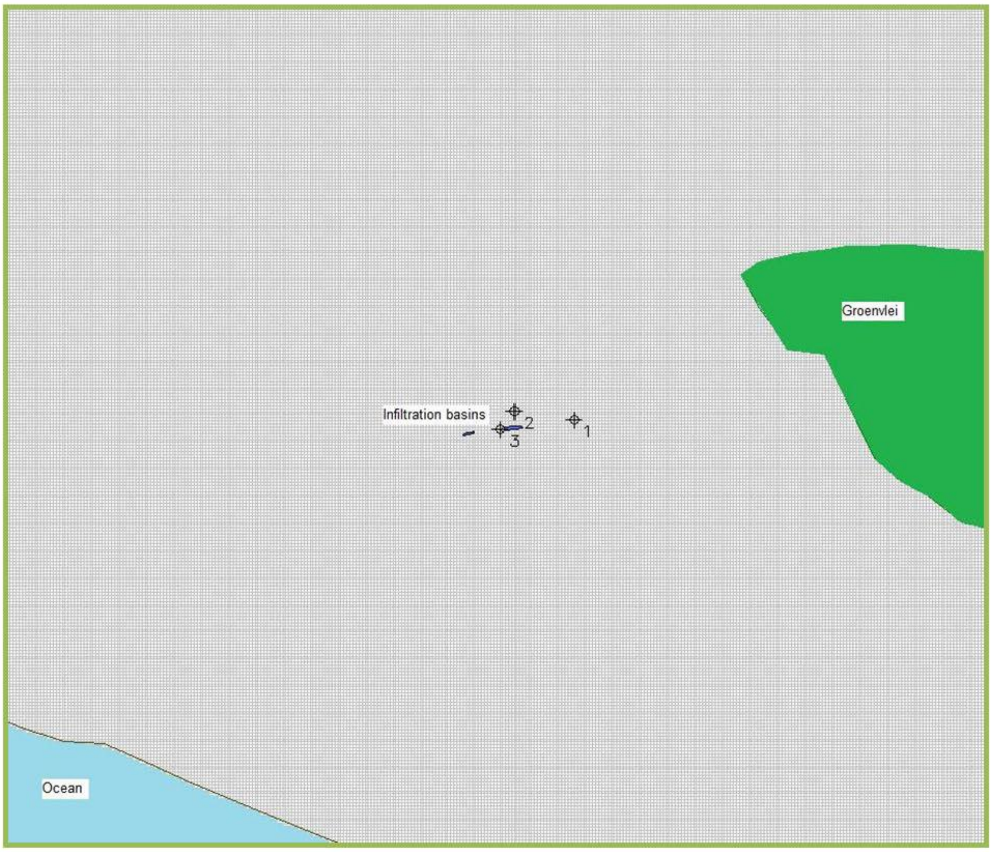


Figure 1: Finite difference network

Once the network has been set up, all initial and boundary conditions, sources, sinks, and aquifer parameters are entered.

1.4 Boundary conditions

One of the first and most demanding tasks in groundwater modelling is that of identifying the model area and its boundaries. Consequently, a model boundary is the interface between the model area and the surrounding environment. Conditions on the boundaries, however, have to be specified. Boundaries occur at the edges of the model area and at locations in the model area where external influences are represented, such as rivers, wells, and leaky impoundments.

Criteria for selecting hydraulic boundary conditions are primarily topography, hydrology and geology. The topography, geology, or both, may yield boundaries such as impermeable strata or potentiometric surface controlled by surface water, or recharge/discharge areas such as inflow boundaries along mountain ranges. The flow system allows the specification of boundaries in situations where natural boundaries are a great distance away.

Boundary conditions must be specified for the entire boundary and may vary with time. At a given boundary section just one type of boundary condition can be assigned. As a simple example, it is not possible to specify groundwater flux and groundwater head at an identical boundary section. Boundaries in groundwater models can be specified as:

- Dirichlet (also known as constant head or constant concentration) boundary conditions
- Neuman (or specified flux) boundary conditions
- Cauchy (or a combination of Dirichlet and Neuman) boundary conditions

The water level in the infiltration basins, Groenvlei and along the coastline were simulated using the Dirichlet boundary conditions.

1.3 Initial conditions

Initial conditions are vital for modelling flow problems. Initial conditions must be specified for the entire area. Generally, the initial water level/head distribution acts as the starting distribution for the numerical calculation.

