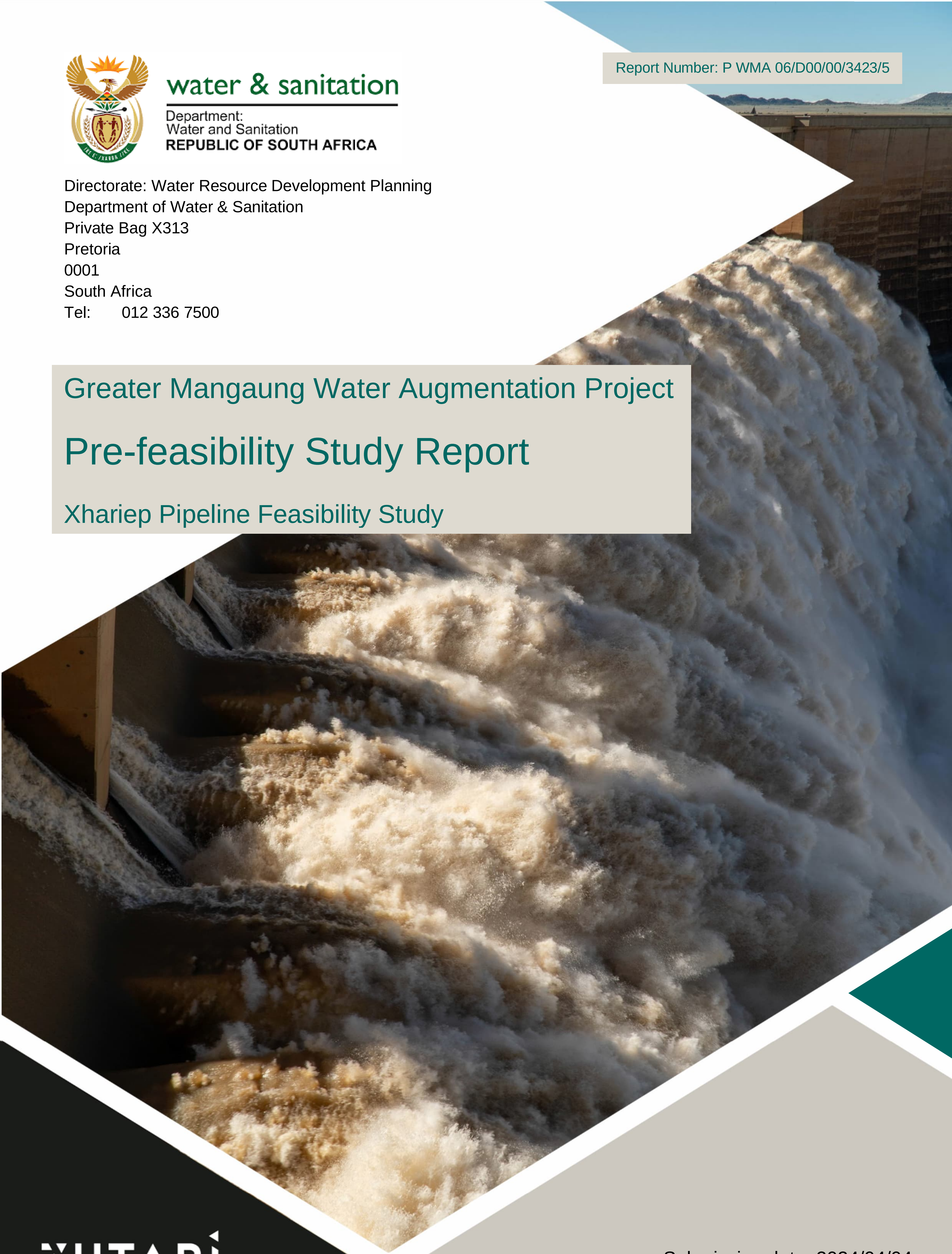

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Greater Mangaung Water Augmentation Project

Pre-feasibility Study Report

Xhariep Pipeline Feasibility Study

ZUTARI
IMPACT. ENGINEERED.

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REPUBLIC OF SOUTH AFRICA

Directorate Water Resource Development Planning

Pre-feasibility Study Report

APPROVAL

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DWS Report Number	:	P WMA 06/D00/00/3423/5
PSP Name	:	Zutari (Pty) Ltd
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Title	Principal Engineer	Title	Design Director

Report Structure

This report forms part of the following suite for the study:

REPORT INDEX	REPORT NAME	DWS NUMBER
1	Inception Report	P WMA 06/D00/00/3423/1
2	Site Visit Report	P WMA 06/D00/00/3423/2
3	Stakeholder Management Report	P WMA 06/D00/00/3423/3
4	Data Analysis and Collection Report	P WMA 06/D00/00/3423/4
5	**Pre-feasibility Study Report**	**P WMA 06/D00/00/3423/5**
6	Main Feasibility Study Report	P WMA 06/D00/00/3423/6
7	Geological and Materials Investigations Report	P WMA 06/D00/00/3423/7
8	Topographical Survey and Mapping Report	P WMA 06/D00/00/3423/8
9	Feasibility Design Report - Pipeline, Pump Stations & Reservoirs	P WMA 06/D00/00/3423/9
10	Socio-Economic Impact Assessment and Legal, Institutional and Financing Arrangements Report	P WMA 06/D00/00/3423/10
11	Feasibility Design Report - Water Treatment Works	P WMA 06/D00/00/3423/11
12	Land Matters	P WMA 06/D00/00/3423/12
13	DFFE Application Submission	P WMA 06/D00/00/3423/13
14	Public Participation Report	P WMA 06/D00/00/3423/14
15	DFFE BAR Submission	P WMA 06/D00/00/3423/15
16	Summary Feasibility Study Report	P WMA 06/D00/00/3423/16
17	Water Use License Application Summary Report	P WMA 06/D00/00/3423/17
18	Integrated Water and Waste Management Plan	P WMA 06/D00/00/3423/18
19	Water Resource Analysis Report	P WMA 06/D00/00/3423/19

Reference

This report is to be referred to in bibliographies as:

Department of Water and Sanitation, South Africa. 2023. *Greater Mangaung Water Augmentation Project – Xhariep Pipeline Feasibility Study: Pre-feasibility Study Report*.

DWS Report Number: P WMA 06/D00/00/3423/5

Prepared by Zutari (Pty) Ltd

Executive Summary

The conclusions and recommendations contain a detail description of the work undertaken as part of this pre-feasibility study report and are as such repeated below as the Executive Summary.

The existing GBWSS has experienced water restrictions since 2014 due to the inability of the existing infrastructure to supply the growth in water demand. Various studies have been undertaken by VCWB, MMM and DWS to identify options to augment the supply to the GBWSS.

The 2012 Reconciliation Strategy identified the following major interventions:

- Implementation of water conservation and water demand management,
- Increase capacity of Tienfontein pump station,
- Implementation of the Welbedacht / Knellpoort bi-directional pipeline, and,
- Implementation of re-use of treated effluent.

Other recommendations from the 2012 Reconciliation Strategy include:

- Addressing the siltation problems at Welbedacht WTW to increase the operating capacity of the plant,
- Improving the integrity of the Welbedacht pipeline, and
- Increasing the capacity of the Maselspoort WTW and raise Mockes Dam.

The above interventions and recommendations were considered the most economical options that can be implemented in the shortest possible timeframes. The 2012 Reconciliation Strategy also identified the transfer to water from Gariep Dam as the next augmentation scheme to be considered after implementation of the above interventions.

MMM and VCWB both investigated the transfer of water from Gariep Dam and came to different conclusions on the preferred solution, i.e.:

- MMM concluded that a direct pipeline from Gariep Dam to Bloemfontein, conveying potable water, will be the optimal solution, and
- VCWB concluded that a pipeline from Gariep Dam to Knellpoort Dam, conveying raw water, will be the optimal solution.

As a result, DWS decided to initiate this study, referred to as the “Greater Mangaung Water Augmentation Project – Xhariep Pipeline Feasibility Study”. The purpose of this study is to appraise, at a pre-feasibility level of detail, the most viable previously identified development options (routes) for the Xhariep Pipeline Project and to recommend the optimal system size (including phasing) and the best water conveyance route from a regional and national perspective that should be taken forward to the feasibility stages of study.

The three most feasible pipeline route options identified from previous studies were:

- Scheme 1: Direct potable pipeline from Gariep Dam to Bloemfontein,
- Scheme 2: Raw water pipeline from Gariep Dam to Knellpoort Dam, and
- Scheme 3: Raw water pipeline from Gariep Dam to the Novo Outfall Structures.

A fourth scheme, referred to as Scheme 4, was identified at the commencement of this study. Scheme 4 is a raw water pipeline from Gariep Dam to Rustfontein Dam, which aims to reduce the losses associated with Scheme 3 where water will be conveyed along the upper reaches of the Modder River before being discharged into Rustfontein Dam. The pipeline routes for the four schemes are shown in Figure 1-2.

Feasibility studies are iterative in nature as interventions/schemes are required such that the water resources yield matches the forecasted water demands, followed by infrastructure option identification, refinement of the yield modelling, refinement of the infrastructure sizing, etc. This iterative process, and the steps followed for this study, is shown in Figure 1-3 (repeated below as Figure E-1 for ease of reference).

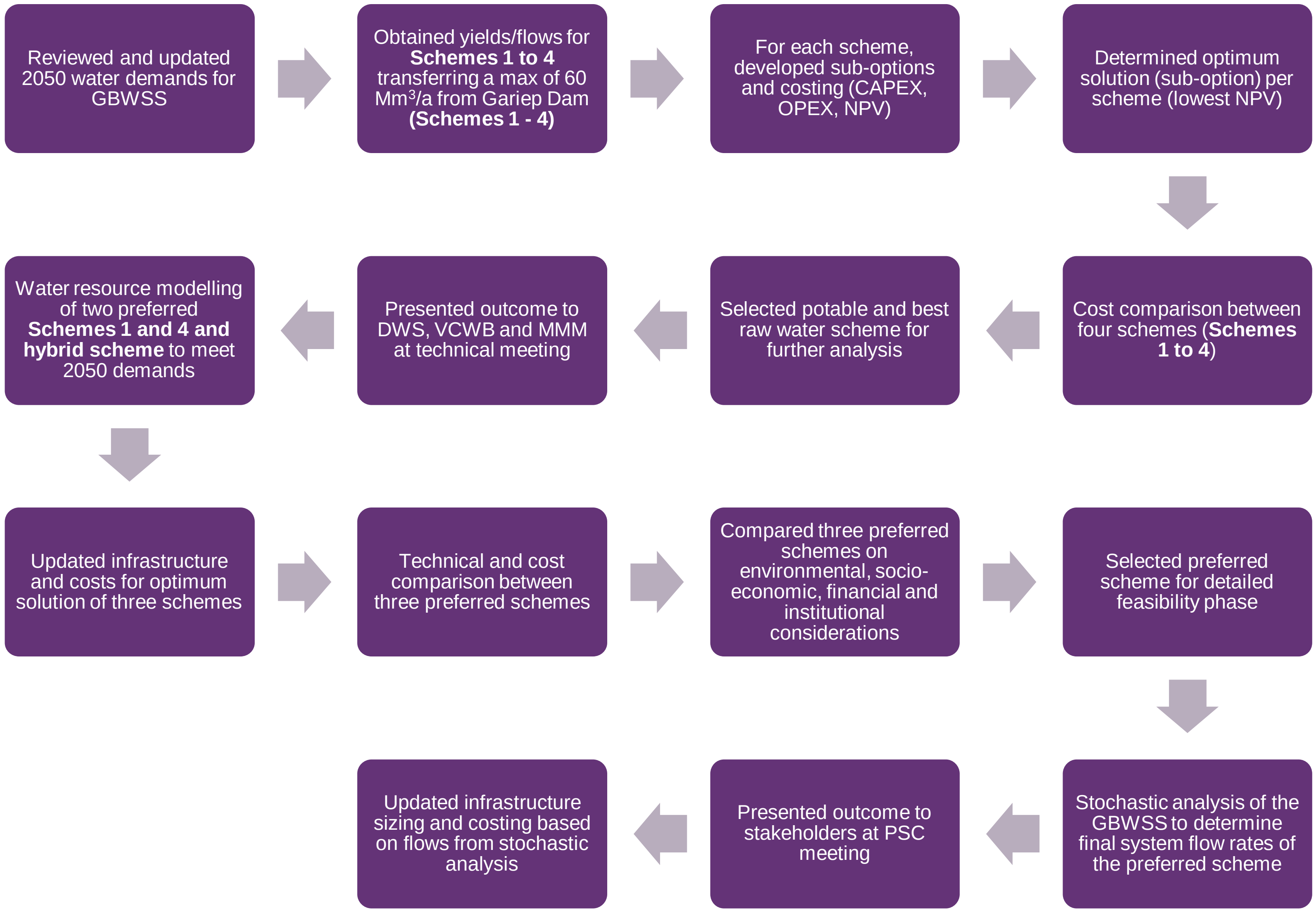

Figure E-1: Flow chart of scheme development process

Previous studies included water demand projections to 2035 or 2040, whereas the planning horizon adopted for this project was 2050. The previous studies recommended annual growth rates in water demands that varied from 1% to 3%. This study evaluated the water demands from two different approaches, i.e. (1) based on historic water demands and assuming a similar growth rate over time, and (2) based on published population data by Stats SA and accounting for an improvement in level of service so that all households will have a house connection by 2050. The actual water demands in 2014 were used as the starting point for the demand projections. Table E- 1 provides a comparison of the water demands calculated in the previous studies against the demands calculated in this study (i.e. 'Observed Projected' is based on historical growth in demand, and 'Scenario 2' is based on population data and improvement in level of service).

Table E- 1: Total comparison results

Source	2023		2035		2050		Average % Increase
	mil m³/a	Mℓ/d	mil m³/a	Mℓ/d	mil m³/a	Mℓ/d	
Observed Projected (this study)	118.21	323.86	145.55	398.78	179.74	492.42	1.56%
Scenario 2 (this study)	110.96	303.99	140.17	384.02	186.40	510.70	1.94%
2012 Reconciliation Strategy	133.92	366.91	186.78	511.74	286.49	784.89	2.86%
2015 Technical Feasibility Study	115.79	317.23	146.85	402.33	197.64	541.48	2.00%
2018 Mangaung Study	114.71	314.26	139.35	381.78	179.86	492.76	1.68%
2022/23 AOA	108.48	297.20	139.25	381.51	185.16	507.29	2.00%

It is evident from Table E- 1 that the 2050 demand projections calculated as part of this project came to 180 million m^3/a and 186 million m^3/a, respectively. This compared favourably with the demand projections of 180 million m^3/a and 185 million m^3/a determined as part of the 2018 Mangaung Study and the 2022/23 Annual Operating Analysis. The demand determined as part of Scenario 2, i.e. 186 million m^3/a, was adopted as the 2050 water demand that had to be satisfied for the GBWSS. Scenario 2 also included the demands of the towns and villages located within 10 km from the proposed pipeline routes, should these towns wish to connect to the proposed Xhariep Pipeline.

The ToR for this study recommended that the first phase of the water resources yield modelling be based on transferring a maximum volume of 60 million m^3/a from Gariep Dam. Table E-2 summarises the HFY determined for each of the four schemes as well as the percentage of the 2050 demands that can be met by each scheme for the major demand centres.

Table E-2: System Historic Firm Yield based on 60 million m³/a transfer from Gariep Dam

Scheme	Historic Firm Yield (million m³/a)	Percentage of 2050 demands met	
		Bloemfontein (%)	Botshabelo & Thaba Nchu (%)
1 (potable water to Bloemfontein)	131	59.1	84.3
2 (raw water to Knellpoort Dam)	119	44.3	92.6
3 (raw water to Novo Outfall Structure)	120	43.1	96.2
4 (raw water to Rustfontein Dam)	134	55.2	97.1

It is evident from Table E-2 that (a) the HFY differs from scheme to scheme, (b) the HFY was considerably lower than the 2050 demand of 186 million m^3/a, and (c) a higher volume would need to be transferred from Gariep Dam to satisfy the 2050 demand.

The infrastructure required for each of the four schemes, based on a maximum transfer volume of 60 million m^3/a from Gariep Dam, was determined and costed to undertake a comparison of the schemes. Multiple sub-options were developed for each scheme where different pipeline diameters and pump stations positions were evaluated. The purpose of these sub-options was to optimise the infrastructure for each scheme.

The infrastructure for each scheme and sub-option was sized based on the peak flows derived from the water resources yield modelling, whereas the operating and maintenance costs were calculated from the average annual flows determined by the yield modelling. The operating and maintenance costs were converted to a NPV using a discount rate of 6% and a discount period of 45 years. Given the different HFY of the four schemes, the yields were also converted to NPVs, which allowed URVs to be calculated. The URV for each sub-option was the total NPV of the costs divided by the NPV of the water demands. Table 9-6 (repeated below as Table E-3 for ease of reference) shows the NPV and URV calculated for the preferred sub-options of Schemes 1, 2, 3 and 4. It is evident from Table E-3 that Scheme 4 was the most economical raw water scheme (compared to Schemes 2 and 3) and that Scheme 1 was 7.5% more expensive than Scheme 4.

Based on the NPVs and URVs shown in Table E-3, it was decided to undertake the additional water resources yield modelling for Schemes 1 (potable option) and 4 (most economical raw water option). A stakeholder engagement with DWS, MMM and VCWB took place on 2 November 2023 where feedback was provided on progress to date and where operational matters could be discussed. The following specific matters were raised at the meeting:

- Botshabelo and Thaba Nchu were experiencing higher levels of restriction compared to other towns within the GBWSS, mainly as these two towns only have Rustfontein WTW as supply whereas Bloemfontein can receive water from Welbedacht, Rustfontein and Maselspoort WTWs,
- VCWB preferred Scheme 2 (raw water supply to Knellpoort Dam) due to greater operational flexibility, e.g. raw water can be supplied from Knellpoort Dam to Welbedacht Dam as well as to Rustfontein and Maselspoort WTWs,
- The supply of potable water to towns located along the proposed pipeline route remains a priority from a regional water supply perspective,
- Scheme 1 is the only potable scheme under consideration but can only supply Bloemfontein and the towns along the pipeline route, i.e. it would not resolve the challenges experienced at Botshabelo and Thaba Nchu, and
- All parties agreed that Scheme 1 and Scheme 4 had limitations in terms of overall flexibility and improving the resilience of the GBWSS.

This led to the development of Scheme 1B (also referred to as the “hybrid” scheme since the pipeline route is a combination of the routes for Schemes 1 and 4) as shown in Figure 7-6 and Figure 7-7 (repeated below as Figure E-2 for ease of reference) where potable water would be supplied from Gariep Dam to a command reservoir located between Bloemfontein and Rustfontein WTW. Water from the command reservoir could then gravitate to Bloemfontein and Rustfontein WTW.

The water resources yield modelling was updated for Schemes 1 and 4 (now referred to Schemes 1A and 4B to distinguish them from Schemes 1 and 4), as well as Scheme 1B, to determine the transfer volume required from Gariep Dam that would satisfy the 2050 demands. Table E-4 summarises the maximum annual transfer volumes required from Gariep Dam and the percentage of demand that could be supplied for each of the large demand centres.

Table E-3: Results of financial comparison for original schemes 1 to 4 with maximum annual transfer of 60 million m3/a

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Present Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme comparison of original transfer at 60 Mm³ / annum	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**Mℓ**	**Mm³/annum**	**Mm³/annum**	**Mm³**	**Million Rands**	**(Million Rands)**	**(Million Rands)**	**R/m³**	**R/m³**	**%**
Scheme 1 [Sc5b] Potable water from Gariep Dam to Bloemfontein	181.2	DN1600	1411 | 348	2148 | 367	-	-	165	133.2	44.5	1986	11895	12543	24438	6.32	12.31	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1434 | 199	-	-	-	-	-	45.2	1986	0	1384	1384	0.70	0.70	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1293 | 247	-	-	-	-	-	40.8	1986	0	1549	1549	0.78	0.78	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	734 | 136	-	-	-	-	-	23.2	1986	0	485	485	0.24	0.24	-
										Total	**11895**	**15962**	**27857**	**8.04**	**14.03**	**+7.5**
Scheme 2 [Sc4b(ii)] Raw water from Gariep Dam to Knellpoort Dam	190.4	DN1300	1035 | 298	1901 | 418	1035 | 132	1901 | 207	64	115.3	32.6	1797	9948	11648	21597	6.48	12.02	-
+ Maselspoort pipeline and PS upgrades	33.5	DN800	257 | 135	570 | 210	-	-	Incl above	Incl above	8.1	1797	498	423	921	0.24	0.51	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1297 | 194	-	-	-	-	-	40.9	1797	0	1224	1224	0.68	0.68	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	963 | 244	-	-	-	-	-	30.4	1797	0	1142	1142	0.64	0.64	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	2438 | 185	-	-	-	-	-	76.9	1797	0	2194	2194	1.22	1.22	-
										Total	**10446**	**16632**	**27078**	**9**	**15.07**	**+15.4**
Scheme 3 [Sc4c(i)] Raw water from Gariep Dam to Novo outfall	197.8	DN1300 and DN1400	1645 | 361	1901 | 377	1645 | 231	1901 | 248	64	116.5	51.9	1808	8918	13811	22729	7.64	12.57	-
+ Maselspoort upgrades	33.5	DN800	235 | 133	570 | 210	-	-	Incl above	Incl above	7.4	1808	498	405	903	0.22	0.50	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1373 | 197	-	-	-	-	-	43.3	1808	0	1312	1312	0.73	0.73	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	985 | 244	-	-	-	-	-	31.1	1808	0	1169	1169	0.65	0.65	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	1054 | 141	-	-	-	-	-	33.2	1808	0	723	723	0.40	0.40	-
										Total	**9416**	**17421**	**26836**	**10**	**14.85**	**+13.7**
Scheme 4 [Sc5f] Raw water from Gariep Dam to Rustfontein Dam	203.9	DN1300	1431 | 378	1901 | 412	1431 | 78	1901 | 104	64	130.3	45.1	1955	8473	12718	21191	6.51	10.84	-
+ Maselspoort upgrades	33.5	DN800	299 | 140	570 | 210	-	-	Incl above	Incl above	9.4	1955	498	459	957	0.24	0.49	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1390 | 197	-	-	-	-	-	43.8	1955	0	1332	1332	0.68	0.68	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1219 | 246	-	-	-	-	-	38.4	1955	0	1457	1457	0.75	0.75	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	863 | 138	-	-	-	-	-	27.2	1955	0	578	578	0.30	0.30	-
										Total	**8971**	**16544**	**25515**	**8**	**13.05**	**100**

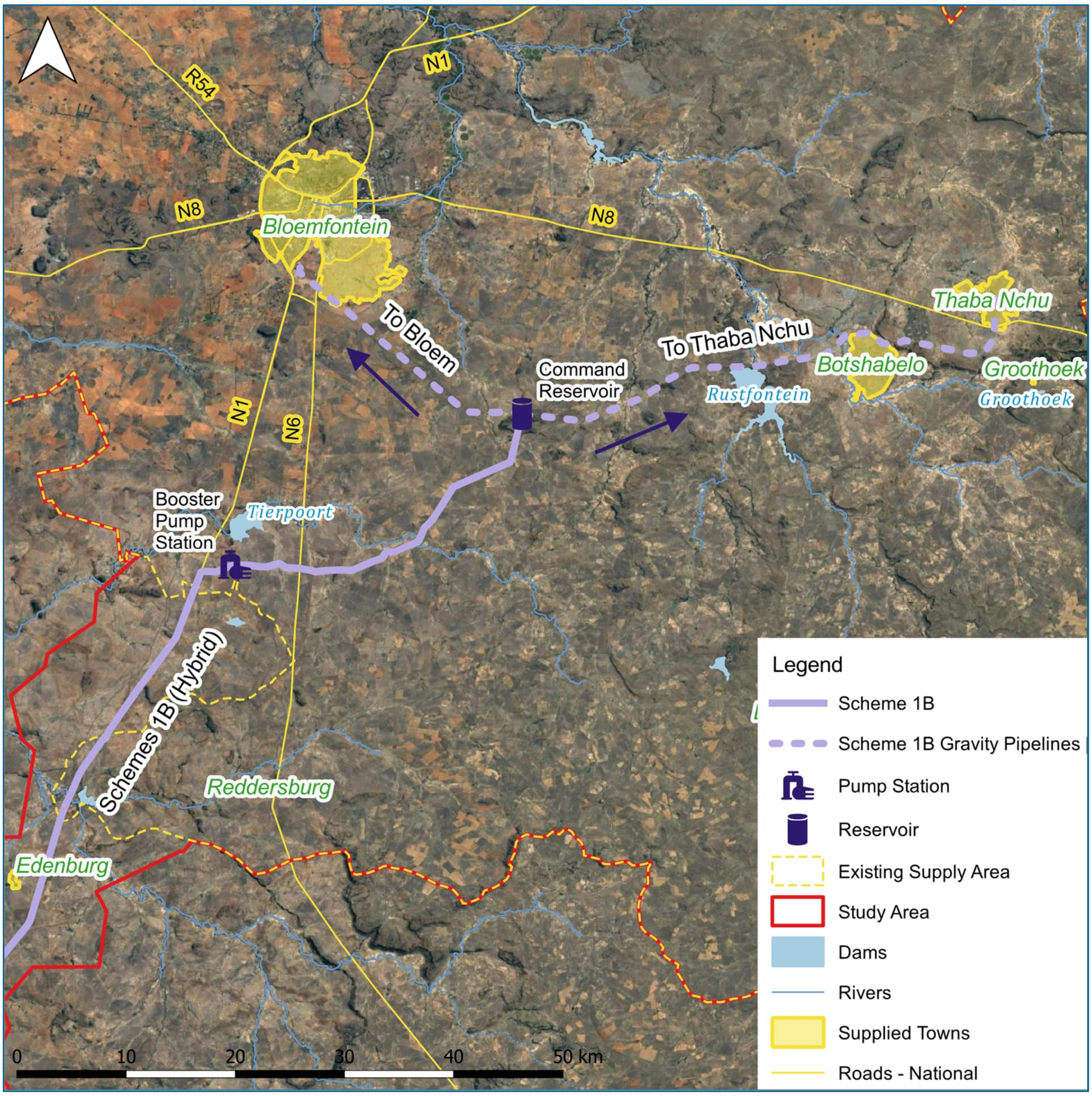

Figure E-2: Scheme 1B supply to Bloemfontein and Rustfontein WTW

Table E-4: Historic Firm Yield required from Gariep Dam to satisfy 2050 demands

Scheme	Historic Firm Yield (million m^3/a)	Maximum transfer volume (million m^3/a)	Percentage of 2050 demands met	
			Bloemfontein (%)	Botshabelo & Thaba Nchu (%)
1A (potable to Bloemfontein)	186	120	100.0	84.4
1B (potable to regional command reservoir)	186	120	100.0	99.6
4B (raw water to Rustfontein Dam)	186	142	100.0	100.0

It is evident from Table E-4 that Schemes 1A and 4B would not be able to supply 100% of the demands for Botshabelo and Thaba Nchu, mainly due to bottlenecks in the existing GBWSS infrastructure. The infrastructure sizing and cost estimates were updated for Schemes 1A, 1B and 4B with the calculated NPV and URV information shown in Table E-5.