1.4 Aquifer parameters

The aquifer parameters initially used to set up the model are listed in Table 1.

Table 1: Aquifer parameters

Layer	Description	Transmissivity[1] (m^2/d)	Specific yield[2]
1	Sands	300	0.2

1.5 Numerical flow model

A steady state groundwater flow model for the study area was constructed to simulate undisturbed groundwater flow conditions. These conditions serve as starting heads for the transient simulations of groundwater flow.

The simulation model (Modflow) used in this modelling study is based on three-dimensional groundwater flow and may be described by the following equation:

[1] Transmissivity is a measure of the ease with which groundwater flows in the subsurface. Transmissivity is related to hydraulic conductivity (K): T = Kd where d is the saturated thickness of the aquifer.

[2] The ratio of the volume of water that drains by gravity to that of the total volume of the saturated porous medium.

$$\frac{\partial}{\partial x}(K_x\frac{\partial h}{\partial x})+\frac{\partial}{\partial y}(K_y\frac{\partial h}{\partial y})+\frac{\partial}{\partial z}(K_z\frac{\partial h}{\partial z})\pm W=S\frac{\partial h}{\partial t} \quad (1)$$

where
h = hydraulic head [L]
Kx,Ky,Kz = Hydraulic Conductivity [L/T]
S = storage coefficient
t = time [T]
W = source (recharge) or sink (pumping) per unit area [L/T]
x,y,z = spatial co-ordinates [L]

For steady state conditions the groundwater flow Equation (1) reduces to the following equation:

$$\frac{\partial}{\partial x}(K_x\frac{\partial h}{\partial x})+\frac{\partial}{\partial y}(K_y\frac{\partial h}{\partial y})+\frac{\partial}{\partial z}(K_z\frac{\partial h}{\partial z})\pm W=0$$

1.6 Calibration of the steady state flow model

The steady state head distribution is dependent upon the recharge, transmissivity, sources, sinks and boundary conditions specified. For a given recharge component and set of boundary conditions, the head distribution across the aquifer under steady-state conditions can be obtained for a specific transmissivity value. The simulated head distribution can then be compared to the measured head distribution and the transmissivity or recharge values can be altered until an acceptable correspondence between measured and simulated heads is obtained. An advantage of a steady state model is that the parameter for storativity is not required to solve the groundwater flow equation therefore, there are less unknown parameters to determine.

The calibration process was done by changing the model parameters for transmissivity. Until the average infiltration from the infiltrations basins equated to the approximate capacity of the Waste Water Treatment Plant. An infiltration value of 825 m^3/d was achieved.

2. Results of Predictive Particle Tracking Scenarios

Seven scenarios were run with the numerical model, they are as follows:

- Scenario 1: The model is run without any abstraction and the time taken for the water from the infiltration basins to the boreholes shown in Figure 2 is estimated. The results of scenario 1 are shown in Figure 3. The results of the scenario are documented in Table 3.
- Scenario 2: The boreholes shown in Figure 2 are pumped according to the values shown in Table 2. The results of scenario 2 are shown in Figure 4. The results of the scenario are documented in Table 3.
- Scenario 3: Only boreholes W1 and W2 are pumped at 5l/s each. The results shown in Figure 5 and documented in Table 4.
- Scenario 4: Only boreholes SA1, SA2 and SA3 are pumped at 5l/s each. The results shown in Figure 6 and documented in Table 4.
- Scenario 5: Only boreholes SB1, SB2 and SB3 are pumped at 5l/s each. The results shown in Figure 7 and documented in Table 4.
- Scenario 6: Only boreholes SA1 and SA3 are pumped at 5l/s each. The results shown in Figure 8 and documented in Table 4.
- Scenario 7: Only boreholes SB1 and SB3 are pumped at 5l/s each. The results shown in Figure 9 and documented in Table 4.

Figure 2: Location of boreholes

Table 2: Abstraction rates for scenario 2

Borehole	Abstraction rate (l/s)
SA1	5
SA2	5
SB1	5
SB2	5
W1	5
W2	5

Table 3: Results of scenarios 1 and 2

Borehole	Time taken to reach borehole (days)	
	Scenario 1	Scenario 2
BH1	-	
BH2	280	
BH3	7	
SA boreholes	450	112
SB boreholes	1260	560
W boreholes	-	-

Table 4: Results of scenarios 3 to 7

Boreholes	Time to reach the boreholes (days)				
	Scenario 3	Scenario 4	Scenario 5	Scenario 6	Scenario 7
W 1 & W2	1500 +				
SA1, SA2 &SA3		120, 115 & 120		200, - & 200	
SB1, SB2 & SB3			710, 680 & 700		890, - &, 860

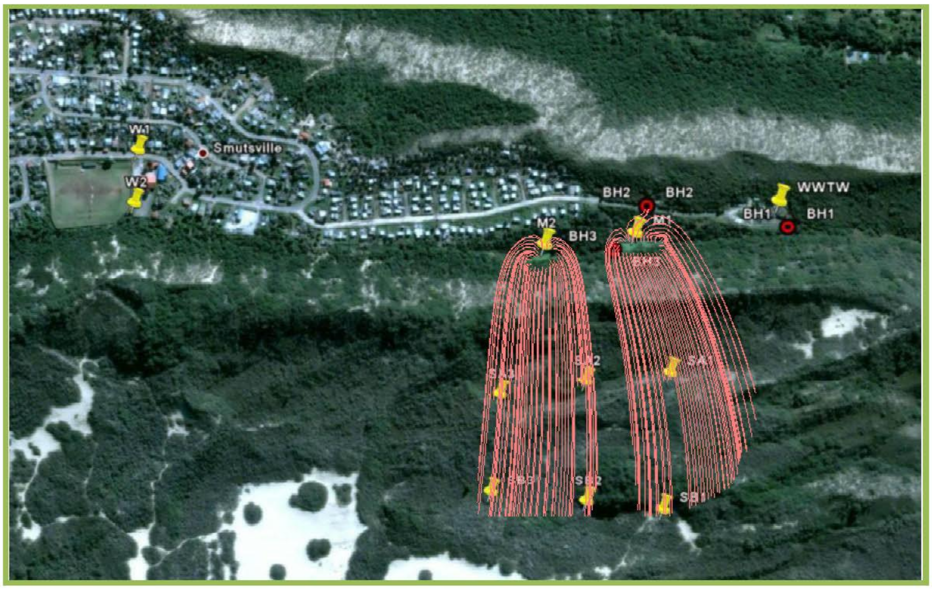


Figure 3: Results of scenario 1

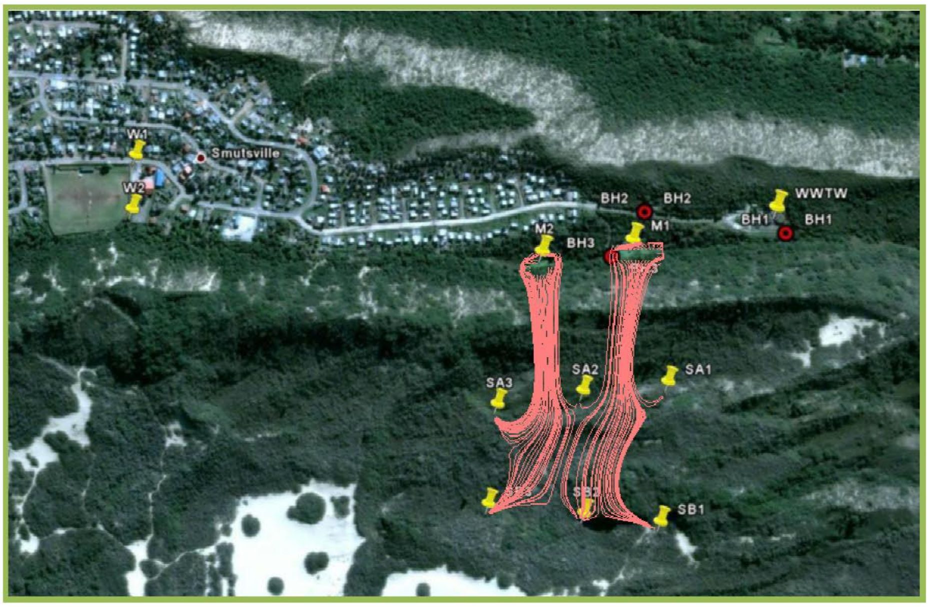


Figure 4: Results of scenario 2

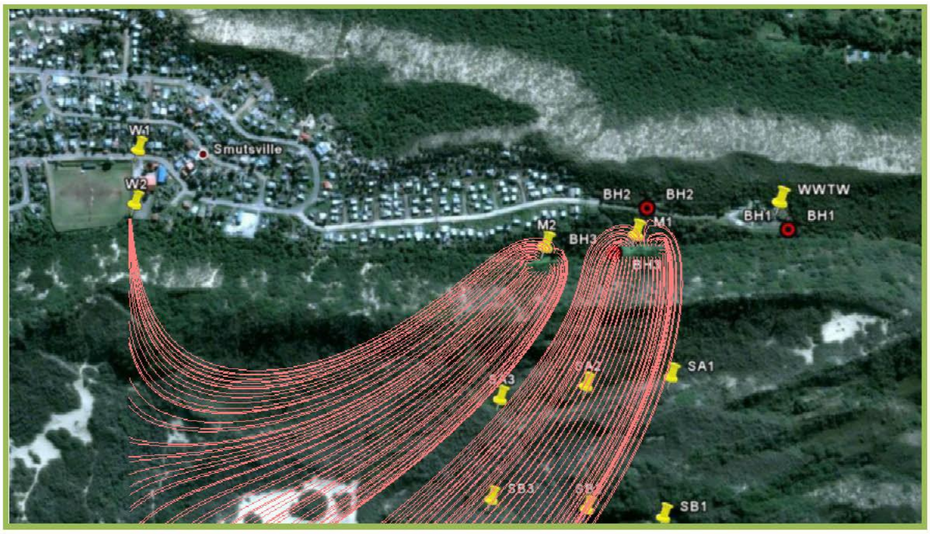


Figure 5: Results of scenario 3

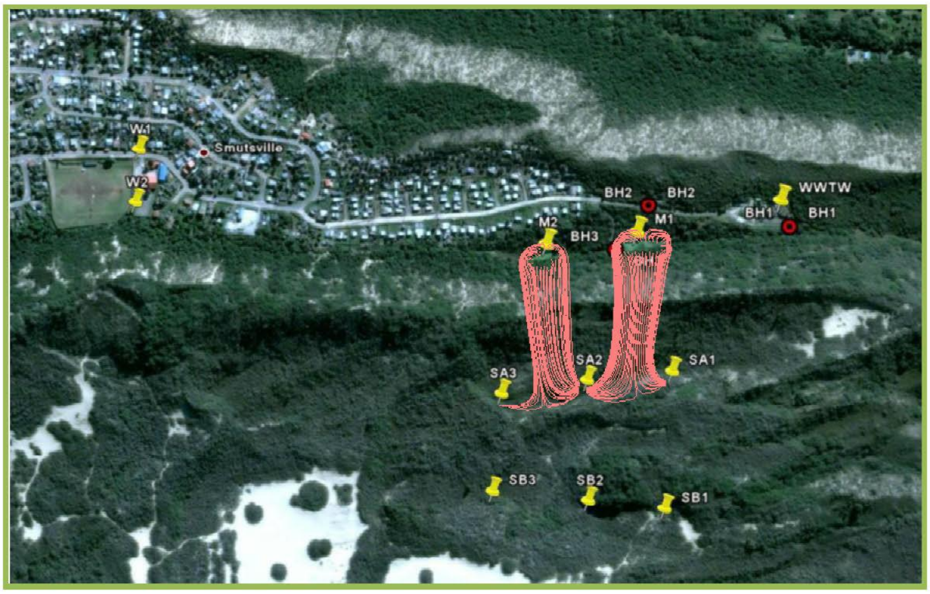


Figure 6: Results of scenario 4

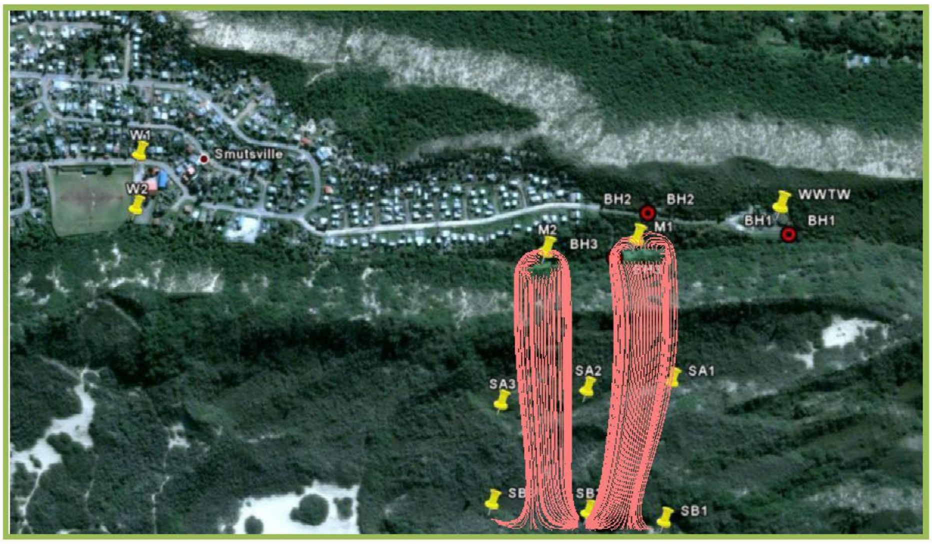


Figure 7: Results of scenario 5

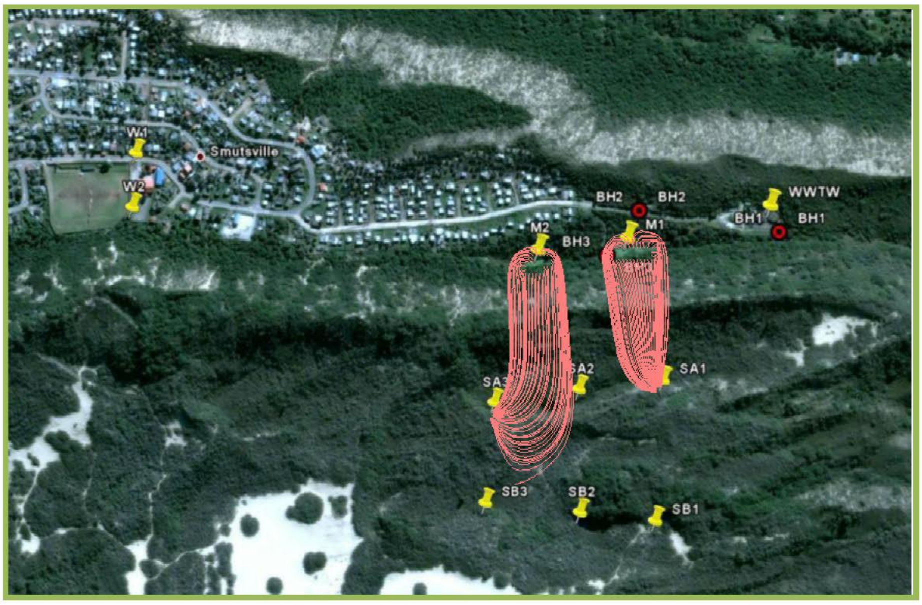


Figure 8: Results of scenario 6

Figure 9: Results of scenario 7