Table E-5: Results of financial comparison for additional schemes 1A, 4B and 1B (Hybrid) with maximum annual transfer of 142 million m³/a

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Present Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme comparison of increased transfer at 142 Mm³ / annum	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**Mℓ**	**Mm³/annum**	**Mm³/annum**	**Mm³**	**Million Rands**	**Million Rands**	**Million Rands**	**R/m³**	**R/m³**	**%**
Scheme 1A [Sc5b] Potable water from Gariep Dam to Bloemfontein	181.2	2 x DN1500	2925 | 356	5085 | 399	2925 | 89	5085 | 119	390	185.9	92.2	2208	25120	21983	47103	9.96	21.33	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	941 | 185	-	-	-	-	-	29.7	2208	0	844	844	0.38	0.38	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	726 | 243	-	-	-	-	-	22.9	2208	0	856	856	0.39	0.39	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	1926 | 165	-	-	-	-	-	60.7	2208	0	1544	1544	0.70	0.70	-
										Total	**25120**	**25226**	**50347**	**11.42**	**22.80**	**+3.1**
Scheme 4B [Sc5f] Raw water from Gariep Dam to Rustfontein Dam	203.9	2 x DN1400 & DN1600	3016 | 367	4500 | 408	-	-	189	182.2	95.1	2208	21317	20941	42258	9.48	19.14	-
+ Maselspoort upgrades	33.5	DN800	706 | 217	570 | 210	-	-	Incl above	Incl above	22.3	2208	505	1034	1539	0.47	0.70	-
+ New pipeline from Rustfontein to Bloemfontein	50.2	DN1000	63 | 98	920 | 242	-	-	-	-	-	2208	914	199	1112	0.09	0.50	-
+ Rustfontein pump upgrades + operating cost (to Bloemfontein)	Varies	Equivalent DN1400	63 | 98	1440 | 156	-	-	-	-	2.0	2208	119	265	384	0.12	0.17	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1529 | 249	-	-	-	-	-	48.2	2208	0	1849	1849	0.84	0.84	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	475 | 133	-	-	-	-	-	15.0	2208	0	307	307	0.14	0.14	-
										Total	**22855**	**25992**	**48847**	**12**	**22.12**	**100**
Scheme 1B [Sc5b] Potable water from Gariep Dam to Rustfontein	186.1	2 x DN1400	2776 | 362	4294 | 401	2776 | 210	4294 | 233	330	182.2	87.5	2208	21846	24675	46521	11.17	21.07	-
+ Gravity pipeline to Longridge reservoir from command reservoir	26.0	DN1200	-	-	-	-	-	-	69.2	2208	663	144	808	0.07	0.37	-
+ Gravity pipeline to Rustfontein from command reservoir	25.7	DN1100	-	-	-	-	-	-	50.7	2208	590	128	718	0.06	0.33	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	-
										Total	**23099**	**27125**	**50224**	**12.28**	**22.75**	**+2.8**

It is evident from Table E-5 that the URVs of Schemes 1A, 1B and 4B were within 3% of each other and therefore considered comparable from a financial perspective. Given that Scheme 1A can only supply 84.4% of the demands to Botshabelo and Thaba Nchu, and that Scheme 4B is a raw water scheme that cannot supply the towns along the pipeline route, it was proposed that Scheme 1B be considered for implementation.

The water resources yield modelling was further optimised for Scheme 1B by testing different operating rules and maximising the utilisation of existing and proposed infrastructure. This optimisation process resulted in reducing the maximum transfer volume from 120 million m^3/a to 101 million m^3/a.

A stochastic analysis was subsequently undertaken for Scheme 1B to confirm that the 2050 demand can be delivered at a minimum of 98% assurance of supply (i.e. a 1:50 year recurrence interval). Figure E-4 shows the outcome of the stochastic analysis, which indicates that yields of approximately 220 million m^3/a and 213 million m^3/a can be delivered at 98% and 99% assurance of supply, respectively. It is recommended that the maximum transfer volume from Gariep Dam remains at 101 million m^3/a as the higher assurance of supply provides flexibility should additional towns be included in future as part of the GBWSS or to cater for any unforeseen delays experienced with the implementation of any of the 2012 Reconciliation Strategy interventions.

Upon completion of the stochastic analysis and based on a maximum transfer volume of 101 million m^3/a, three alternative configurations for Scheme 1B were considered as part of the design optimisation process (refer to Figure 7-10). These alternative configurations mainly evaluated different locations for the booster pump station, different elevations for the second command reservoir, and connecting pipeline sizes between the second command reservoir and Bloemfontein as well as between the command reservoir and Rustfontein WTW. The NPVs and URVs for the three configurations were determined and are shown in Table E-6. The URVs of the three configurations differ by less than 2%, meaning that the configurations are comparable from a financial perspective.

A site visit was undertaken in January 2024 to evaluate the various infrastructure sites in terms of topography, impact on farming activities, location relative to existing access roads and powerlines, as well as any other observations that could impact the feasibility of the sites. Operational aspects were also considered, e.g. preference will be given to configurations where demands can be met under gravity flow, rather than flow being pumped. Based on the findings of the site visit and accounting for operational considerations, it is recommended that the detailed feasibility design proceed based on Configuration 1B1(A) as shown in Figure E-3.

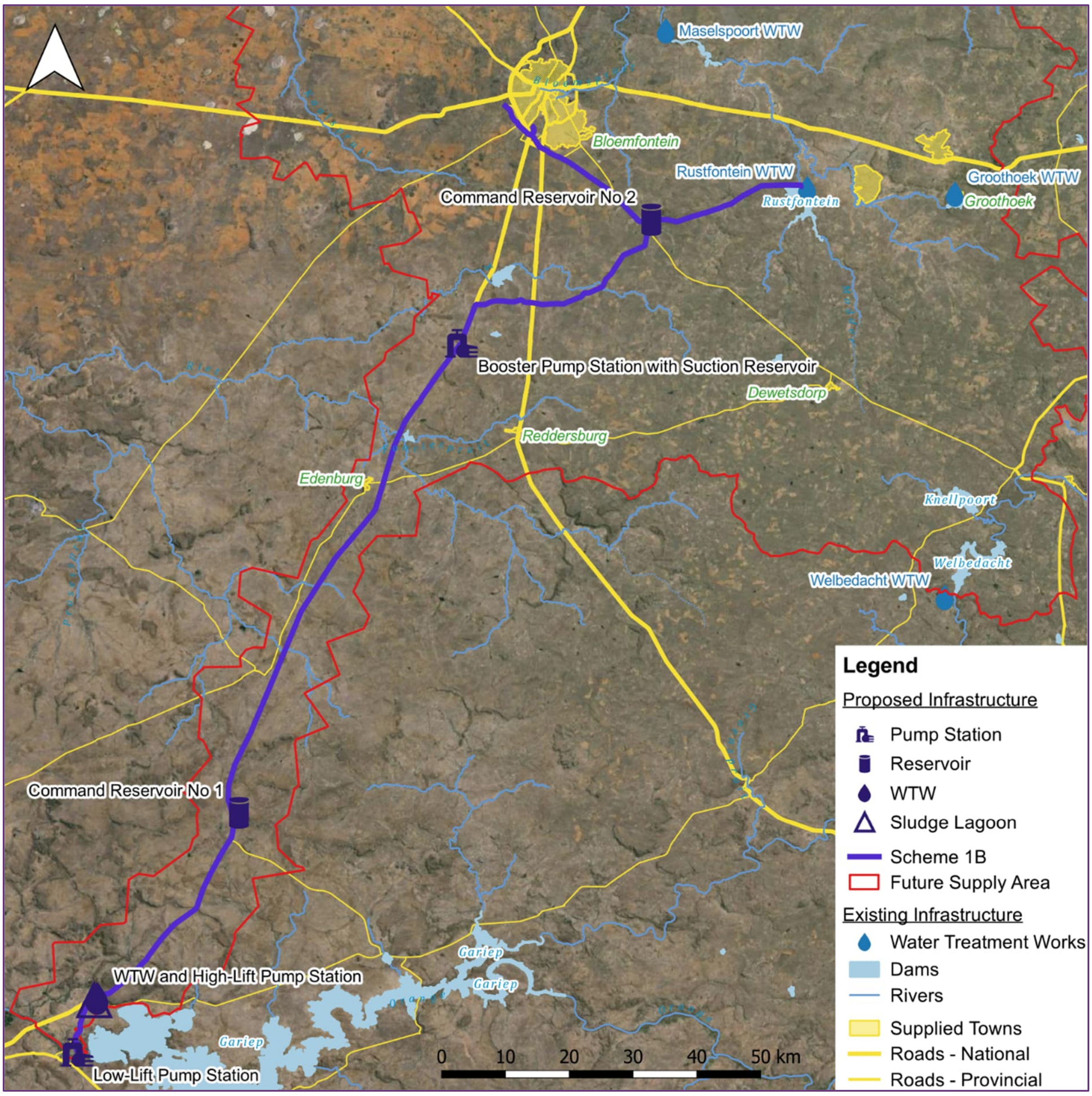

Figure E-3: Configuration of Scheme 1B1(A)

The main infrastructure components for Configuration 1B1(A) are shown in Table E-7.

A system risk failure analysis was undertaken for Schemes 1A, 1B and 4B that involved assessing the resilience of schemes by considering potential failures at supply sources or at a WTW. This evaluation entailed examining the system's ability to meet the 2050 demands of the major demand centres, Bloemfontein, Botshabelo and Thaba Nchu, in the event of a failure. The critical point of failure was found to be the supply from Rustfontein WTW, which if it fails, Schemes 1A and 4B result in the supply to Botshabelo and Thaba Nchu being reduced to 10% of the 2050 demands. Scheme 1B provides the most resilience and operational flexibility of the three schemes, as in the event of failure from any one of the four WTWs over 80% of the 2050 demands can still be supplied.

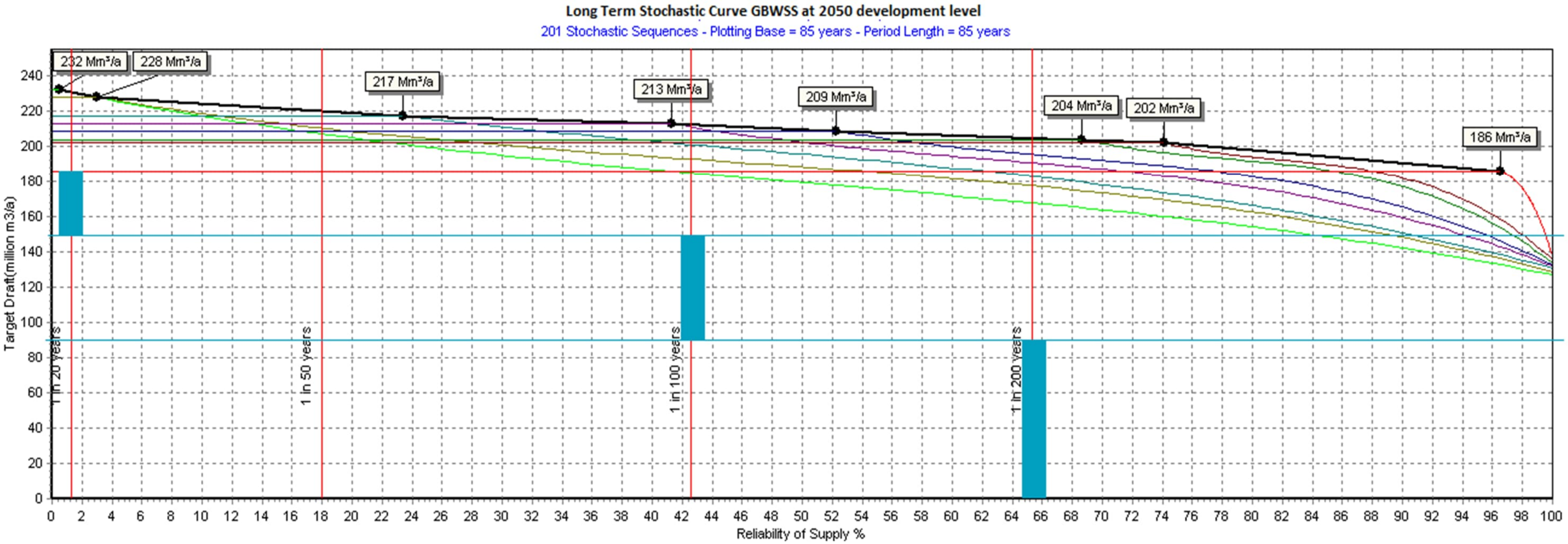

Figure E-4: Stochastic yield analysis for Scheme 1B

Table E-6: Results of financial comparison for optimization of scheme 1B (Hybrid)

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Present Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme 1B optimization	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**Mℓ**	**Mm³/annum**	**Mm³/annum**	**Mm³**	**Million Rands**	**(Million Rands)**	**(Million Rands)**	**R/m³**	**R/m³**	**%**
Scheme 1B1A [Sc5b] Potable water from Gariep Dam to Rustfontein	186.9	2 x DN1400	2776 | 363	4294 | 403	2776 | 106	4294 | 138	330	182.2	87.5	2208	21493	22965	44458	10.40	20.13	-
+ Gravity pipeline to Brandkop reservoir from command reservoir	31.4	DN1600	-	-	-	-	-	-	69.2	2208	1100	239	1339	0.11	0.61	-
+ Gravity pipeline to Rustfontein from command reservoir	24.5	DN1400	-	-	-	-	-	-	68.9	2208	648	141	789	0.06	0.36	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	-
										Total	**23241**	**26920**	**50161**	**12.19**	**22.72**	**+1.2**
Scheme 1B1B [Sc5b] Potable water from Gariep Dam to Botshabelo	186.9	2 x DN1400	2776 | 363	4294 | 403	2776 | 106	4294 | 138	330	182.2	87.5	2208	21493	22965	44458	10.40	20.13	-
+ Gravity pipeline to Brandkop reservoir from command reservoir	31.4	DN1600	-	-	-	-	-	-	69.2	2208	1100	239	1339	0.11	0.61	-
+ Gravity pipeline to Botshabelo from command reservoir	30.3	DN2000	-	-	-	-	-	-	68.9	2208	1571	342	1913	0.15	0.87	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	579 | 180						18.3	2208	0	506	506	0.23	0.23	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	
										Total	**24164**	**26230**	**50394**	**11.88**	**22.82**	**+1.7**
Scheme 1B2 [Sc5b] Potable water from Gariep Dam to Rustfontein	180.2	2 x DN1400 & DN1600	2776 | 363	4294 | 403	-	-	330	182.2	87.5	2208	22081	20695	42776	9.37	19.37	-
+ Pumped pipeline to Brandkop reservoir from command reservoir	36.4	DN1500	1489 | 49	2193 | 85	-	-	-	-	69.2	2208	1171	777	1948	0.35	0.88	-
+ Gravity pipeline to Rustfontein from command reservoir	29.3	DN1600	-	-	-	-	-	-	68.9	2208	1025	223	1248	0.10	0.57	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	
										Total	**24277**	**25271**	**49547**	**11**	**22.44**	**100**

Table E-7: Summary of main infrastructure components of Scheme 1B1(A)

Infrastructure component	Capacity / Size	Length (km)
Low-lift pump station	3,797 m^3/s @ 102 m	-
Raw water pipeline	1800 mm	10.5 km
Water treatment works	312 Mℓ/d	-
High-lift pump station	3,616 m^3/s @ 325 m	-
1st command reservoir	80 Mℓ	-
Booster pump station	3,616 m^3/s @ 124 m	-
2nd command reservoir	80 Mℓ	-
Potable pipeline from high-lift pump station to 2nd command reservoir	1800 mm	176.4 km
Potable pipeline from 2nd command reservoir to Bloemfontein	2000 mm	31.4 km
Potable pipeline from 2nd command reservoir to Rustfontein WTW	1400 mm	24.5 km

The total capital cost of Scheme 1B1(A) is estimated at R 23,24 million (excluding VAT) at pre-feasibility level of detail.

A high-level comparison between Schemes 1A, 1B and 4B were undertaken based on the following considerations:

- Socio-economic,
- Financing arrangements,
- Institutional arrangements, and,
- Environmental impacts.

It was concluded from this high-level comparison that none of these considerations will dictate the scheme to be implemented. The decision on which scheme to implement must therefore be based on strategic, financial and operational considerations.

Based on the above conclusions, it is recommended that the detailed feasibility study proceed based on Scheme 1B1(A) for the following reasons:

- Financially it is comparable to all other schemes that were considered,
- It is the only scheme that can satisfy 100% of the 2050 demands to all the demand centres located within GBWSS, and,
- It is the scheme with the greatest operational flexibility and resilience, even when failures at any of the WTWs are experienced.

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Sub-Option Hydraulic Gradelines

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Gariep WTW Site Inspection Report

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Abbreviations

Acronyms

Acronym	Description
AADD	Average Annual Daily Demand
AOA	Annual Operating Analysis
BW	Bloem Water
CBD	Central business district
CECs	Contaminants of Emerging Concern
COC	Current Operating Capacity
CT	Contact Time
DAF	Dissolved Air Flotation
DAFF	Dissolved Air Flotation and Filtration
DEM	Digital Elevation Model
DWS	Department of Water & Sanitation
EFR	Environmental flow requirements
EIA	Environmental Impact Assessment
GAADD	Gross Average Annual Daily Demand
GIS	Geographical information systems
HDPE	High-Density Polyethylene
HFY	Historic Firm Yield
HLPS	High Lift Pump Station
IAP	Interested and Affected Parties
IDP	Integrated Development Plan
ISP	Internal Strategic Perspective
IWULA	Integrated Water Use License Application
LAS	LiDAR Aerial Survey
LiDAR	Light Detection and Ranging
Lo	Longitude of origin
LLPS	Low Lift Pump Station
LOS	Level of Service
MDC	Maximum Design Capacity
MMM	Mangaung Metropolitan Municipality
NWRP **NTU**	DWA Directorate: National Water Resource Planning Nephelometric Turbidity Unit
ORS	Orange River System
PACℓ	Poly Aluminium Chloride
P/s	Pump station
PPP	Private Public Partnerships
PPPr	Public Participation Process
PRV	Pressure reducing valve

Acronym	Description
PVC	Polyvinyl Chloride
RDP	Reconstruction and Development Programme
RO	Reverse osmosis
RPST	Reconciliation Planning Support Tool
SDF	Spatial Development Plan
SMC	Study Management Committee
ToR	Terms of Reference
UAW	Unaccounted for Water
URV	Unit Reference Values
VCWB	Vaal Central Water Board
WARMS	Water Authorisation and Registration Management System
WC/WDM	Water Conservation and Water Demand Management
WMA	Water Management Area
WRC	Water Research Commission
WRYM	Water Resources Yield Model
WSDP	Water Service Development Plan
WTP	Water Treatment Plant (same meaning as “Water Treatment Works”)
WTW	Water Treatment Works (same meaning as “Water Treatment Plant”)

Measurement Units

Symbol	Description
Ha	Hectares
km	Kilometres
m	Meters
m^3/a	Cubic meters per annum
m^3/d	Cubic meters per day
m^3/s	Cubic meters per second
million m^3	Million cubic meters
mm/a	Millimetres per annum
million m^3/a	Million cubic meters per annum
Rand/a	Rand per annum

1 Introduction

1.1 Background

The Water Reconciliation Strategy Study for the Larger Bulk Water Supply Systems: Greater Bloemfontein Area (DWS, 2012) (henceforth referred to as the “2012 Reconciliation Strategy”) identified that the Greater Bloemfontein Water Supply System (GBWSS) would need to secure a sustainable water supply for the future water demands in the area. The 2012 Reconciliation Strategy recommended that the development of a major surface water augmentation scheme should be given consideration as a possible option in conjunction with the implementation of various other interventions.

Following the 2012 Reconciliation Strategy, the area experienced water shortages and the major surface water augmentation scheme option, now called the Greater Mangaung Water Augmentation Project – Xhariep Pipeline, was accelerated. Vaal Central Water Board (VCWB), previously known as Bloem Water, and Mangaung Metropolitan Municipality (MMM) independently investigated the same three route options from Gariep Dam to tie-in points within the GBWSS area (see Figure 1-1). Each institution reached a different conclusion as to which of the three was the best route/scheme.

The Xhariep Pipeline project was and remains of critical importance to address growing water demands on a regional basis; thus, the Department of Water and Sanitation (DWS, the Client) appointed Zutari to complete the pre-feasibility study, which included reviewing all previous studies, and recommending the optimal scheme from a national and regional perspective. This included determining routing and sizing to be taken forward to a detailed feasibility stage. Upon completion of the pre-feasibility stage, DWS will approve the preferred option and thereafter Zutari will carry out the detailed feasibility study, the water use license application and the environmental authorisation process.

1.2 Study Objectives

This pre-feasibility study conducted an independent investigation that built on the information collected and analysed in previous work. The objective of this study was to:

- Evaluate options for the Greater Mangaung Water Augmentation Project with Gariep Dam as the source,
- Conduct additional pre-feasibility level investigations, and,
- Select the optimal size, phasing, and configuration of the best water conveyance infrastructure option.

After DWS has approved the selected option and the pre-feasibility stage has been concluded, the study will continue to the detailed feasibility stage where the objectives will be to:

- Assess the technical, financial, economic, and environmental aspects at detailed feasibility level,
- Assess the risks and redundancy of the proposed bulk infrastructure system when operated in conjunction with the existing bulk infrastructure,
- Assess the impact of the project on existing systems including the Orange River System (ORS),
- Integration and utilisation of the available capacities in the existing infrastructure, and,
- Conduct stakeholder engagement workshops.

As this study is complex in nature, the detailed feasibility stage must also consider:

- Institutional arrangements for ownership and operation,
- Financing options,
- Affordability and bankability in line with the National Treasury guidelines,
- Opportunities for phased implementation, and,
- Stakeholder preferences.

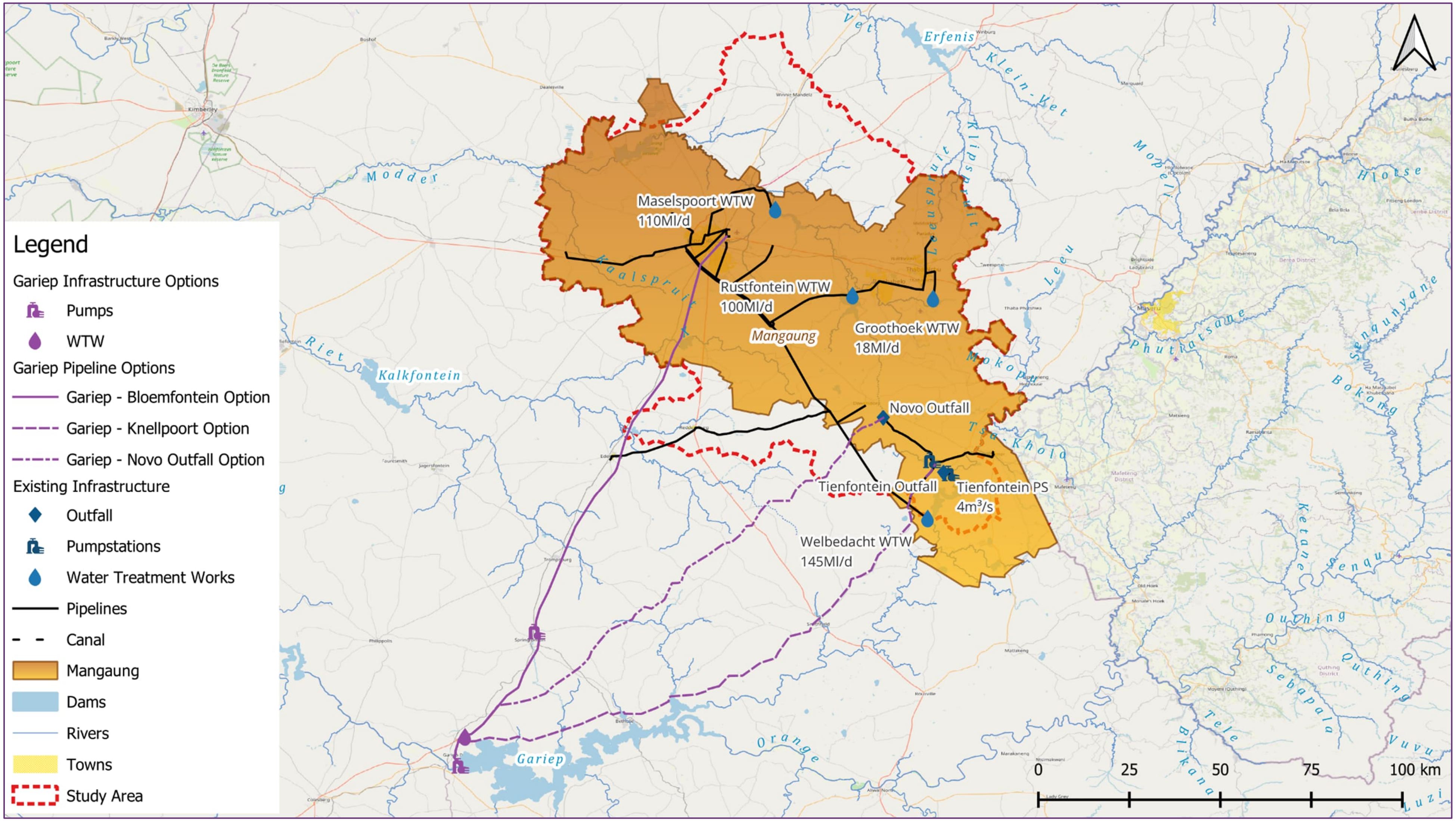


Figure 1-1: Previously studied route options from Gariep Dam to the GBWSS

1.3 Study Parameters

The following parameters were specified in the Terms of Reference (ToR).

- The maximum average annual abstraction rate from the Gariep Dam was initially limited to 60 million m^3/a.(1)(2)
- The sizing horizon for the proposed Xhariep Pipeline Project must be at least 30 years (i.e., 2050 or beyond).(3)
- Previously identified options were investigated at varying levels of detail. This study undertook a comparative analysis of these schemes at a consistent level of detail to enable fair comparison to each other. (4)

Notes:

(1) The 2012 Reconciliation Strategy mentioned a potential additional yield of 60 million m^3/a that could be abstracted from the Caledon River system. However, the Strategy does not explicitly quantify a future water demand, rather, it indicates a range that corresponds to low and high growth rate scenarios. While this provided a good indication of the annual volume to plan for, it was essential to assess its feasibility from both the receiving infrastructure and water availability perspectives.

(2) Although the ToR for this study assumed that a scheme yield of 60 million m^3/a from the ORS was available before allocation, this needed to be confirmed. It was recognised that the Orange River catchment is currently fully allocated, and any additional allocations may need concurrent implementation of permanent water efficiency measures or projects. These could have an associated cost that would need to be added to the Xhariep Pipeline scheme cost, if applicable. This is evaluated further in Chapter 3 of this report.

(3) This project adopted a planning horizon of 2050 for the following reasons:

(a) Other studies covering the same supply areas have adopted 2050 as planning horizon, which improved the reliability of the modelling results as information was available regarding proposed developments in the Caledon and Orange rivers; and,

(b) No information was available in Lesotho on proposed developments or anticipated water demands beyond 2050. Extending the planning horizon beyond 2050 would have significantly reduced the accuracy of the results. Furthermore, updating of the water demands within Lesotho, as well as obtaining further information on proposed long-term developments, were beyond the scope of this project.

(4) Three schemes/options were previously investigated as shown in Figure 1-1. The scheme investigated by MMM would supply potable water from Gariep Dam to Bloemfontein, whereas the two schemes investigated by VCWB would supply raw water to Knellpoort Dam or to the Novo Outfall Structure, the latter located in the upper reaches of the Modder River. Due to the losses associated with transporting water along the Modder River to Rustfontein Dam, a fourth scheme/option was developed as part of this study that transfers raw water from Gariep Dam directly to Rustfontein Dam. The horizontal alignment of this fourth scheme, as well as the other three schemes, are shown in Figure 1-2.

The four schemes compared in this pre-feasibility study are:

- Scheme 1: Potable water from Gariep Dam to Bloemfontein,
- Scheme 2: Raw water from Gariep Dam to Knellpoort Dam,
- Scheme 3: Raw water from Gariep Dam to Novo Outfall Structure, and,
- Scheme 4: Raw water from Gariep Dam to Rustfontein Dam.

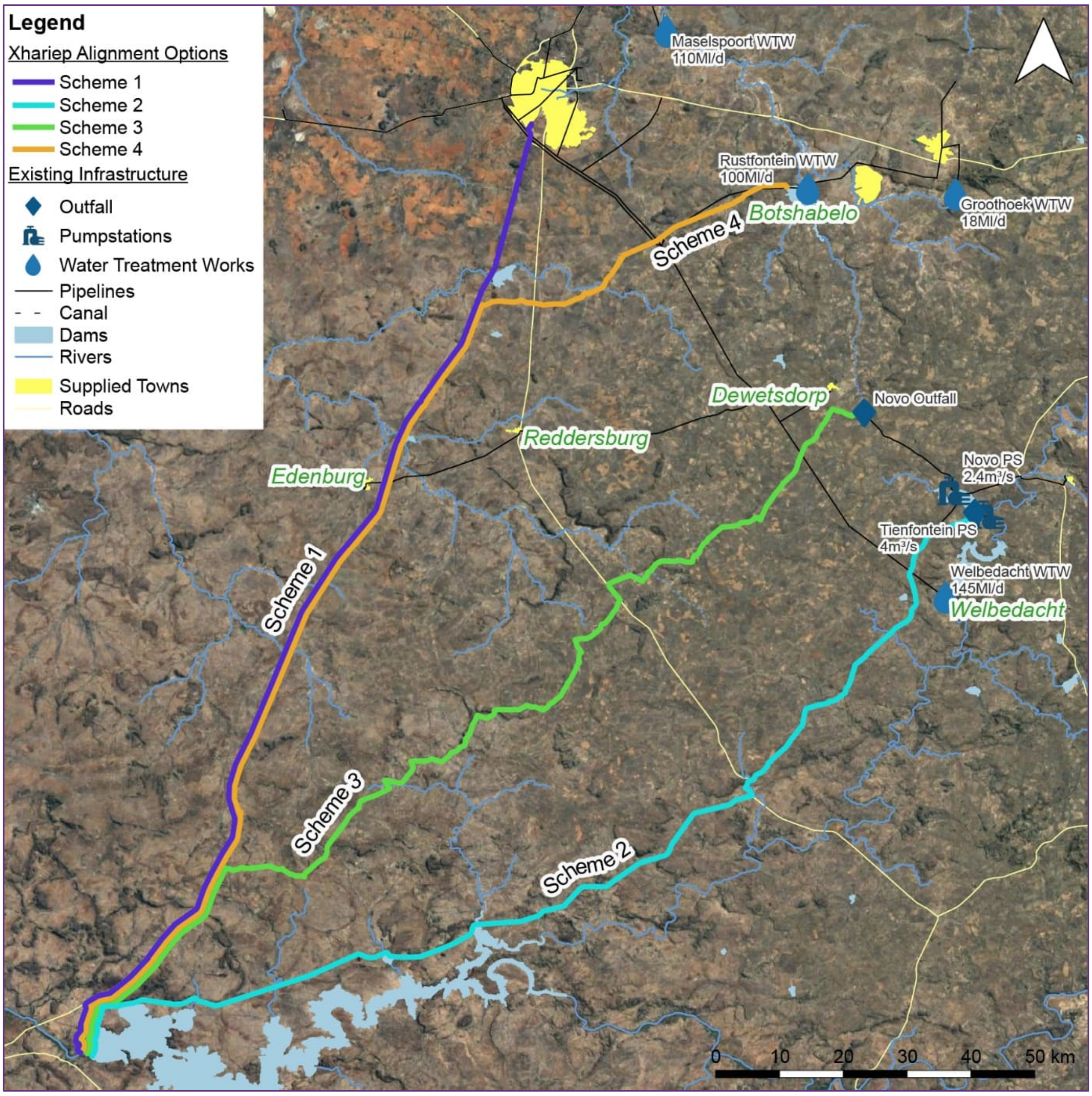


Figure 1-2: Routes of four schemes to be investigated

1.4 Study Methodology

Comparing infrastructure schemes/options is an iterative process, especially where the proposed scheme must be integrated into an existing bulk water supply system. This iterative process was necessary for this study, as the Xhariep Pipeline infrastructure needs to be integrated into the existing GBWSS infrastructure.

As noted in Section 1.3, the initial water resources yield modelling was based on transferring a volume of 60 million m^3/a. However, as this was found to be insufficient to satisfy the 2050 demands, additional yield modelling was required to determine the actual volume to be transferred. The initial and additional yield modelling results are described in Chapter 3.

Each scheme/option can have a different historic firm yield due to losses in the system based on its configuration, for example, conveying water along the Modder River will incur more losses than conveying water in a pipe. This results in schemes needing to transfer different volumes to meet the 2050 water demands. The different volumes impact the infrastructure sizing as well as the capital and operating costs.

Figure 1-3 shows a flow chart of the process followed to determine the optimal scheme to be taken forward to the detailed feasibility study phase. The following points, detailed in Sections 3.2 and 7.5 were considered in the development of the process flow chart:

- The volume to be transferred from Gariep Dam to meet the 2050 demands exceeded 60 million m^3/a, which required further water resources modelling, and,
- An alternative to the previously identified potable water and raw water schemes was required to satisfy the overall objectives of the study. This alternative is referred to as a “hybrid” option.

The following definitions were applied with respect to the schemes/options considered:

- **Scheme/Option** – this refers to the four main schemes/options being considered as shown in Figure 1-2, and,
- **Sub-Option** – for each scheme/option, multiple sub-options were developed to consider alternative locations of booster pump stations, reservoirs, and different pipeline sizes.

The historic firm yield (HFY) was determined for each scheme/option and used for comparison and selection of schemes/options. Once the preferred scheme was selected, a stochastic analysis was undertaken to verify that the 2050 demands could be met at a minimum reliability of 98% assurance of supply.

It is evident from Figure 1-3 that an iterative process was followed between the water resources analysis and infrastructure sizing. The process also included stakeholder engagement at key points in the project to present progress and findings, which provided an opportunity for stakeholders to comment on the proposed solution(s).

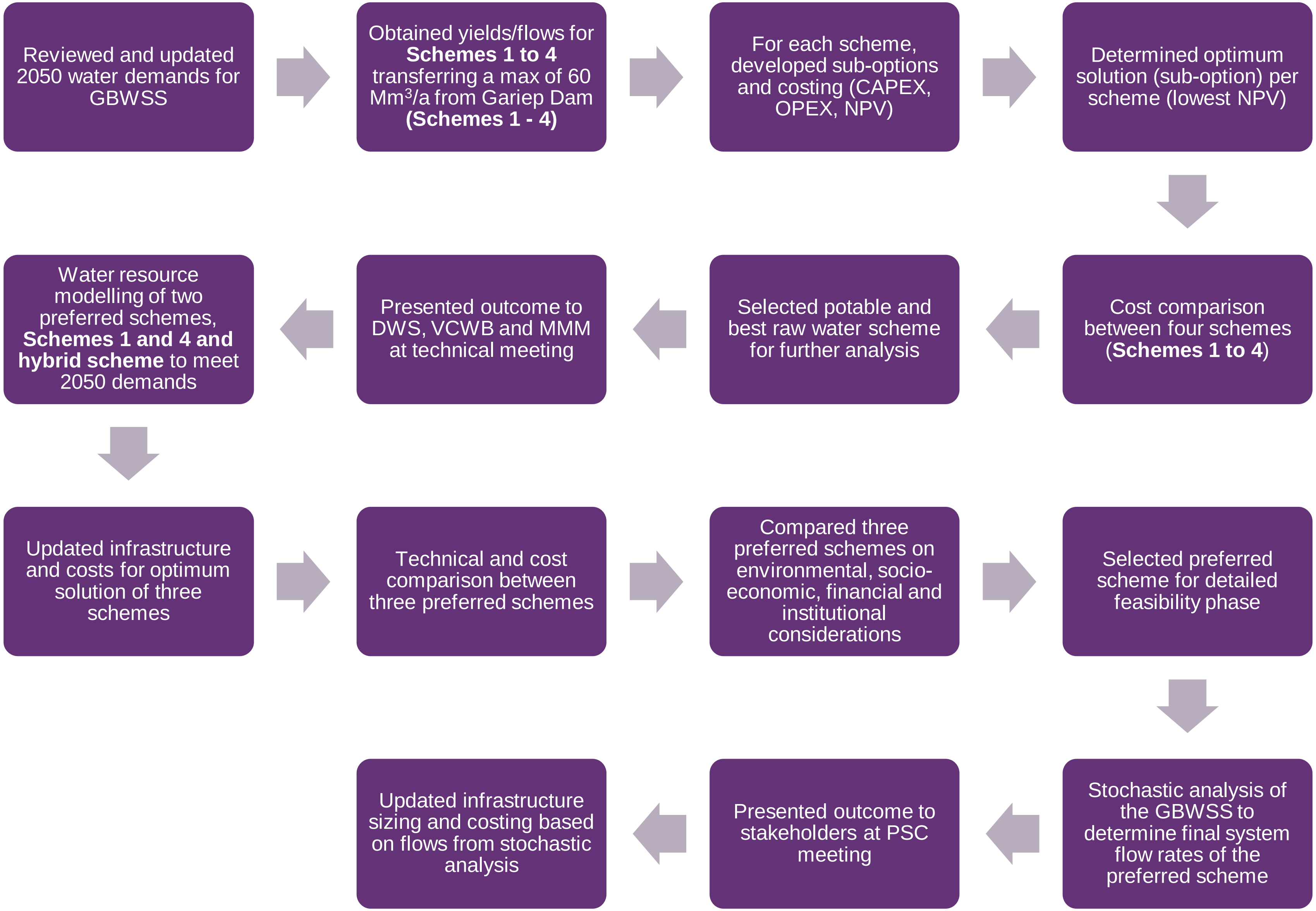


Figure 1-3: Flow chart of scheme development process

1.5 Report Structure

The purpose of this Pre-feasibility Study Report is present the findings from the pre-feasibility level investigations and comparative analysis required to enable DWS to select the preferred option that must be taken forward to the detailed feasibility phase of the project. The Pre-Feasibility Study Report is structured as follows:

Chapter 1 presents the background and objectives of the study.

Chapter 2 provides details on the status quo comprehension of the Greater Bloemfontein Water Supply System.

Chapter 3 presents the water demand projections and water resources yield analysis.

Chapter 4 details the water quality analysis undertaken for the Gariep Dam.

Chapter 5 presents the desktop level results of the geotechnical investigations.

Chapter 6 describes the available topographical survey data and details the data to be obtained during the next phase of the study.

Chapter 7 presents the pre-feasibility design, cost estimates and programming of the various infrastructure options evaluated.

Chapter 8 describes the water supply and infrastructure system risks.

Chapter 9 provides a financial comparison of infrastructure options.

Chapter 10 describes the economic, financial, and institutional evaluations undertaken in the pre-feasibility study.

Chapter 11 summarises the environmental evaluation undertaken in the pre-feasibility study.

Chapter 0 contains the conclusions and recommendations on the way forward.

2 Status Quo Comprehension of the GBWSS

2.1 Overview of the GBWSS in the ORS

The GBWSS falls within the ORS which is part of the larger Orange/Senqu River system that spans Lesotho, South Africa and Namibia. The GBWSS abstracts water from the Caledon and Modder Rivers. The Caledon River is a tributary of the Orange River. The Orange River Project (ORP), consisting of the Gariep and Vanderkloof dams, is located downstream of the GBWSS. The operation and abstraction of the GBWSS therefore has a direct impact on the ORP which in turn is impacted by upstream developments within Lesotho and the upper catchment of the Orange River in South Africa.

Figure 2-1 shows a schematic of the GBWSS, the ORP and the Orange/Senqu River system. The existing dams are shown in blue, with Welbedacht, Gariep and Vanderkloof dams located on the South African side of the Orange/Senqu River system. Metolong, Mohale and Katse dams are the existing dams located within Lesotho, that influence the system. Future dams are shown in yellow. The only dam currently under construction is the Polihali Dam shown in green, located in Lesotho.

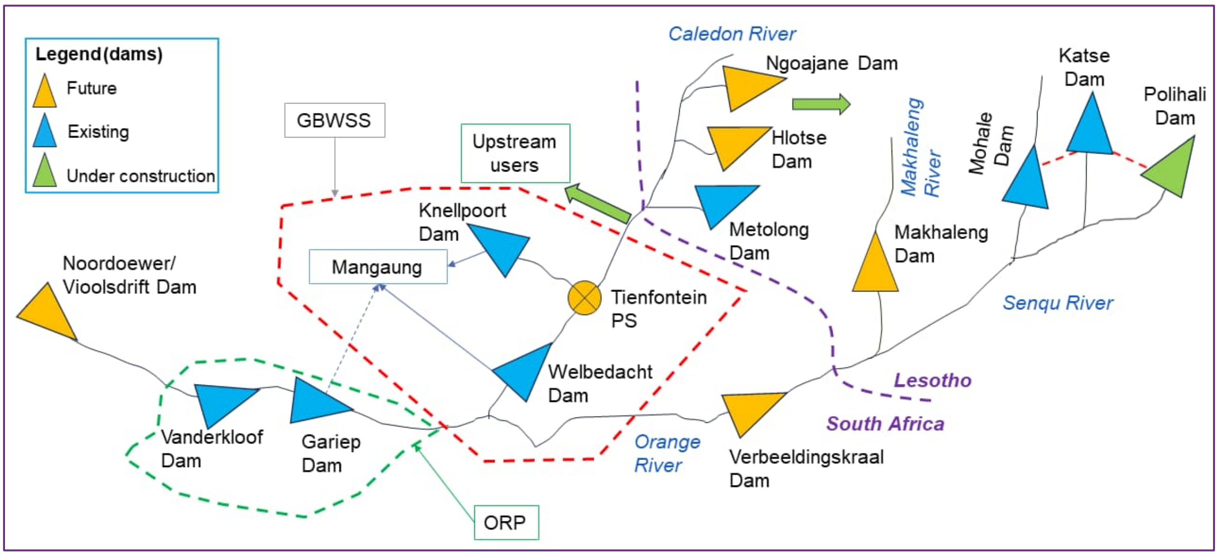


Figure 2-1: Overview of the Orange/Senqu River, ORP and GBWSS in 2050

The GBWSS also encompasses the Modder River, however, this does not form part of the ORP and is a tributary that flows into the Vaal River, before the Vaal River joins the Orange River. The configuration of how the Caledon and Modder rivers and Welbedacht Dam forms the GBWSS is described in the next section.

2.2 Existing Configuration of the GBWSS

The existing configuration of the GBWSS is shown in Figure 2-2, while a detailed description of the GWWSS infrastructure can be found in the Data Collection, Review & Analysis Report (Report No. P WMA 06/D00/00/3423/4), which forms part of the published reports for this study. The Welbedacht-Bloemfontein scheme (shown in orange) supplies De Hoek, Wepener, De Wetsdorp, Edenburg, Reddersburg, Uitkyk and Bloemfontein. The Rustfontein-Thaba Nchu scheme (light green) supplies Botshabelo and Thaba Nchu as well as smaller towns (e.g. Tabane, Blydskap, Motlatla and Houtnek). This scheme has the capability to supplement the Welbedacht-Bloemfontein pipeline. The smallest supply scheme is from Maselspoort WTW to Bloemfontein (dark green). The Rustfontein and Mockes Dams can be supplemented by the Tienfontein-Knellpoort-Novo transfer (dashed blue).