3. Mass Transport Model

3.1 Introduction

Mass transport modelling in this situation refers to the simulation of water contamination or pollution due to deteriorating water quality in response to man's disturbance of the natural environment. Transport through a medium is mainly controlled by the following two processes:

- Advection is the component of contaminant movement described by Darcy's Law. If uniform flow at a velocity V takes place in the aquifer, Darcy's law calculates the distance (x) over which a labelled water particle migrates over a time period t as x = Vt.
- Hydrodynamic dispersion comprises two processes:
    - Mechanical dispersion is the process whereby the initially close group of labelled particles are spread in a longitudinal as well as a transverse direction because of the velocity distribution (as a result of varying microscopic streamlines) that develops at the microscopic level of flow around the grain particles of the porous medium. Although this spreading is both in the longitudinal and transversal direction of flow, it is primarily in the former direction. Very little spreading can be caused in the transversal direction by velocity variations alone.
    - Molecular diffusion mainly causes transversal spreading by the random movement of the molecules in the fluid from higher contaminant concentrations to lower ones. It is thus clear that if V = 0, the contaminant is transported by molecular diffusion, or in other words the higher the velocity of the groundwater, the less the relative effect of molecular diffusion on the transportation of a labelled particle.

The MT3D software was used to provide numerical solutions for the concentration values in the aquifer in time and space. Input required in the software is:

- input concentrations of contaminants (unknown);
- transmissivity values;
- porosity values (assumed to be equivalent to specific yield);
- longitudinal dispersivities;
- transversal dispersivities;
- hydraulic heads/water levels in the aquifer over time.

As water levels for the entire modeling area were not available, topography as shown in Figure 10 were used as the initial water levels.