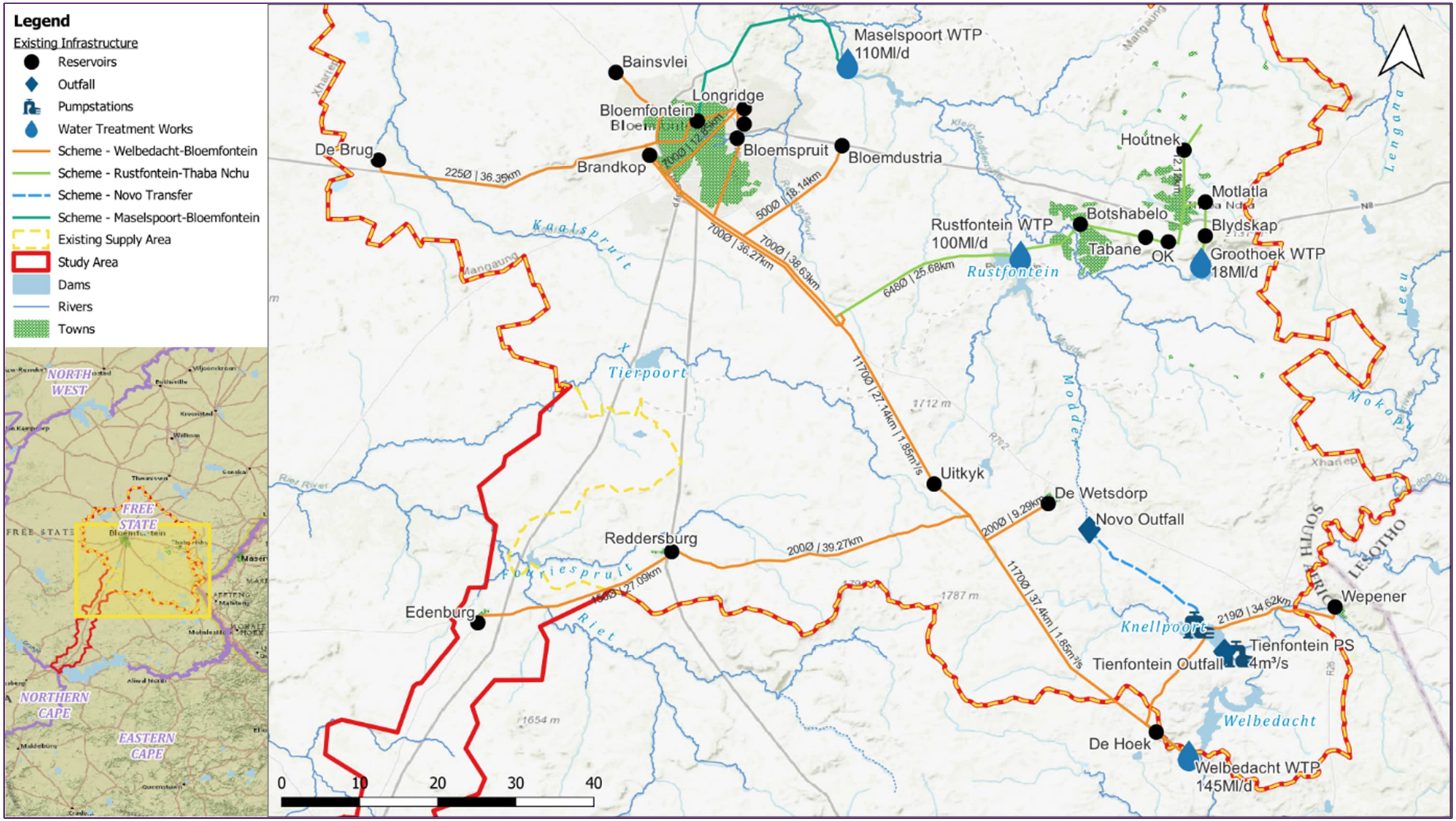


Figure 2-2: Existing infrastructure making up GBWSS

The total storage capacity of the system is 181.5 million m^3. The installed capacity of Tienfontein Pump Station is 5.7 m^3/s however the maximum operational capacity is limited to 3.8 m^3/s. The Novo Pump Station has an installed capacity of 2.95 m^3/s and a maximum operational capacity of 2.2 m^3/s. The system has a total treatment capacity of 355 Mℓ/d (excluding the Groothoek water treatment works (WTW) but distribution of potable water is uneven due to WTW location and capacity.

2.3 Existing Challenges

There are several existing challenges that the GBWSS faces, chief among them being that restrictions were imposed since 2016 due to an inability to meet demands. Botshabelo and Thaba Nchu face the brunt of the restrictions as they are dependent on supply from Rustfontein and Groothoek WTWs, the latter of which has limited capacity.

Siltation at Welbedacht Dam remains an ongoing problem for VCWB, and attempts to clear silt upstream of the dam wall have been unsuccessful.

Though the Welbedacht WTW is not designed to cope with the high siltation loads experienced in high rainfall seasons, the Welbedacht WTW is reasonably well operated. There are, however, issues regarding timely maintenance which result in challenges for the operations team to produce fully compliant potable water on a continuous basis. This is also true of the Rustfontein WTW which additionally faces challenges with operation of the backwash pumps and air scour valves in the filters, as well as a deterioration in water quality from its natural catchment.

2.4 Ongoing and Planned Future Upgrades

The 2012 Reconciliation Strategy listed several options to augment the GBWSS water supply, one of which was to secure the Welbedacht pipeline which experiences frequent bursts. VCWB initiated Phase 1, a 33.7km section of the pipeline which has been re-laid and the construction is nearing completion. Phase 2, replacement of the next 71.3km of pipeline, has yet to start.

On account of the 2012 Reconciliation Strategy's recommendation, the Novo and Tienfontein Pump Stations pumping capacities were upgraded in 2016 to 3.8 m^3/s and 2.2 m^3/s respectively by DWS.

A feasibility study for a bidirectional or parallel pipeline/s between Knellpoort Dam and Welbedacht WTW was commissioned by VCWB and completed in 2019. The project has not progressed beyond the initial study.

MMM recently commenced with the upgrading of the Maselspoort WTW, which will increase the treatment capacity from 110Mℓ/d to 120Mℓ/d.

VCWB is also planning to increase the capacity of Rustfontein WTW by an additional 50Mℓ/d to a total capacity of 150Mℓ/d.

Neither the progress of interventions as recommended by the 2012 Reconciliation Strategy for implementation by MMM or VCWB, nor additional planned upgrades on the MMM distribution network, was made available to the study team.

A written request to share planning information, including draft documents, was submitted to MMM and VCWB. At the time of drafting the report, no response has been received from either party.

3 Water Demand and Resource Analysis

3.1 Water Demand Analysis

3.1.1 Greater Bloemfontein Water Supply System

The GBWSS supplies the majority of the potable water demand of Bloemfontein, Thaba Nchu and Botshabelo, as well as the small towns of Wepener, Dewetsdorp, Reddersburg, Edenburg and Excelsior. Most of the small towns use their own groundwater resources in combination with surface water support from the GBWSS. The small towns currently make up approximately 4% of the water demand of the GBWSS. VCWB is the main supplier of bulk potable water, and MMM supplies the remainder of Bloemfontein's water demands via the Maselspoort Scheme.

The Xhariep Pipeline has the potential to supply several small towns along the pipeline route. These small towns include Gariep, Springfontein, Trompsburg and Bethanie. In addition, and subject to their current security of supply, it might also be possible to supply other towns that are located further away from the pipeline such as Colesberg, Bethulie, Philippolis, Jagersfontein and Fauresmith. Figure 3-1 shows the location of these towns relative to the proposed bulk water infrastructure for Scheme 1B (discussed in Section 7.5).

To determine the yield to be transferred from Gariep Dam, it was necessary to review the water demands for the GBWSS and to update the demand projections until the year 2050. The following approach was followed with respect to the demand projections:

- A review was undertaken of demand projections presented in previous studies,
- A review of the historical growth in water demand was undertaken to compare projected growth against actual growth in demand,
- Future demand projections per town were calculated based on the latest available population data and taking cognisance of anticipated improvements in the level of service over time,
- The demand projections from the previous studies were compared with the calculated demand projections, and,
- The impact of including towns located within 50km from the Xhariep Pipeline infrastructure, as shown in Figure 3-1, was evaluated.

3.1.2 Demand Projections from Previous Studies

Previous studies considered future water demand up to the year 2035. The objective of this task was to review the water demand requirements and to extend the forecast to the year 2050. There are various water demand projections available for the GBWSS.

The following previous studies were used as reference information:

- Water Reconciliation Strategy Study for the Large Bulk Water Supply Systems: Greater Bloemfontein Area completed in June 2012 (2012 Reconciliation Strategy), commissioned by DWS,
- Assessment of Potential Bulk Water Supply Schemes, where a draft report was completed in March 2015, and Gariep Dam to Bloemfontein Bulk Water Augmentation Scheme: Technical Feasibility Study completed in May 2015 (2015 Technical Feasibility Study), commissioned by MMM,
- Mangaung Gariep Water Augmentation Project: Scenario Analyses Greater Bloemfontein completed in August 2018 (2018 Scenario Analysis), commissioned by MMM. This study used the demands from the 2015 Technical Feasibility Study, and,

- The 2022/23 Annual Operating Analyses (AOA) where projections were based on the 2018 Mangaung Study (2022/23 AOA).

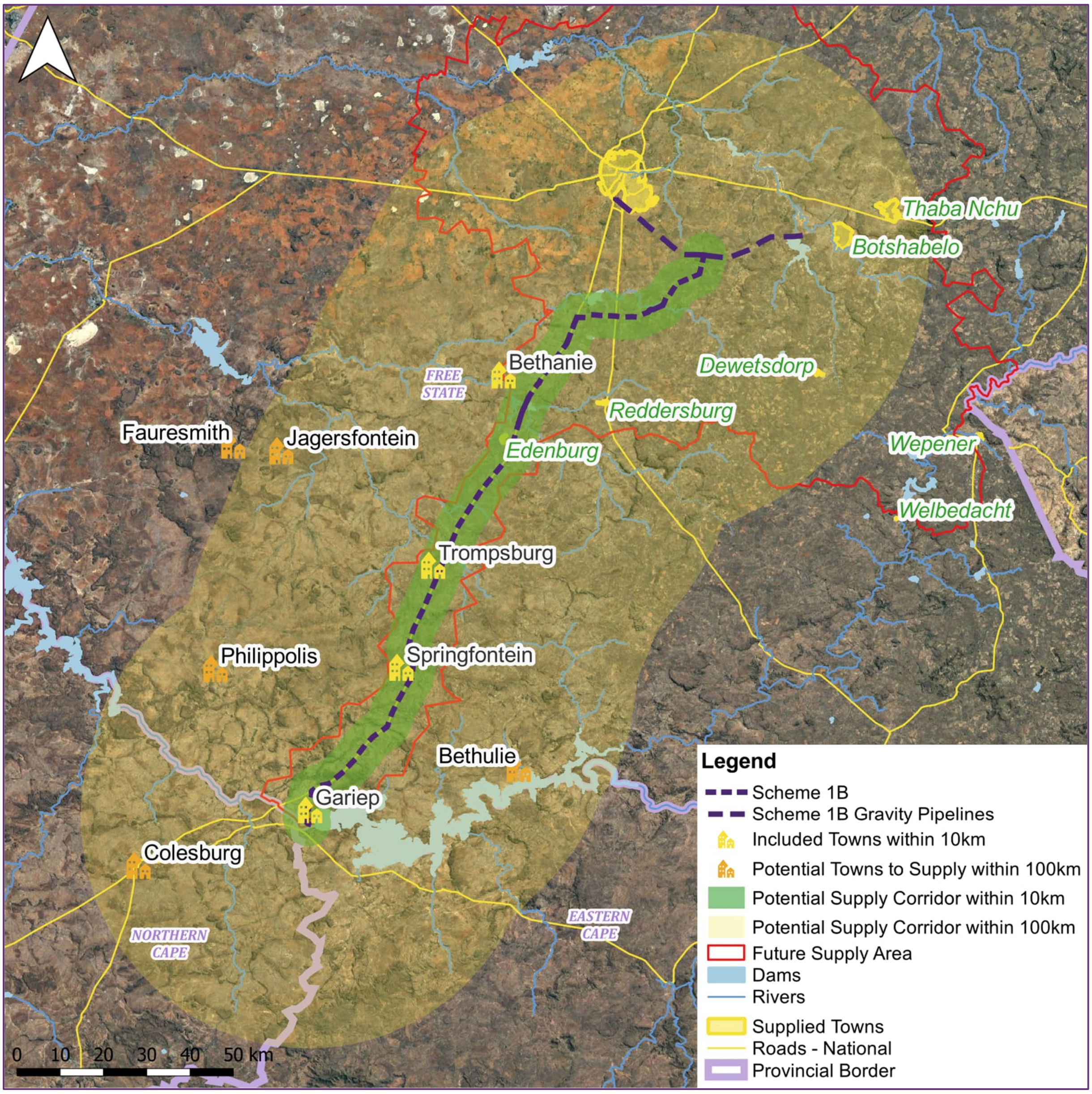


Figure 3-1: Towns and villages located close to proposed Scheme 1B bulk infrastructure

3.1.2.1 2012 Reconciliation Strategy

The 2012 Reconciliation Strategy made the following assumptions for the development of future water demands scenarios from the GBWSS using a 2035 planning horizon:

- High growth water demand scenario will take place on account of high population growth rate and high economic growth rate. A 3% growth rate per annum was assumed as the basis for this scenario.
- Low growth water demand scenario will take place on account of low population growth rate and low economic growth rate. A 1% growth rate per annum was assumed as the basis for this scenario.

The 2012 Reconciliation Strategy used 12% bulk conveyance losses to determine the gross average annual daily demand (GAADD). The future water demand projections for this study can be seen in Figure 3-2.

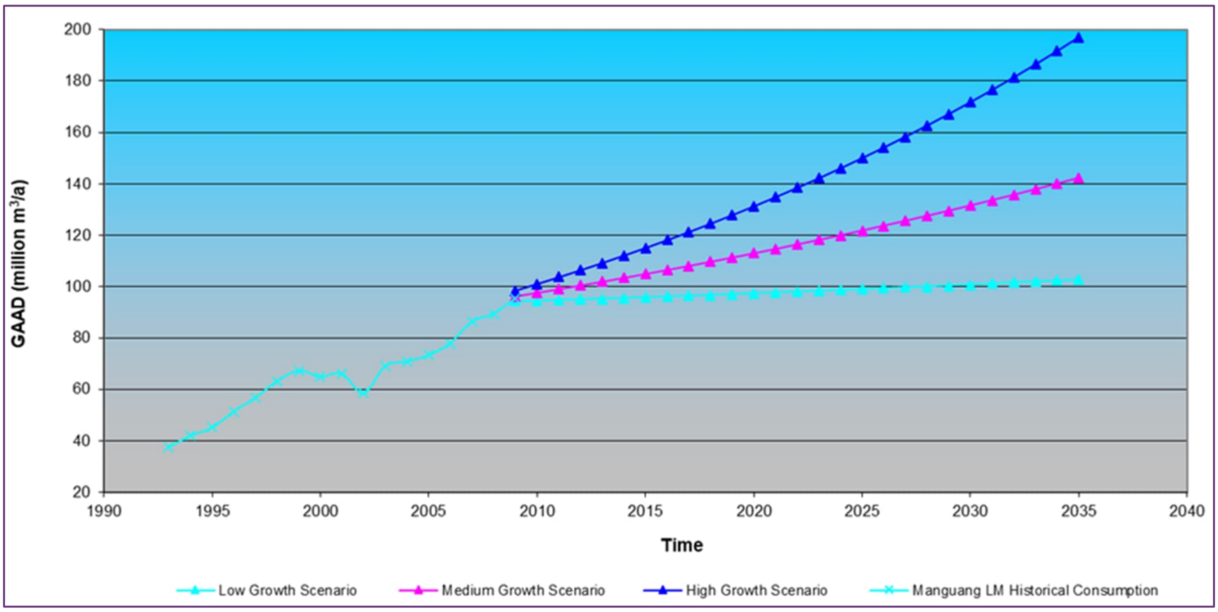


Figure 3-2: Water demand scenarios from the 2012 Reconciliation Strategy

3.1.2.2 2015 Technical Feasibility Study

The 2015 Technical Feasibility Study adopted 2035 as a design horizon and two water demand growth scenarios were considered, namely:

- A 1% growth scenario; and,
- A 2% growth scenario.

The demand projections assumed extensive implementation of water conservation and water demand management (WC/WDM) measures and expected water demands to exceed the 2% growth rate if these measures were not implemented. The impact of new housing developments, the ventilated improved pit toilet eradication programme and water re-use was considered as part of the demand forecasts.

It is uncertain what percentage was used for the bulk conveyance losses since different losses are reported in the draft Assessment of Potential Bulk Water Supply Schemes Report completed in March 2015 and in the Technical Feasibility Study completed in May 2015. It was assumed that a 3% loss was used as stated in the 2015 Technical Feasibility Study. The future water demand projections for this study are shown in Figure 3-3.

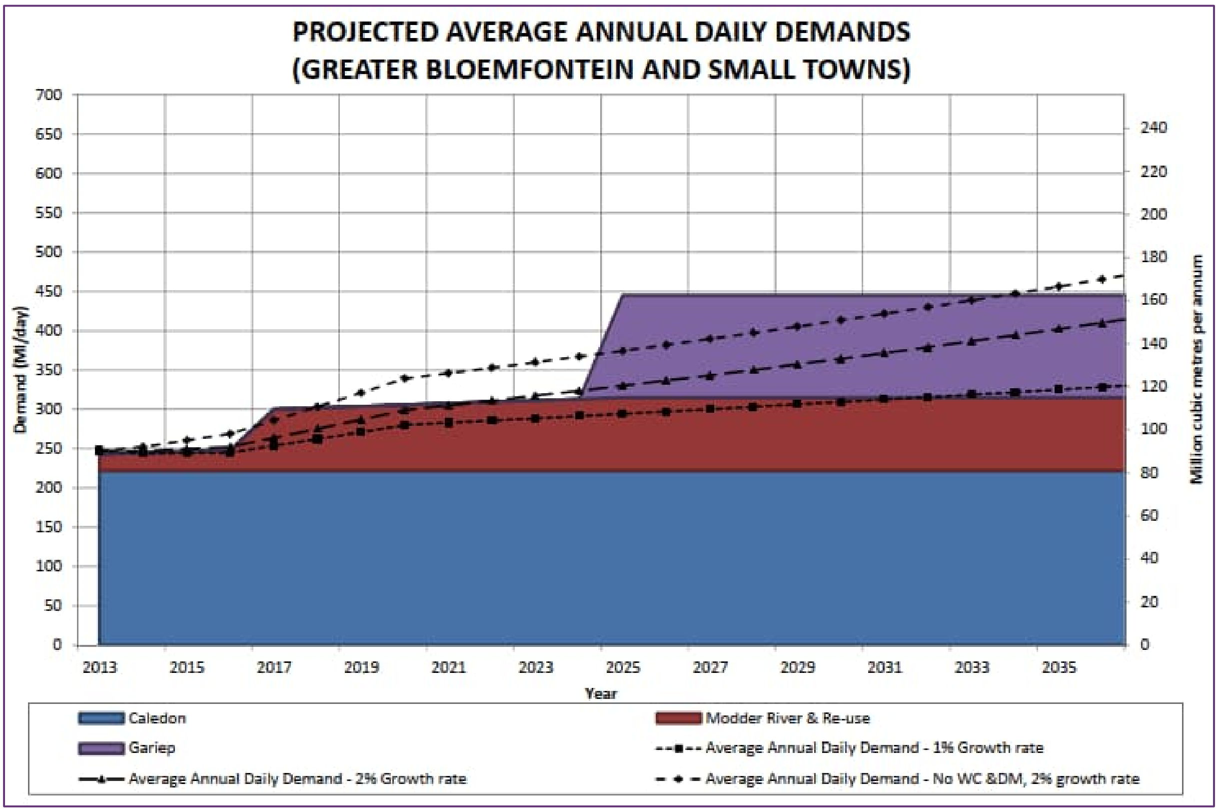


Figure 3-3: Water demand scenarios from 2015 Technical Feasibility Study

3.1.2.3 2018 Scenario Analysis Study

The 2015 Technical Feasibility Study demand projections were used for the purpose of this study. These demands were subsequently used in the 2022/23 Annual Operating Analysis Study (DWS, 2022) and are shown in Table 3-1.

Table 3-1: GBWSS potable water demands for 2023, 2035 and 2050 (DWS, 2022)

Demand centre	2023			2035			2050		
	Mil m^3/a	Mℓ/d	m^3/s	Mil m^3/a	Mℓ/d	m^3/s	Mil m^3/a	Mℓ/d	m^3/s
Bloemfontein	71.86	196.74	2.277	88.08	241.16	2.791	109.90	300.90	3.483
Botshabelo	19.20	52.58	0.609	27.38	74.96	0.868	40.99	112.21	1.299
Thaba Nchu	13.10	35.87	0.415	18.68	51.14	0.592	27.96	76.55	0.886
Total	**104.16**	**285.18**	**3.301**	**134.14**	**367.26**	**4.251**	**178.85**	**489.66**	**5.667**
Small towns	**4.39**	**12.02**	**0.139**	**5.21**	**14.26**	**0.165**	**6.39**	**17.48**	**0.202**
Total system	**108.55**	**297.20**	**3.440**	**139.35**	**381.52**	**4.416**	**185.23**	**507.14**	**5.870**

3.1.2.4 2022/23 Annual Operating Analysis

The 2022/23 AOA based its projections on the 2018 Mangaung Study (refer to Table 3-1).

3.1.3 Historic Water Demands

Data for actual water use by the MMM was extracted from previous studies discussed above and more recent data was obtained from the 2022/23 Annual Operating Analysis undertaken by WRP Consulting.

Historic data included water purchased by MMM from VCWB and water supplied from the Maselspoort system. The following historic data was captured:

- Average annual daily demands (AADD) for Bloemfontein, Botshabelo, Thaba Nchu and Dewetsdorp was available from 1997 to 2013,
- AADD for all the individual small towns, were only available for the years 2006 to 2011, and
- Total AADD data for the years 2010 to 2023 was available.

Figure 3-4 shows the GAADD for the years 2006 to 2023. The drop in demand during 2010 and 2011 was likely due to the significant summer rains experienced in 2011. From 2014 onwards restrictions were applied, which explains the drop in demand after 2014.

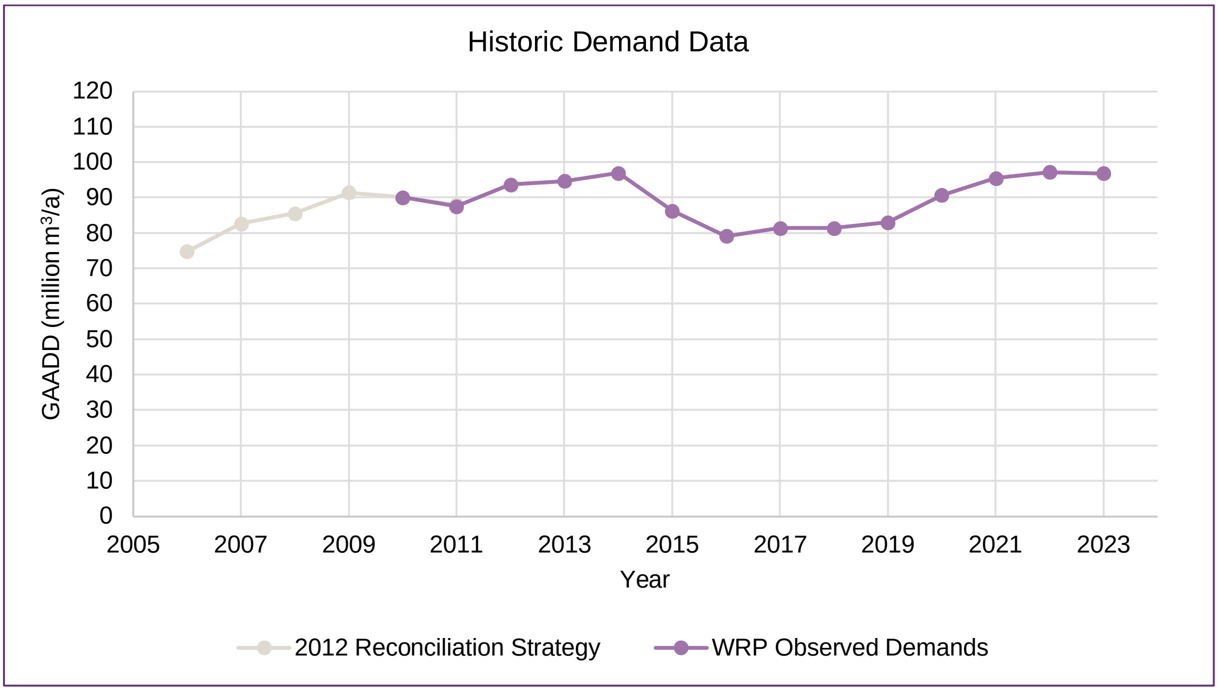


Figure 3-4: Total historic water demands

Figure 3-5, Figure 3-6 and Figure 3-7, shows the GAADD for Bloemfontein, Botshabelo and Thaba Nchu, and for the Small Towns, respectively, based on the historic data.