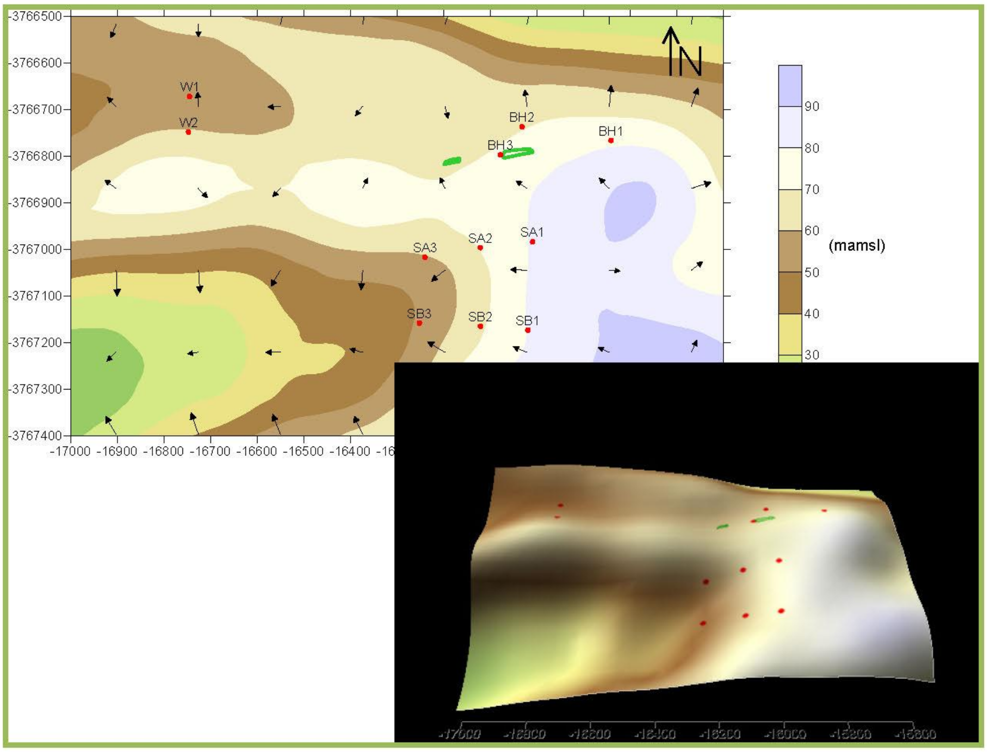


Figure 10: Topography

A 100% concentration was assumed at the infiltration basins. Boreholes W1 and W2 were pumped at 5 l/s each.

3.2 Results

The results after 8 years are shown in Figure 11. The model indicates that the waste water never reached the western boreholes. It is however recommended that the mass transport model be calibrated with more detailed information than available for the current model to ensure high confidence results.

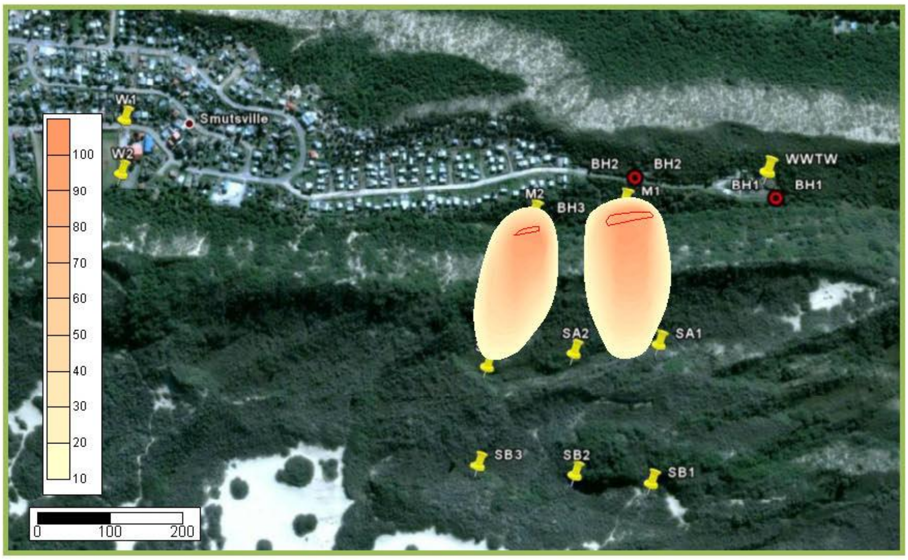


Figure 11: Results of mass transport scenario