GAADD potable demand data was available for Bloemfontein for 16 years. A trendline was plotted to determine the average water demand growth rate. It was found to have an average demand growth rate of 1.81%. It was reported in the 2012 Reconciliation Strategy that 2010 and 2011 were significant wet years and therefore lower water demands were experienced. If 2010 and 2011 water demands were excluded and the average water demand growth rate calculated, a 1.88% growth rate was found.

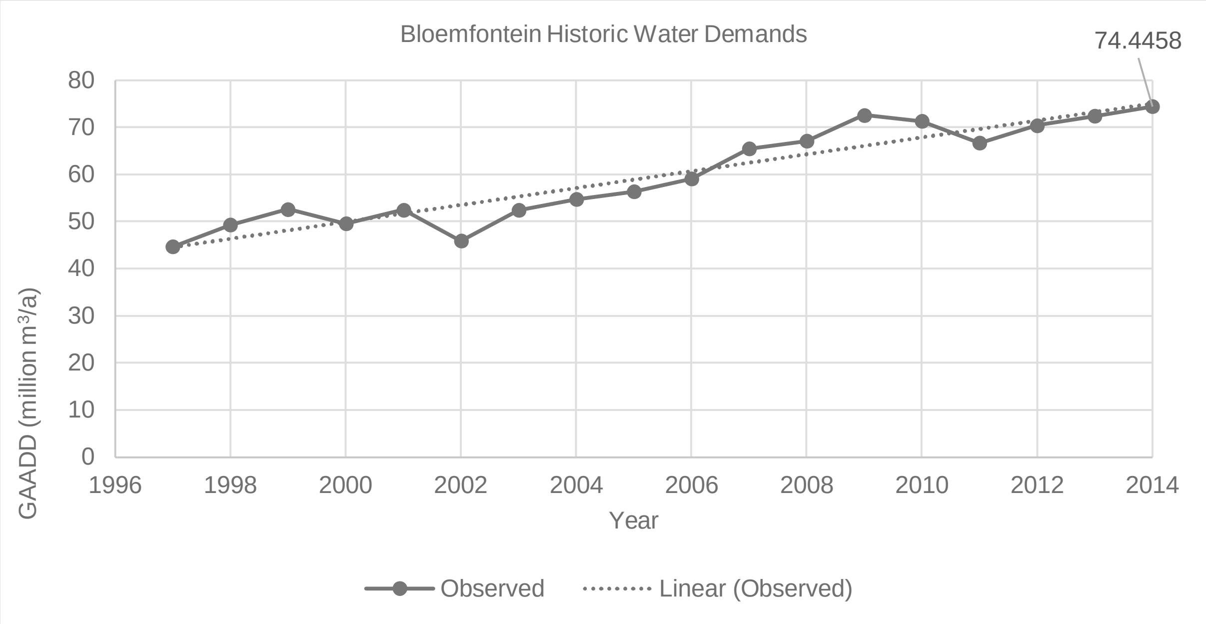


Figure 3-5: Bloemfontein historic water demands (including 2010 and 2011 demands)

GAADD potable demand data for Botshabelo and Thaba Nchu was available for 16 years. A trendline was plotted to determine the average water demand growth rate. The towns were found to have an average demand growth rate of 2.34%.

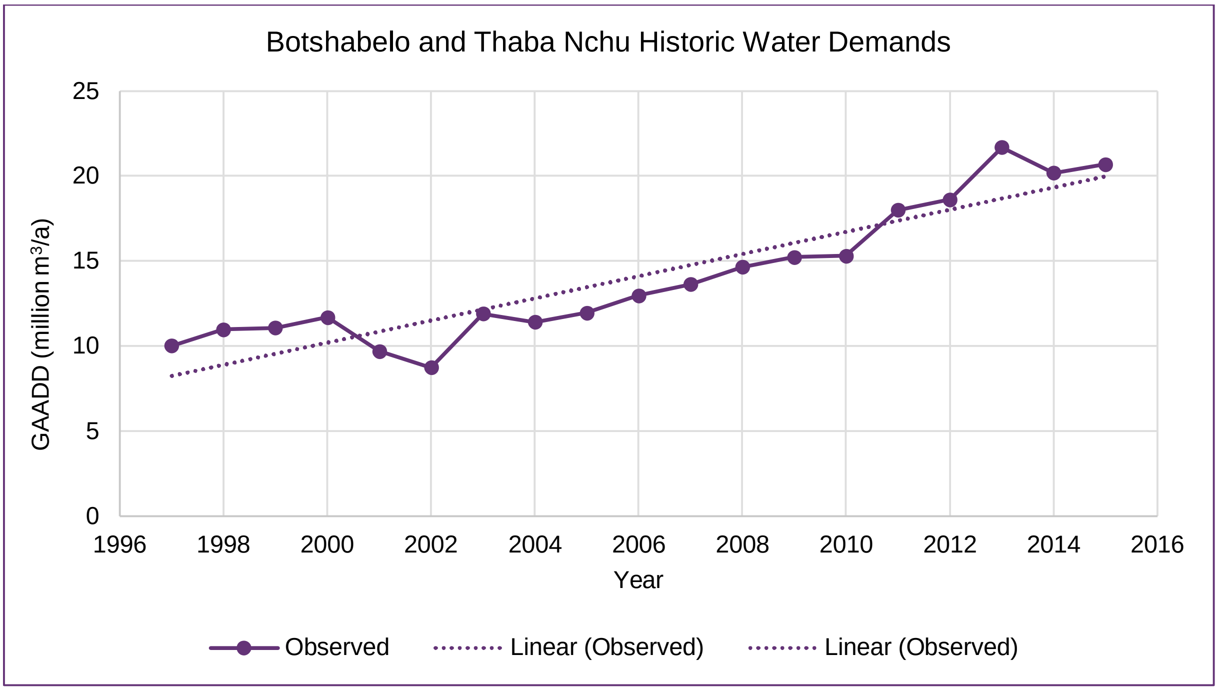


Figure 3-6: Botshabelo and Thaba Nchu historic water demands

GAADD potable demand data for the Small Towns were only available for 6 years. This period was considered too short to reach any meaningful conclusion on the actual growth in water demand. Notwithstanding, a trendline was plotted to determine the average water demand growth rate. It was found to have an average demand growth rate of 1.52%. It should, however, be noted that the potable demands declined from 2008 to 2011.

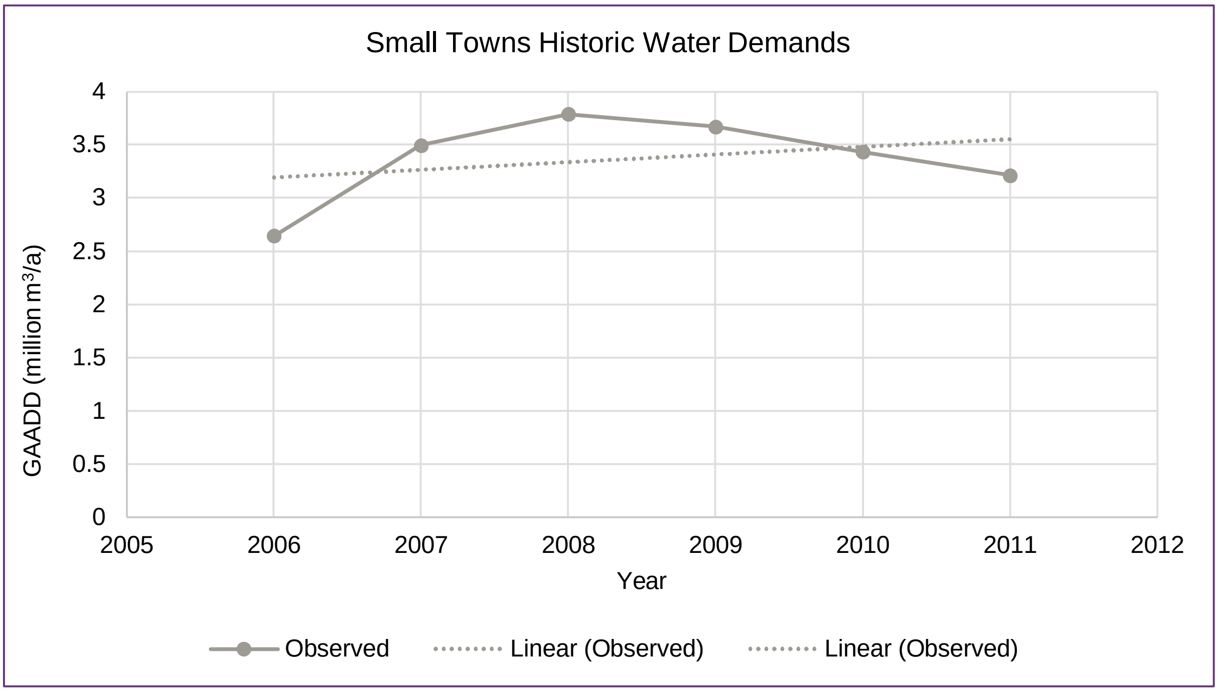


Figure 3-7: Small towns historic water demands

3.1.4 Future Water Demands

This study based the projections of the future water demands on the following factors:

- Population growth, and,
- Level of service.

In 2014 water restrictions were applied to the MMM and therefore total water demands dropped. For this reason, the future water demand projections for this study used 2014 as a base year. Historic demands were provided as average annual demands (AAD). The average annual daily demands (AADD) were calculated from the AAD, whereafter a 10% bulk conveyance loss was applied to the AADD to determine the GAADD. In cases where the 2014 demands were not available for a specific town, the baseline was calculated as a portion of the total demand observed for the study area in 2014, based on water demand records from previous years. The 2014 base water demands are reflected in Table 3-2.

Table 3-2: 2014 Base water demands

2014	AAD (Mℓ/a)	AADD (Mℓ/d)	GAADD (Mℓ/d)
Bloemfontein	67678	185.42	203.96
Botshabelo	10970	30.05	33.06
Thaba Nchu	7380	20.22	22.24
Excelsior	160	0.44	0.48
Wepener	835	2.29	2.52
Dewetsdorp	969	2.65	2.92
Reddersburg	930	2.55	2.80
Edenburg	582	1.60	1.75

3.1.4.1 Population Growth

Population growth is one of the main drivers for predicting future water demands. Various sources were consulted to determine the population growth rate and it was found that these sources vary greatly in their predictions. Some if the sources consulted are listed below:

- District Municipality (DM) Integrated Development Plan (IDP),
- Statistics South Africa (Stats SA),
- United Nations World Population Prospects,
- The 2012 Reconciliation Strategy, and,
- The 2015 Technical Feasibility Study.

The population Census data published by Stats SA, was used as the main population data source, the reason being that Stats SA produces accurate and official data and it therefore remains a credible source. At the time of writing this report, the 2022 Census data was not available at a lower sub-place level. However, population counts were provided for the relevant district municipalities. The demand centres concerned all fell within the following district and metropolitan municipalities:

- Mangaung Metropolitan Municipality,
- Thabo Mofutsanyana DM, and,
- Xhariep DM.

The demand centres, concerned with this study, that fall within the MMM, are Bloemfontein, Botshabelo and Thaba Nchu. The population for MMM, for the years where a census or community survey was conducted, is reflected in Table 3-3. Since the population data for Bloemfontein, Botshabelo and Thaba Nchu was not available for 2022, historical distributions were used to determine the respective populations for 2022. In 2022, Bloemfontein represented approximately 65.08% of the population, Botshabelo 23.93% and Thaba Nchu 9.15%.

Table 3-3: MMM Population

Population	1996	2001	2011	2016	2022
MMM	630,784	675,281	747,431	787,803	811,431
Bloemfontein	329,733	352,993	465,447	496,316	519,316
Botshabelo	164,234	175,819	181,712	189,073	197,271
Thaba Nchu	62,836	67,269	70,118	70,902	76,122

To determine the population growth for the respective MMM demand centres, the population count indicated in Table 3-3, was plotted graphically and a trendline drawn as shown in Figure 3-8. The corresponding population growth rate, as shown in Table 3-4, was calculated using the linear trend equation.

The Small Towns of Wepener, Dewetsdorp, Reddersburg and Edenburg, are located within the Xhariep DM with the exception of Excelsior which falls within Thabo Mofutsanyana DM. Very limited population data was found for these individual towns and therefore the DM population counts were considered and applied. The population according to Census 2011, 2016 and 2022 conducted by Stats SA, for Xhariep and Thabo Mofutsanyana DM is illustrated in Figure 3-9. The corresponding population growth rates are reflected in Table 3-4.

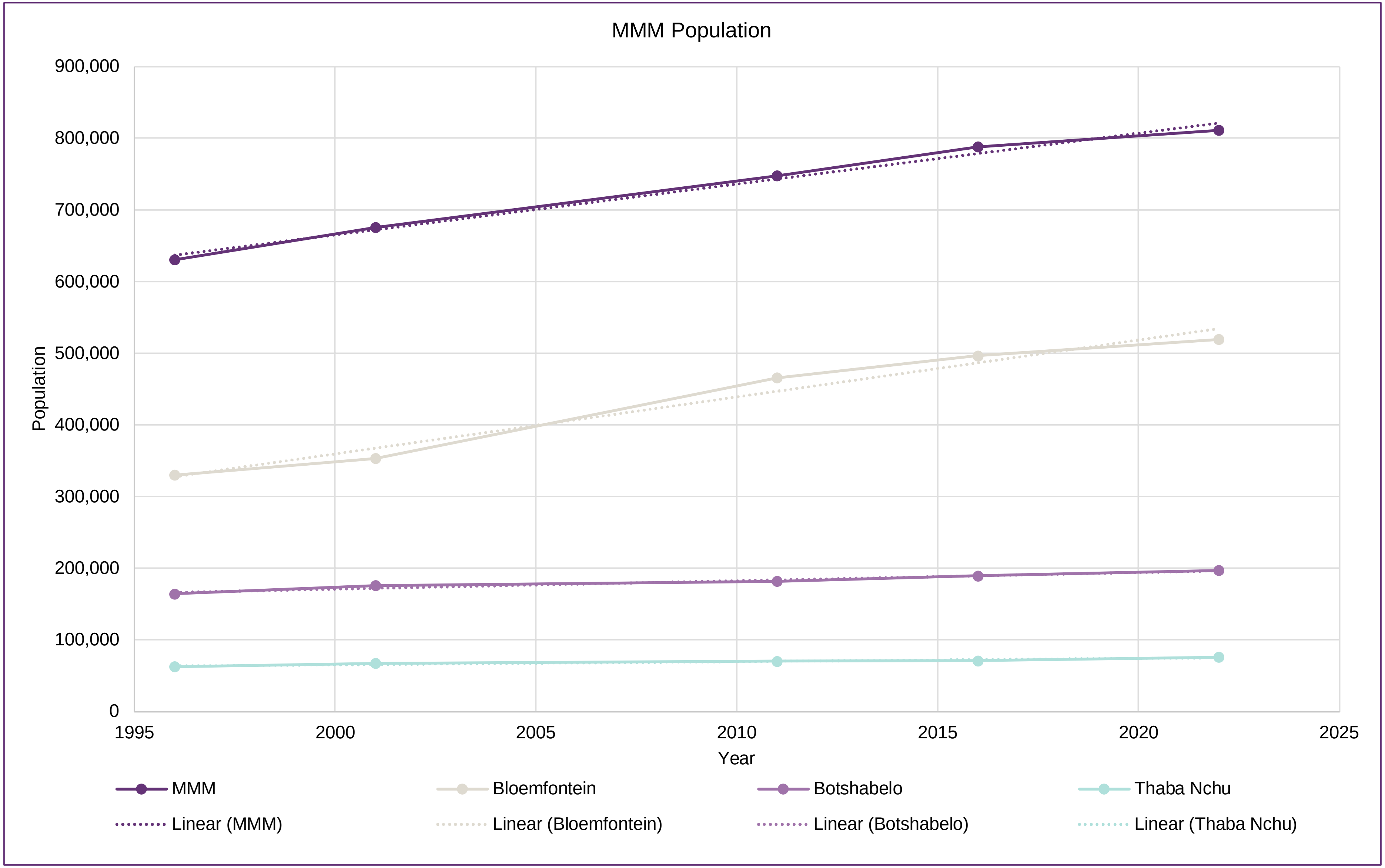


Figure 3-8: MMM Population

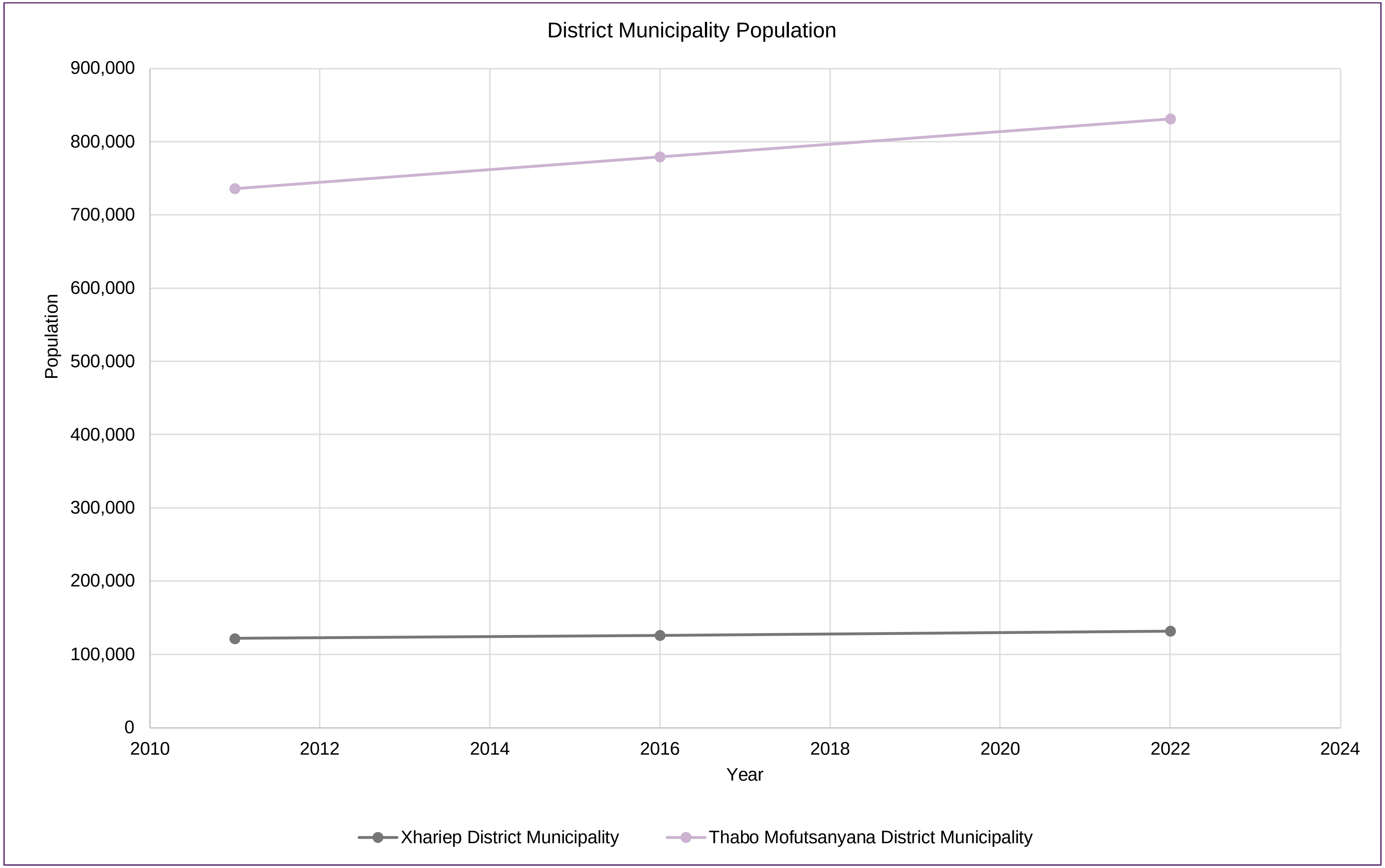


Figure 3-9: Xhariep and Thabo Mofutsanyana DM

Table 3-4 shows a comparison between the expected population growth rate and the historic growth in water demand.

Table 3-4: Population growth rates

Demand Centre	Population Growth Rate (% per annum)	Historic Growth in Water Demand (% per annum)
Bloemfontein	1.56%	1.81%
Botshabelo	0.59%	2.34%
Thaba Nchu	0.59%	2.34%
Excelsior	1.12%	1.52%
Wepener, Dewetsdorp, Reddersberg and Edenburg	0.72%	1.52%

It is clear from Table 3-4 that the historic growth of water demands was higher than the relatively low expected population growth rate. The water demands calculated using the population growth rate in this study were compared against the water demands calculated based on the historic water demand growth rate in Section 3.1.7.

The representative populations for 2011, 2023, 2035 and 2050 is summarised in Table 3-5 and illustrated in Figure 3-10 and Figure 3-11.

Table 3-5: Population projections for each demand centre

Population for each Demand Centre	Annual Growth Rate	2011	2023	2035	2050
Bloemfontein	1.56%	465,447	560,530	675,038	851,607
Botshabelo	0.59%	181,712	195,106	209,487	228,964
Thaba Nchu	0.59%	70,118	75,278	80,818	88,320
Excelsior	1.12%	6,339	7,244	8,278	9,781
Wepener	0.72%	9,553	10,407	11,337	12,618
Dewetsdorp	0.72%	9,498	10,347	11,272	12,545
Reddersburg	0.72%	4,886	5,323	5,799	6,454
Edenburg	0.72%	6,460	7,037	7,667	8,532
Total		**754,013**	**871,272**	**1,009,696**	**1,218,821**

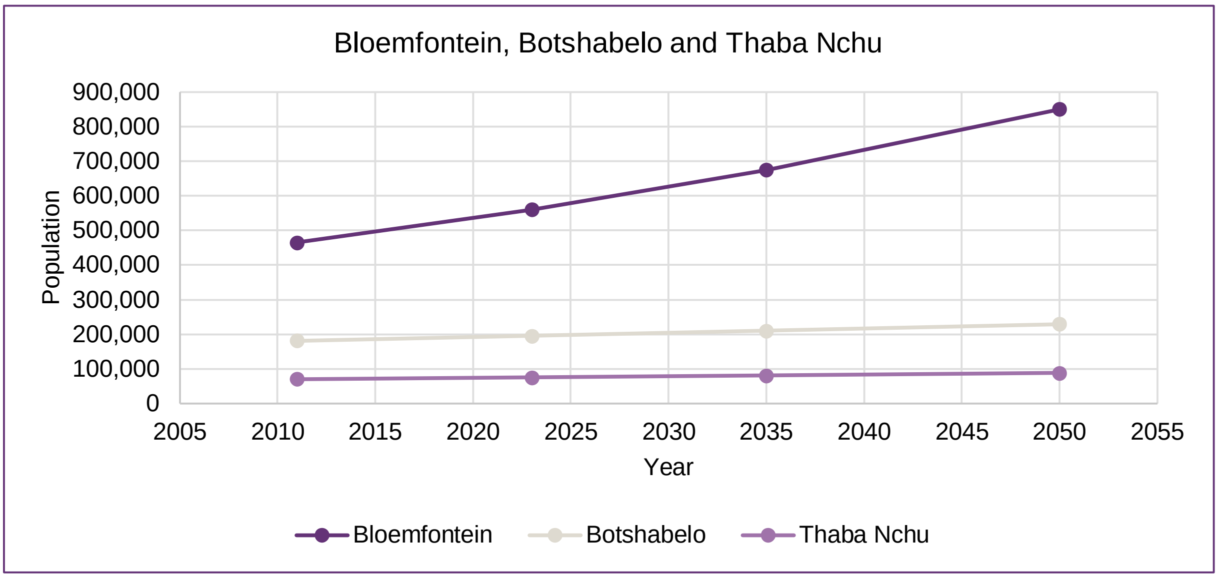


Figure 3-10: Population projection for each major demand centre

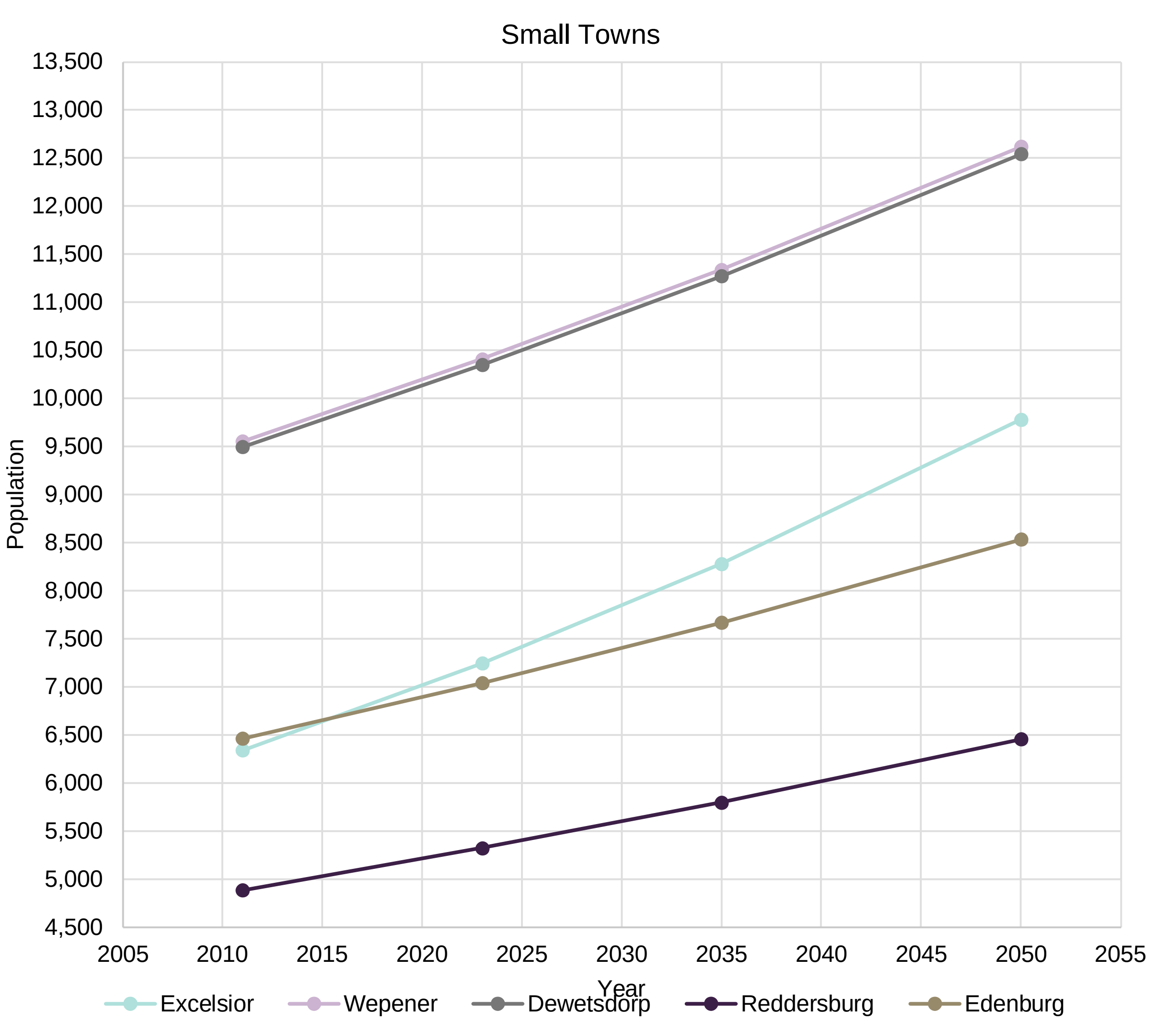


Figure 3-11: Population projection for each small town

3.1.4.2 Change in Level of Service

The change in level of service (LOS), such as improving water services, densification, waterborne sanitation and health awareness will all have an impact on the future water demand. Currently there are people living in the major demand centres and small towns who do not have access to water or only have access to water via a communal standpipe located a specific distance away from where they live. The minimum standard, according to the CSIR Neighbourhood Planning and Design Guide (Red Book), for basic water supply is a minimum quantity of 25 l/p/d and this should be within 200 m from your place of dwelling.

The method followed to determine the increase in water demand that a change in LOS would result in, is as follows:

- Identify the LOS scenario to consider,
- Understand the number of households with access to water and the types of water supply the households have access to,
- Identify, from historic information, the water demand consumption per capita for 2014 (base year),
- Determine the average domestic water demand consumption to be used for the different types of water supply based on the Red Book guidelines and cross check it with historic water demand consumption,

- Determine the average domestic water consumption based on the LOS scenario chosen,
- Calculate the increase in water demand consumption when LOS increase is considered, and
- Assume the LOS will be implemented gradually from 2023 to 2050 and determine the additional water consumption to be applied.

The typical information regarding access to water per household that Stats SA gathers includes:

- Piped (tap) water inside dwelling/institution,
- Piped (tap) water inside yard,
- Piped (tap) water on community stand: distance less than 200 m from dwelling/institution,
- Piped (tap) water on community stand: distance between 200 m and 500 m from dwelling/institution,
- Piped (tap) water on community stand: distance between 500 m and 1000 m (1 km) from dwelling /institution,
- Piped (tap) water on community stand: distance greater than 1000 m (1 km) from dwelling/institution, and
- No access to piped (tap) water.
- The types of water supply considered above were grouped into the following:
  - House connection,
  - Yard connection,
  - Communal standpipe, and,
  - No water supply.

The following change in LOS scenario was considered:

- LOS: By 2050 all households would have piped (tap) water inside dwelling/institution.

According to the Stats SA 2011 census data, a total of 54% (i.e. 42% + 10% + 2%) of households in the major demand centres and small towns mentioned above, did not have access to *piped (tap) water inside dwelling/institution*, as shown in Figure 3-12.

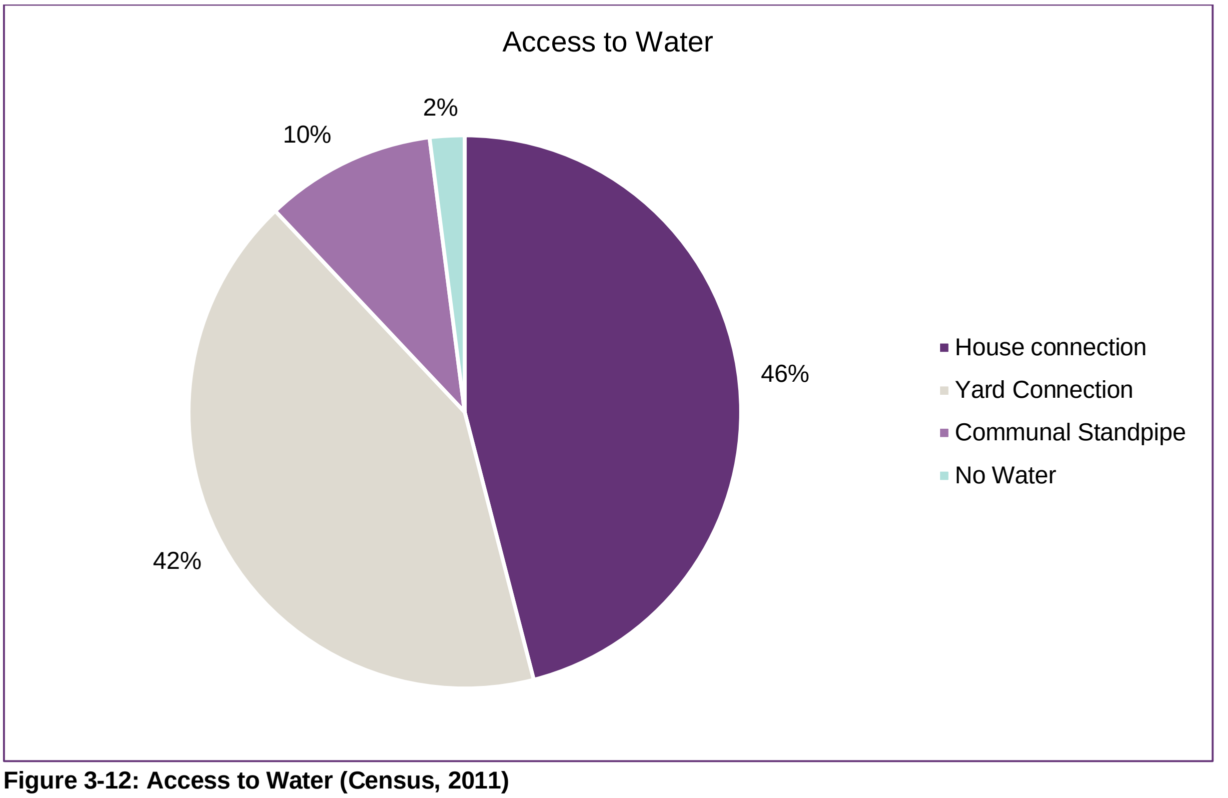


Figure 3-12: Access to Water (Census, 2011)

A breakdown of "access to water" for each type of water supply for the major demand centres and small towns is shown in Table 3-6.

Table 3-6: Access to water

	Bloemfontein / Mangaung	Botshabelo	Thaba Nchu	Excelsior	Wepener	Dewetsdorp	Reddersburg	Edenburg
Annual Population Growth Rate	1.56%	0.59%	0.59%	1.12%	0.72%	0.72%	0.72%	0.72%
Household average size	3.11	3.05	3.59	3.22	3.49	3.06	3.29	3.32
Number of Households: Access to Water (2011)								
House connection	83,273	13,999	6,537	358	891	830	732	801
Yard Connection	44,916	32,847	12,770	1,454	2,214	1,932	687	1,165
Communal Standpipe	18,633	2,808	1,834	6	11	22	26	8
No Water	3,058	938	650	0	10	104	27	8
Total Households	**149,880**	**50,592**	**21,791**	**1,818**	**3,126**	**2,888**	**1,472**	**1,982**
Population: Access to Water (2014)								
House connection	270,902	45,541	21,266	1,165	2,899	2,700	2,381	2,606
Yard Connection	146,120	106,857	41,543	4,730	7,203	6,285	2,235	3,790
Communal Standpipe	60,617	9,135	5,966	20	36	72	85	26
No Water	99,48	3,051	2115	0	33	338	88	26
Total Population	**487,587**	**164,585**	**70,890**	**5,914**	**10,169**	**9,395**	**4,789**	**6,448**
Percentage Population: Access to Water (2014)								
House connection	55.6%	27.7%	30.0%	19.7%	28.5%	28.7%	49.7%	40.4%
Yard Connection	30.0%	64.9%	58.6%	80.0%	70.8%	66.9%	46.7%	58.8%
Communal Standpipe	12.4%	5.6%	8.4%	0.3%	0.4%	0.8%	1.8%	0.4%
No Water	2.0%	1.9%	3.0%	0.0%	0.3%	3.6%	1.8%	0.4%

Historic AADD data from 2014 was used to determine the water demand consumption for each major demand centre and small town as discussed in Section 3.1.4 above.

The water demand consumption, shown in Table 3-7, includes domestic, commercial, industrial, unaccounted for water and other demands. Domestic water is the demand that will be driven by population growth and the change in LOS. The focus of this study was therefore on the domestic demand.

Table 3-7: Historic water consumption per capita, 2014

2014	Population	AAD (Mℓ/a)	Per capita consumption (l/c/d)
Bloemfontein	487,587	67678	380.28
Botshabelo	164,585	10970	162.48
Thaba Nchu	70,890	7380	283.29
Excelsior	5,914	160	241.73
Wepener	10,153	835	234.37
Dewetsdorp	9,395	969	273.52
Reddersburg	4,616	930	510.38
Edenburg	6,396	582	241.73

A cross checking exercise compared the domestic demands assumed from historic data and the typical domestic water demand consumption calculated according to the Red Book criteria (The Department of Human Settlements, 2019). The Department of Human Settlements developed water consumption estimates per capita which can be used to calculate total water demands with reliable population estimates. Using the criteria as per Figure 3-13, the total domestic water demand was determined by applying the water consumption per category with the respective population per category. Table 3-8 shows the average domestic water consumption per capita calculated.

Table 9.10: Water demand for developing areas (IRC 1980)

TYPE OF WATER SUPPLY	TYPICAL CONSUMPTION (ℓ/c/d)	RANGE ℓ/c/d
Communal water point		
• well or standpipe at considerable distance (>1000 m)	7	5-10
• well or standpipe at medium distance (250-1000 m)	12	10-15
• well nearby (<250 m)	20	15-25

Table 9.11: Water consumption in areas equipped with standpipes, yard connections and house connections (adapted from Department of Water Affairs & Forestry, (1992): Guidelines for the selection of design criteria)

DOMESTIC WATER CONSUMPTION

TYPE OF WATER SUPPLY		TYPICAL CONSUMPTION (ℓ/c/d)	RANGE ℓ/c/d
Standpipe (200 m walking distance)		25*	10 - 50
Yard connection			50 - 100
With dry sanitation		55	30 - 60
With LOFLOs			45 - 75
With full-flush sanitation			60 - 100
House connection (developed areas) #			60 - 475
Development level:	Moderate	80	48 - 98
	Moderate to high	130	80 - 145
	High	250	130 - 280
	Very high	450	260 - 480

* This consumption of 25 ℓ/c/d is the minimum to be made available per person in terms of government policy.
# The water demand in this category, based on a different approach, is also given in Table 9.14 and Figure 9.9.

Figure 3-13: Typical water consumption per capita (The Department of Human Settlements, 2019)

Table 3-8: Domestic water consumption (2014) based on Red Book guidelines

Town	Mℓ/d	l/c/d
Bloemfontein / Mangaung	74.67	160.42
Botshabelo	136.62	90.06
Thaba Nchu	75.11	121.88
Excelsior	74.70	86.06
Wepener	77.16	125.82
Dewetsdorp	142.75	101.11
Reddersburg	162.39	148.95
Edenburg	218.48	135.89

The domestic water demands shown in Table 3-8 reflects the current LOS. To determine the impact on water demands due to a change in LOS, where all households will have a house connection by 2050, two approaches can be considered, namely:

- Apply the per increased capita demands as per the Red Book guidelines, or,
- Use the DWS guideline where per capita consumption is linked to income categories.

In the Universal Access Plan for Water Services Phase 2 Report (Umgeni Water, 2016), DWS defined the per capita consumption for the different income categories as shown is Table 3-9. The annual household income range for each demand centre as per the Census 2011 is reflected in Table 3-10.

Table 3-9: Water demand consumption per capita based on income

Category	Description of consumer category	Household Annual Income Range	Average per capita consumption (l/c/d)
1	Very high income: villas, large, detached house, large luxury flats	>R1,228,000	410
2	Upper middle income: detached houses, large flats	R153,601 – R1,228,000	295
3	Average middle income: 2-3 bedroom houses or flats with 1 or 2 WC, kitchen and one bathroom, shower	R38,401 – R153,600	228
4	Low middle income: Small houses or flats with WC, one kitchen, one bathroom	R9,601 – R38,400	170
5	Low income: flatlets, bedsits with kitchen and bathroom, informal household	R0 – R9,600	100

The per capita consumption shown in Table 3-9 was applied to the corresponding income category reflected in Table 3-10. The distributions for the households per income category were calculated and corresponding average water demand consumption determined, as shown in Table 3-10.

Table 3-10: Average per capita domestic consumption

Income categories	Per capita consum. (l/c/d)	Number of households (2014)							
		Bloemfontein	Botshabelo	Thaba Nchu	Excelsior	Wepener	Dewetsdorp	Reddersburg	Edenburg
No income	100	7,791	6,382	2,809	15	107	4	17	31
R 1 - R 4800	100	2,256	3,827	1,337	10	22	201	0	20
R 4801 - R 9600	100	3,652	5,263	1,941	3	23	296	0	30
R 9601 - R 19 600	170	9,076	11,029	4,688	33	86	667	16	75
R 19 601 - R 38 200	170	11,857	12,694	4,896	32	48	725	33	92
R 38 201 - R 76 400	228	10,167	6,812	2,892	31	43	316	45	58
R 76 401 - R 153 800	228	10,842	2,987	1,786	25	44	194	45	9
R 153 801 - R 307 600	295	12,133	1,191	977	28	37	123	28	6
R 307 601 - R 614 400	295	9,270	381	380	12	28	34	12	5
R 614 001 - R 1 228 800	295	3,259	61	64	0	3	9	7	9
R 614 001 - R 1 228 800	295	3,259	61	64	0	3	9	7	9
R 1 228 801 - R 2 457 600	410	756	35	41	0	3	11	0	0
R 2 457 601 or more	410	637	30	27	3	0	8	0	3
Unspecified	0	10	0	0	0	0	0	0	0
Total households		**81,706**	**50692**	**21836**	**193**	**445**	**2898**	**203**	**429**
Average annual income		**R188,144.57**	**R38,552.40**	**R54,961.11**	**R131,291.19**	**R89,771.74**	**R57,394.74**	**R133,600.94**	**R108,139.81**
Average per capita consumption l/c/d		**216.04**	**164.18**	**171.80**	**206.50**	**178.17**	**169.38**	**218.79**	**193.38**

Table 3-11 provides a comparison of the per capita consumption calculated with the two methods when assuming that every household will have a house connection by 2050.

Table 3-11: Per capita consumption based on Red Book and DWS guidelines for changed LOS

Town	Per capita consumption incl. LOS (DWS) l/c/d	Total demand (DWS) Mℓ/d	Per capita consumption incl. LOS (Red Book) l/c/d	Total demand (Red Book) Mℓ/d
Bloemfontein / Mangaung	216.04	147.31	198.46	141.85
Botshabelo	164.18	21.13	146.86	19.68
Thaba Nchu	171.80	11.65	178.34	11.87
Excelsior	204.37	0.69	146.39	0.48
Wepener	176.97	1.43	179.70	1.44
Dewetsdorp	169.44	1.69	156.51	1.63
Reddersburg	216.66	1.49	188.06	1.41
Edenburg	193.83	1.01	180.90	0.97
Total demand (mil m^3/a)	**-**	**186.40**	**-**	**179.33**
Total demand (Mℓ/d)	**-**	**510.70**	**-**	**491.30**

It is evident from Table 3-11 that both methods present similar water demands in 2050, with the DWS method producing slightly higher water demands in 2050 compared to the Red Book method. Adopting a more conservative approach because projections are up to 2050, it was proposed that the calculated increase in water demand due to the change in LOS be based on the DWS guidelines – i.e. linked to the household incomes.

The difference an increase in the LOS had on the per capita consumption is shown in Table 3-12. It was assumed that the LOS increase will gradually be implemented as this is a more realistic approach, which will result in an incremental increase in demand from 2023 to 2050. Therefore, by 2050 all households were assumed to have a house connection for access to water. Figure 3-14 provides a graphical interpretation of the additional water demand applied to the base water demand.

Table 3-12: Per capita consumption increase for LOS

Town	Per capita consumption incl. LOS (DWS Guideline) l/c/d	Per capita consumption (historic) l/c/d	Difference l/c/d	Difference per year (2023 to 2050) l/c/d
Bloemfontein / Mangaung	216.04	160.42	55.62	2.06
Botshabelo	164.18	90.06	74.12	2.75
Thaba Nchu	171.80	121.88	49.92	1.85
Excelsior	204.37	86.06	118.31	4.46
Wepener	176.97	125.82	51.15	1.94
Dewetsdorp	169.44	101.11	68.33	2.53
Reddersburg	216.66	148.95	67.71	2.59
Edenburg	193.83	135.90	57.93	2.13

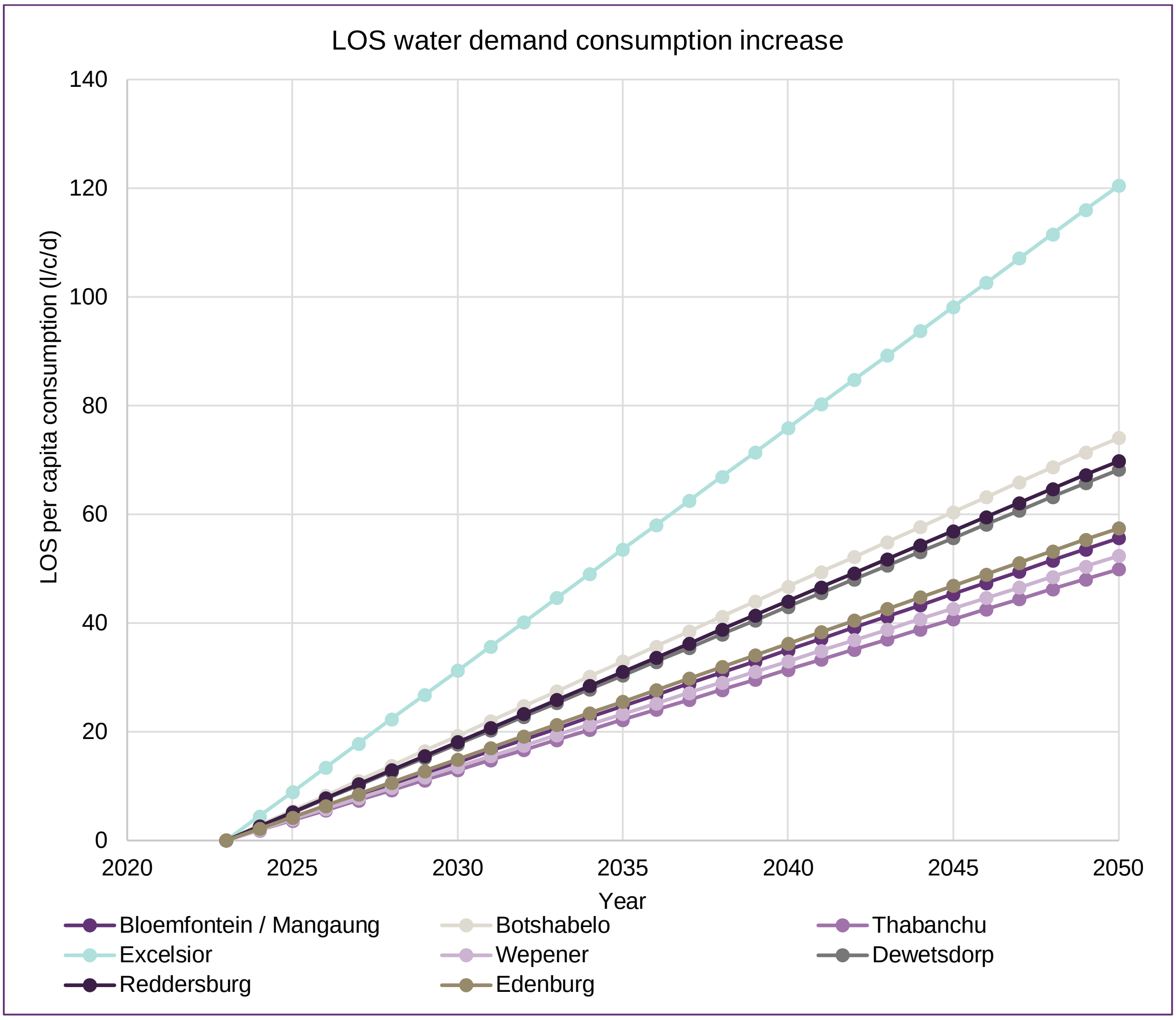


Figure 3-14: Additional LOS per capita consumption applied from 2023 to 2050

3.1.5 Future Water Demand Scenarios

The drivers contributing to the water demand projections, discussed above, were used to determine which future water demands scenarios to consider. The identified scenarios are shown in Table 3-13.

Table 3-13: Future water demand projection scenarios

Scenario	Population Growth	Change in level of service LOS
Scenario 1	x	
Scenario 2	x	x

The base year used for projections was 2014 as this was prior to water restrictions being implemented. Historic AADD data and the population data for 2014 was used to determine the water consumption requirement for each of the demand centres. A 10% bulk conveyance loss was applied to determine the GAADD.

The water consumption for Scenario 1 used the base per capita demand and Scenario 2 used the base per capita demand from year 2014 to 2023 and from 2024 to 2050 an additional LOS demand is incrementally applied each year. The 2014 base data applied for each scenario is compiled in Table 3-14.

Table 3-14: 2014 Base water demand data

Scenario 1	Bloemfontein	Botshabelo	Thaba Nchu	Excelsior	Wepener	Dewetsdorp	Reddersburg	Edenburg
Annual Population Growth Rate	1.56%	0.59%	0.59%	1.12%	0.72%	0.72%	0.72%	0.72%
Population (2014)	487,587	184,972	71,374	6,554	9,760	9,703	4,992	6,600
Water Consumption (l/c/d)	380.28	162.48	283.29	67.04	234.37	273.52	510.38	241.73
Water Consumption incl. 10% losses (l/c/d)	418.31	178.73	311.61	73.75	257.81	300.88	561.42	265.90
Scenario 2	**Bloemfontein**	**Botshabelo**	**Thaba Nchu**	**Excelsior**	**Wepener**	**Dewetsdorp**	**Reddersburg**	**Edenburg**
Annual Population Growth Rate	1.56%	0.59%	0.59%	1.12%	0.72%	0.72%	0.72%	0.72%
Population (2014)	487,587	184,972	7,1374	6,554	9,760	9,703	4,992	6,600
Water Consumption – AADD (l/c/d)	380.28	162.48	283.29	67.04	234.37	273.52	510.38	241.73
Water Consumption incl. 10% losses - GAADD (l/c/d)	418.31	178.73	311.61	73.75	257.81	300.88	561.42	265.90
LOS water consumption yearly increase, only from 2024 (l/c/d)	2.06	2.75	1.85	4.46	1.94	2.53	2.59	2.13

3.1.6 Future Water demand Projections

Applying the above-mentioned growth scenarios to the base water demand in 2014 (refer to Table 3-14), water demand projections were determined based on population growth and change in level of service for each demand centre within the GBWSS. The small towns were grouped so that the demand projections are presented for the following towns/areas:

- Bloemfontein (largest town)
- Botshabelo & Thaba Nchu (medium-large towns)
- Other Small Towns

3.1.6.1 Bloemfontein

Figure 3-15 displays the historical and projected water demands for Bloemfontein. Scenario 1, where the water demand was projected based only on population growth, calculated a water demand of approximately 130 million m^3/a in 2050. Scenario 2 included both population growth and an increase in the LOS and projected a demand of 147 million m^3/a in 2050.

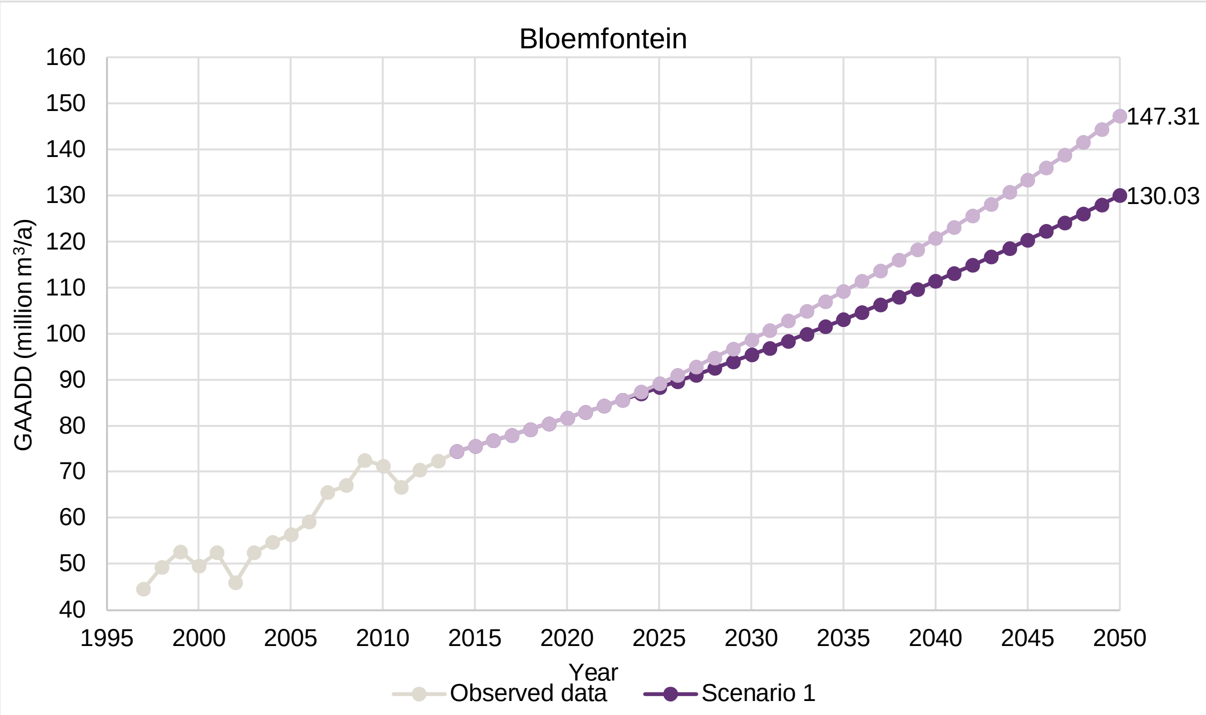


Figure 3-15: Water demand Projections for Bloemfontein

It is evident from Figure 3-15 that:

- Investing in water service delivery increased future demands by approximately 10%, and,
- The projected water demands increased at roughly the same rate as the historic water demands.

3.1.6.2 Botshabelo

The projected water demands for Botshabelo are shown in Figure 3-16. The demands for Scenarios 1 and 2 were projected to reach approximately 15 million m^3/a and 21 million m^3/a by 2050, respectively.

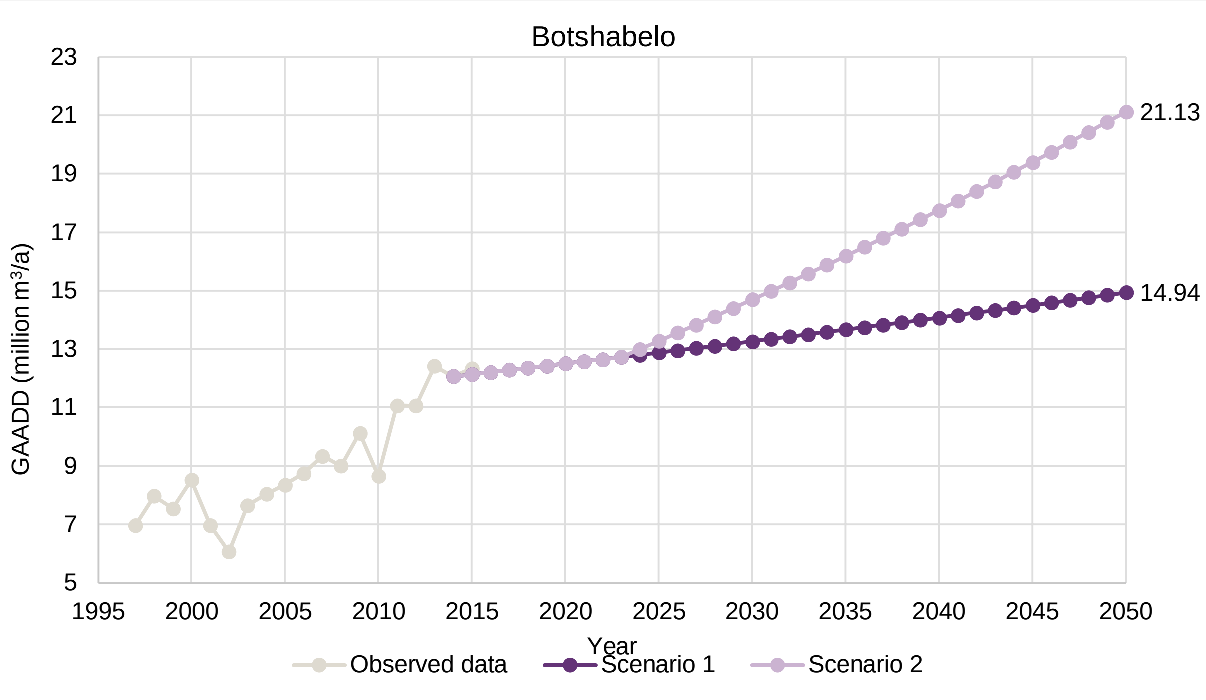


Figure 3-16: Water demand Projections for Botshabelo

It is evident from Figure 3-16 that:

- Increasing the LOS increased the water demand by approximately 40% in 2050, and,
- The projected water demands grew at a lower rate compared to the historic growth in water demand.

3.1.6.3 Thaba Nchu

The projected water demands for Thaba Nchu are shown in Figure 3-17. The demands for Scenarios 1 and 2 were projected to reach approximately 10 million m^3/a and 12 million m^3/a by 2050, respectively.

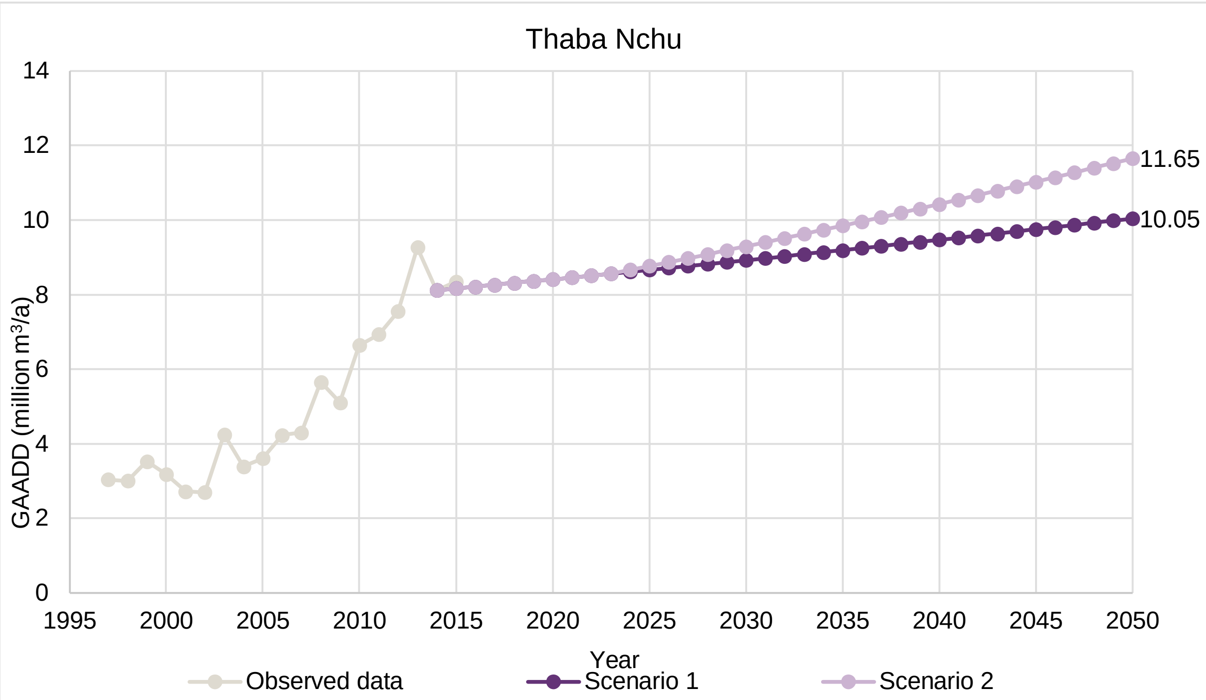


Figure 3-17: Water demand Projections for Thaba Nchu

It is evident from Figure 3-17 that:

- Increasing the LOS increased the water demand by approximately 40% in 2050, and,
- The projected water demands grew at a lower rate compared to the historic growth in water demand.

3.1.6.4 Small Towns

The combined projected water demands for the small towns of Excelsior, Wepener, Dewetsdorp, Reddersburg and Edenburg are shown in Figure 3-18. The demands for Scenarios 1 and 2 were projected to reach approximately 5 million m^3/a and 6 million m^3/a by 2050, respectively. It was also evident that the increase in LOS increased the water demand by approximately 27% by 2050.

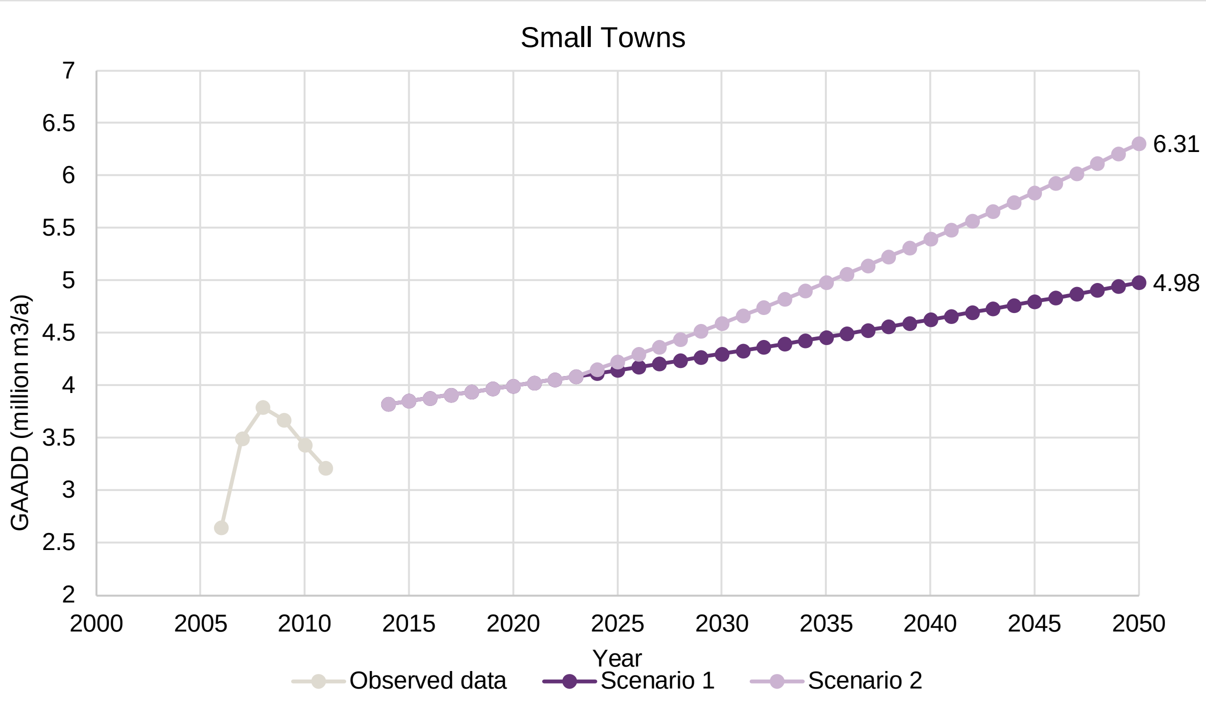


Figure 3-18: Water demand Projections for Small Towns

3.1.6.5 Total Demands

Figure 3-19 displays the historical and projected water demands for the complete study area. Key demand figures have been summarised in Table 3-15 for all the scenarios. Scenario 1, where population growth was used to project the water demands to 2050, resulted in a water demand of 160 million m^3/a. When the LOS was incrementally increased from 2023 to 2050, the impact was an additional 26 million m^3/a, equating to 186 million m^3/a.

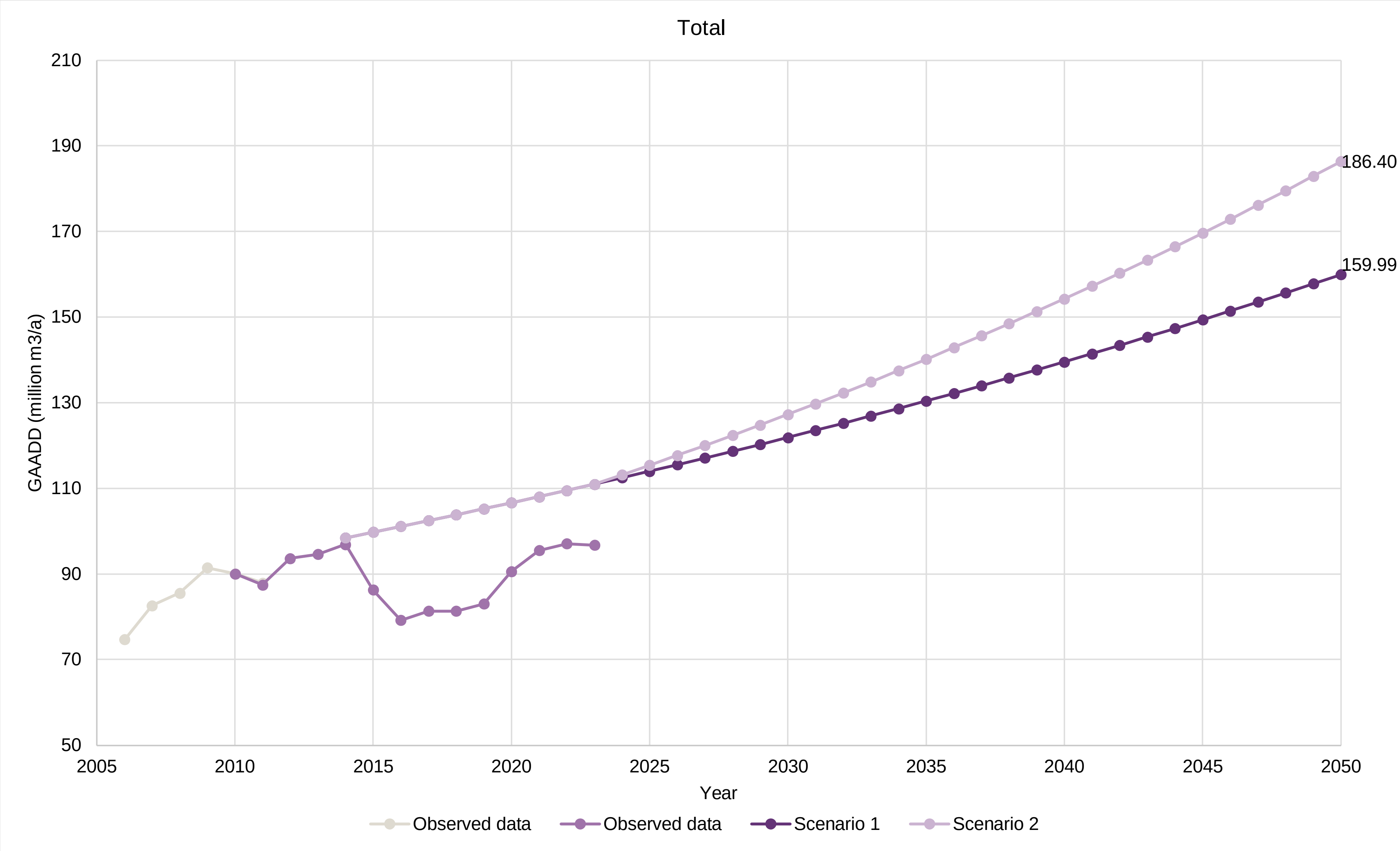


Figure 3-19: Total Water demand Projections for the Study Area

Table 3-15: Future Water demands for Scenario 1 and 2

Water Demand (million m³/a)					
Scenario 1	**2014**	**2023**	**2035**	**2040**	**2050**
Bloemfontein	74.45	85.58	103.07	111.37	130.03
Botshabelo	12.07	12.73	13.67	14.08	14.94
Thaba Nchu	8.12	8.56	9.19	9.47	10.05
Excelsior	0.18	0.19	0.22	0.24	0.26
Wepener	0.92	0.98	1.07	1.11	1.19
Dewetsdorp	1.07	1.14	1.24	1.28	1.38
Reddersburg	1.02	1.09	1.19	1.23	1.32
Edenburg	0.64	0.68	0.74	0.77	0.83
Total million m³/a	**98.45**	**110.96**	**130.38**	**139.54**	**159.99**
Total Mℓ/d	**269.74**	**303.99**	**357.22**	**382.30**	**438.32**
Scenario 2	**2014**	**2023**	**2035**	**2040**	**2050**
Bloemfontein	74.45	85.58	109.16	120.69	147.31
Botshabelo	12.07	12.73	16.19	17.75	21.13
Thaba Nchu	8.12	8.56	9.85	10.42	11.65
Excelsior	0.18	0.19	0.38	0.48	0.69
Wepener	0.92	0.98	1.16	1.25	1.43
Dewetsdorp	1.07	1.14	1.36	1.47	1.69
Reddersburg	1.02	1.09	1.25	1.33	1.49
Edenburg	0.64	0.68	0.82	0.88	1.01
Total million m³/a	**98.45**	**110.96**	**140.05**	**154.07**	**186.06**
Total Mℓ/d	**269.74**	**303.99**	**383.69**	**422.12**	**509.75**

Given that the LOS will increase in future, it is proposed that Scenario 2 be adopted for the projected water demands. These projected demands shall be compared to water demands determined in previous studies in Section 3.1.7.

3.1.7 Comparison of Projections to Previous Studies

Several previous studies were complete by the time this study commenced. The following future water demand projections are compared in this section:

- 2012 Reconciliation Strategy,
- 2015 Technical Feasibility Study,
- 2018 Mangaung Study,
- 2022 / 2023 AOA,
- Observed Projected water demands, and
- Scenario 2 water demands.

To compare the various water demand growth predictions, Scenario 2's projected demands from this study have been plotted alongside the following demands:

- Water demands projected in the previous studies,
- "Observed Projected" which refers to the historic water demands growth rate used to project water demands into the future.

These comparisons are graphically shown for Bloemfontein (Figure 3-20), Botshabelo and Thaba Nchu (Figure 3-21), Small Towns (Figure 3-22) and a Total of all the water demands combined (Figure 3-23), respectively. The resulted are also displayed in Table 3-16, Table 3-17, Table 3-18, and Table 3-19.

Table 3-16: Bloemfontein water demand comparison

Source	2023		2035		2050		Average % Increase
	mil m^3/a	Mℓ/d	mil m^3/a	Mℓ/d	mil m^3/a	Mℓ/d	
Observed Projected	91.46	250.58	113.15	309.99	140.25	384.25	1.60%
Scenario 2	85.58	234.47	109.16	299.06	147.31	403.60	2.03%
2012 Reconciliation Strategy	117.73	322.55	169.82	465.26	268.45	735.49	3.10%
2015 Technical Feasibility Study	85.96	235.52	109.02	298.69	146.73	402.00	2.00%
2018 Mangaung Study	71.81	196.74	88.02	241.16	111.40	305.21	1.64%
2022/23 AOA	71.81	196.74	88.02	241.16	109.83	300.89	1.59%

The annual average water demand growth rate for Bloemfontein varied from 1.59% to 3.10%. Scenario 2, 2022/23 AOA and the 2018 Mangaung Study had an average water demand growth rate between 1.59% and 1.64%. The 2015 Technical Feasibility Study averaged 2.0% and the 2012 Reconciliation Strategy chose a conservative growth rate of 3.1% to project the water demands although the 3% was considered the upper limit with 2% considered an “average” growth scenario.

The previous studies’ information around the percentage bulk conveyance loss applied to the AADD is vague and therefore this uncertainty potentially plays a role in the different water demands projected. This study used 10% losses, the 2012 Reconciliation Strategy used 12% and it is believed the other studies used 3%, however this was not entirely clear from the previous reports and clarity was not received on this matter when requested from MMM.

The 2015 Technical Feasibility Study and this study (Scenario 2) reported similar demand projections, for 2050, in the range of 147 million m^3/a. The 2012 Reconciliation Strategy assumed a much higher growth rate of 3.1% and therefore projects a demand of 268 million m^3/a. The 2018 Mangaung Study and the 2022/23 AOA reported slightly lower demands of 111 million m^3/a and 110 million m^3/a respectively. The Observed Projected water demands reported 140 million m^3/a in 2050.

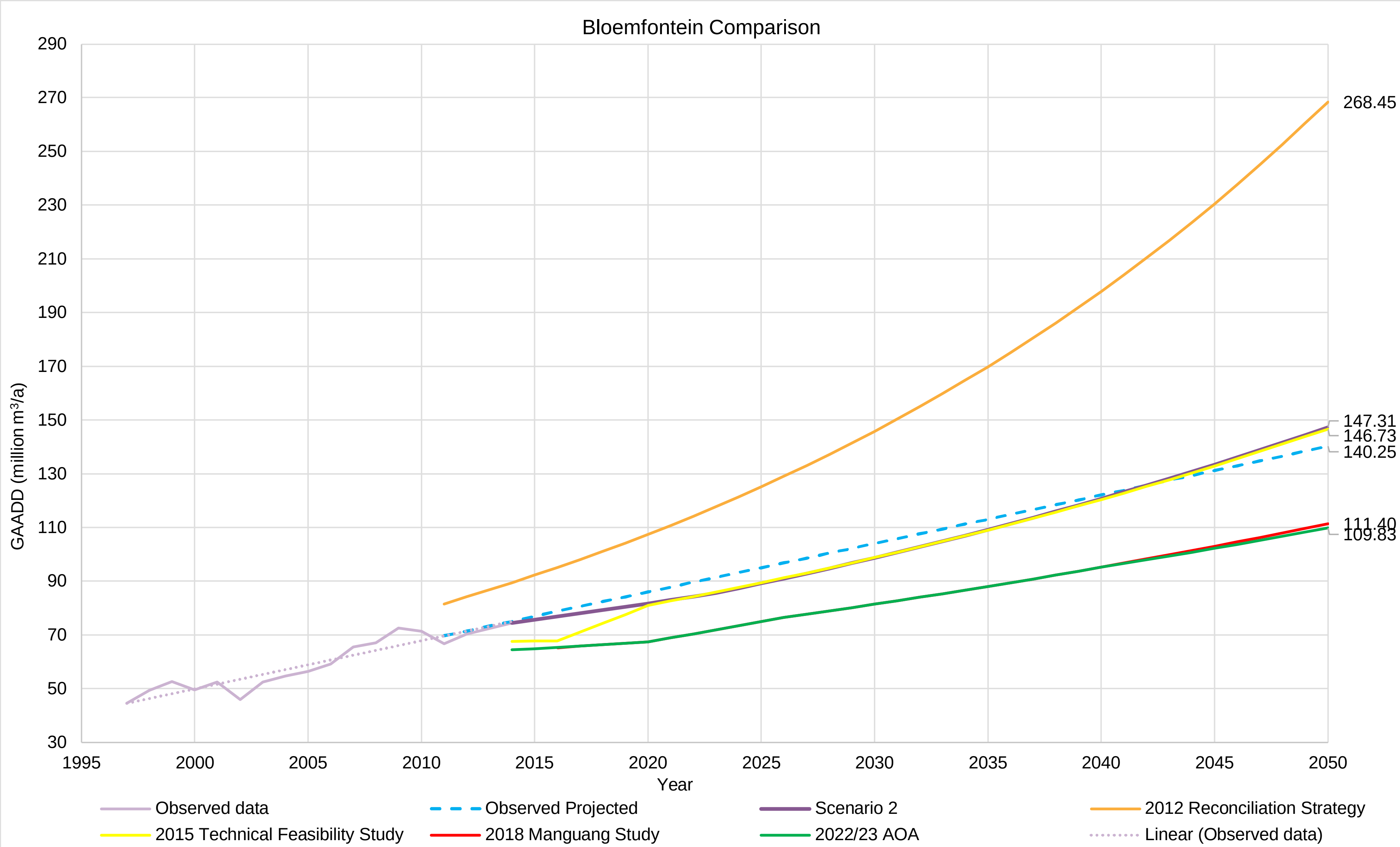


Figure 3-20: Bloemfontein future water demand requirements comparison with Scenario 2

Table 3-17: Botshabelo and Thaba Nchu water demand comparison

Source	2023		2035		2050		Average % Increase
	mil m^3/a	Mℓ/d	mil m^3/a	Mℓ/d	mil m^3/a	Mℓ/d	
Observed Projected	25.19	69.01	33.01	90.44	42.78	117.22	1.98%
Scenario 2	21.29	58.33	26.03	71.32	32.79	89.82	1.61%
2012 Reconciliation Strategy	10.95	30.01	11.17	30.61	11.45	31.38	0.17%
2015 Technical Feasibility Study	27.55	75.48	34.94	95.73	47.03	128.84	2.00%
2018 Mangaung Study	38.51	105.49	47.41	129.88	62.06	170.04	1.78%
2022/23 AOA	32.28	88.44	46.03	126.10	68.90	188.78	2.85%

Botshabelo and Thaba Nchu's demand projections varied substantially depending on the assumptions that were made for the chosen growth rates. It was found that even though the population growth rates for these two towns were low (approximately 0.59%), previous studies reported very high water demand growth rates. These growth rates varied from 0.17% to 2.85% when looking at the years 2023 to 2050.

The 2012 Reconciliation Strategy assumed a much lower population growth rate and therefore projects a demand of 11 million m^3/a. Scenario 2 reported a water demand of 33 million m^3/a, while the 2015 Technical Feasibility Study projected a water demand of 47 million m^3/a. The 2018 Mangaung Study and the 2022/23 AOA reported much higher demands ranging from 62 million m^3/a to 69 million m^3/a. The population growth rates for these two towns do not support the very high future water demands represented in the 2018 Mangaung and 2022/23 AOA.

However, it has been visually observed that informal settlements in these two towns are growing at a rapid pace. Previous studies raised the point that the non-revenue water has been very high and continues to grow. When the historic water demand growth rate was projected into the future, the water demand in 2050 was found to be 42 million m^3/a.

Table 3-18: Small Towns water demand comparison

Source	2023		2035		2050		Average % Increase
	mil m^3/a	Mℓ/d	mil m^3/a	Mℓ/d	mil m^3/a	Mℓ/d	
Observed Projected	4.35	11.93	5.22	14.29	6.30	17.25	1.38%
Scenario 2	4.08	11.19	4.97	13.63	6.29	17.23	1.61%
2012 Reconciliation Strategy	5.24	14.35	5.79	15.86	6.58	18.02	0.85%
2015 Technical Feasibility Study	2.27	6.23	2.88	7.90	3.88	10.63	2.00%
2018 Mangaung Study	4.39	12.03	5.21	14.27	6.39	17.51	1.40%
2022/23 AOA	4.39	12.02	5.20	14.25	6.43	17.62	1.43%

The Small Towns demand projections varied depending on the assumptions that were made for the chosen growth rates. The average water demand growth rates varied from 0.85% to 2% for 2023 to 2050. Very little historic water demand data was available for the individual Small Towns. All studies, except for the 2015 Technical Feasibility Study, derived a water demand in 2050 of approximately 6 million m^3/a. The 2015 Tech Feasibility Study reported a demand of 3.88 million m^3/a which was much lower than any of the other projections.

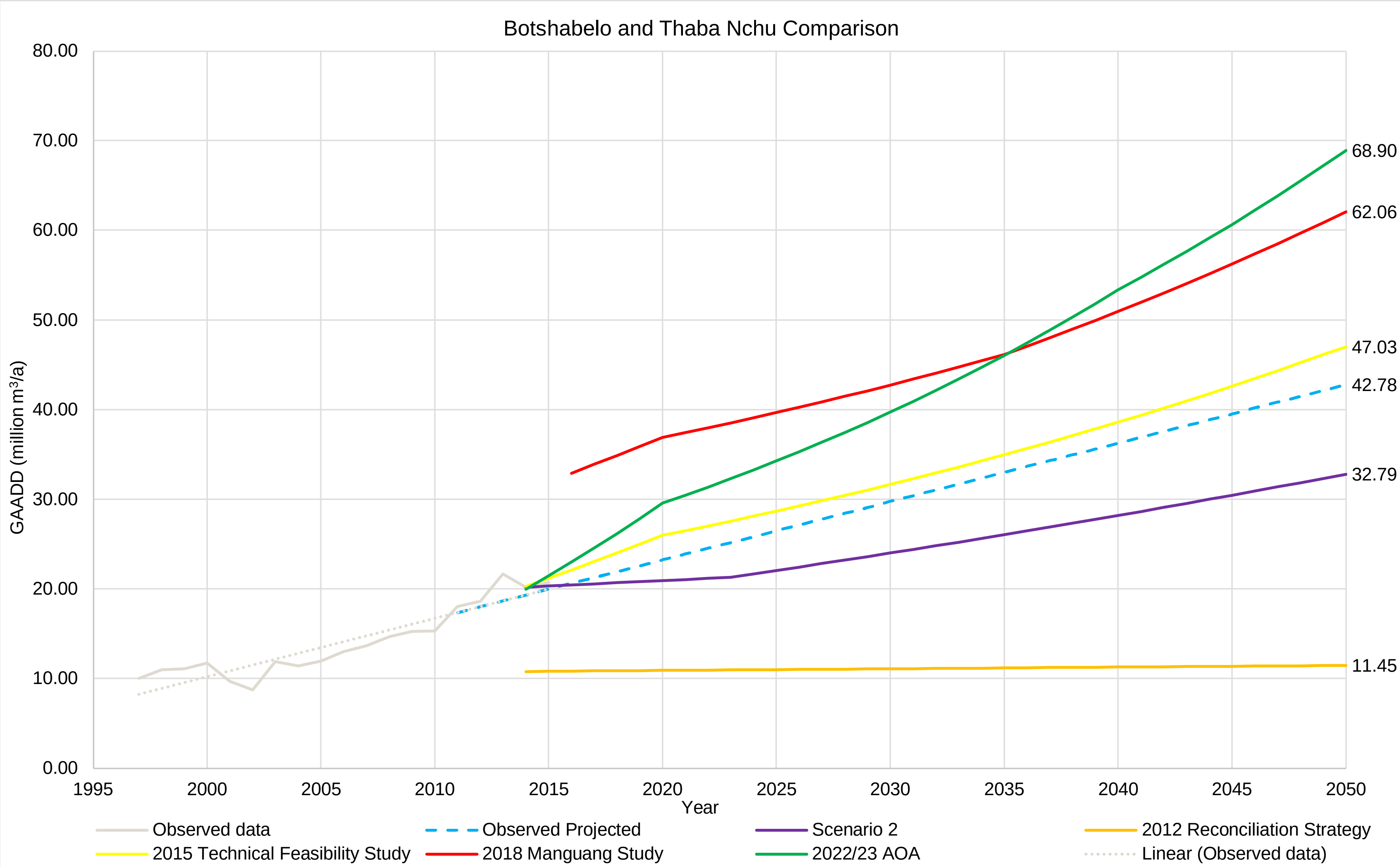


Figure 3-21: Botshabelo and Thaba Nchu Future Water Demand Requirements comparison with Scenario 2

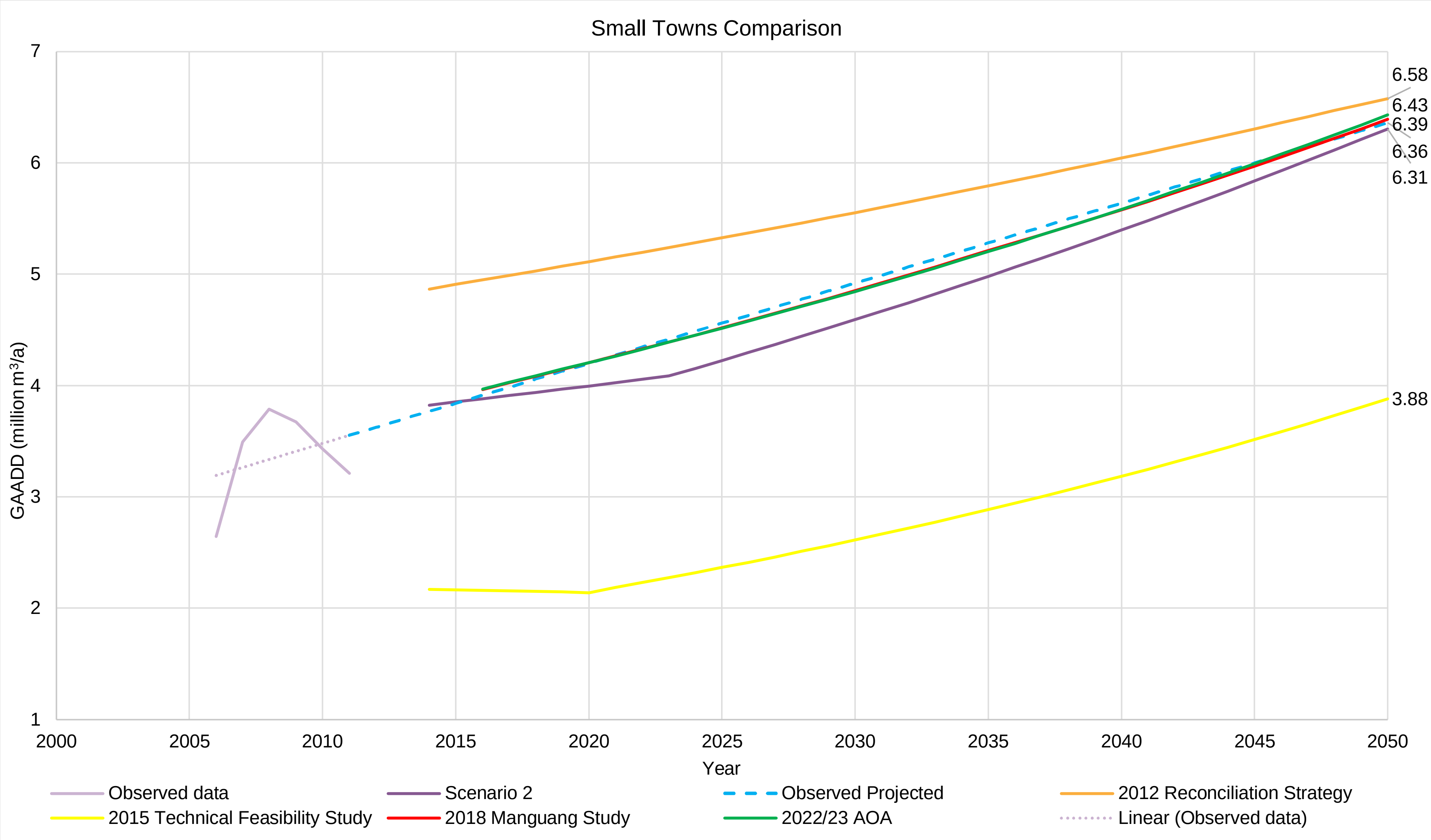


Figure 3-22: Small Towns Future Water Demand Requirements comparison with Scenario 2

Table 3-19: Total comparison results

Source	2023		2035		2050		Average % Increase
	mil m³/a	Mℓ/d	mil m³/a	Mℓ/d	mil m³/a	Mℓ/d	
Observed Projected	118.21	323.86	145.55	398.78	179.74	492.42	1.56%
Scenario 2	110.96	303.99	140.17	384.02	186.40	510.70	1.94%
2012 Reconciliation Strategy	133.92	366.91	186.78	511.74	286.49	784.89	2.86%
2015 Technical Feasibility Study	115.79	317.23	146.85	402.33	197.64	541.48	2.00%
2018 Mangaung Study	114.71	314.26	139.35	381.78	179.86	492.76	1.68%
2022/23 AOA	108.48	297.20	139.25	381.51	185.16	507.29	2.00%

For the total water demands for all the demand centres, the Observed Projected water demands were calculated by determining the linear historic water demand growth rate and using that rate to project into the future. The 2018 Mangaung Study and the Observed Projected water demands were both similar and reached 180 million m^3/a by 2050. The 2022/23 AOA predicted a slightly higher water demand of 185 million m^3/a by 2050, like that of Scenario 2, which predicted a water demand of 186 million m^3/a.

The 2015 Technical Feasibility Study predicted a water demand of 198 million m^3/a by 2050. Due to the conservative approach taken in the 2012 Reconciliation Strategy and the higher water demand growth rate that was used, a water demand of 286 million m^3/a was calculated.

In summary, all studies with the exception of the 2012 Reconciliation Strategy, reported a range for the year 2050 between 179 million m^3/a to 197 million m^3/a. The demands determined as part of Scenario 2 corresponded closely to demand projections from previous studies.

There was, however, some discrepancies noted between this study (Scenario 2) and previous studies relating to the distribution of the water demands between the three largest demand centres (i.e. Bloemfontein, Botshabelo and Thaba Nchu). This highlighted the importance of incorporating flexibility into the proposed bulk water infrastructure to enable an increase or decrease in supply to the respective demand centres. This is further discussed in Chapter 7.5 of the report.

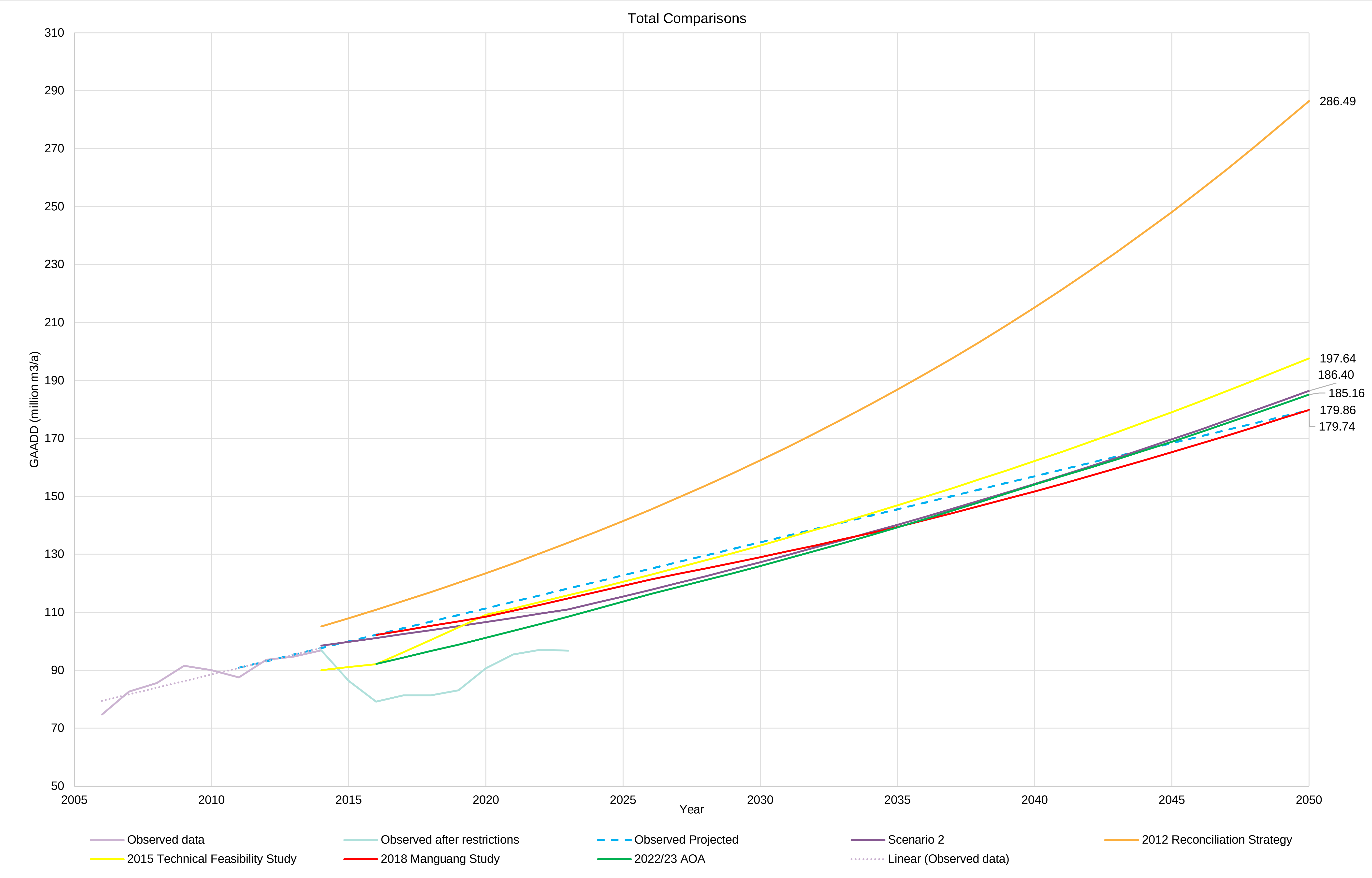


Figure 3-23: Total Future Water Demand Requirements comparison with Scenario 2

3.1.8 Possible Future Intervention Options

There is potential to supply several small towns along the pipeline route as shown in Figure 3-1. These towns were identified as Bethulie, Bethany, Colesberg, Fauresmith, Gariep, Jagersfontein, Philippolis, Springfontein, and Trompsburg. Current water demand data was not readily available for these small towns and all projections for future water demands were based on information provided in the Development of a Reconciliation Strategy for All Towns in the Central Region (DWS, 2011), referred to as the All Towns Study. A brief description of the small towns including the 2008 population and water demands is contained in Table 3-20.

Table 3-20: Small Towns Data (2008)

Town	Settlements	Local Municipality	2008	
			Population	AADD (M/d)
Bethulie	Bethulie, Lephoi and Cloetespark	Kopanong Local Municipality	6610	4.62
Bethany		Kopanong Local Municipality	1000	0.20
Colesburg	Colesberg, Kuyasa, Lowryville, Towervallei and Chris Hani	Umsombomvu Local Municipality	11741	3.02
Fauresmith	Fauresmith, Ipopeng and Frayville	Kopanong Local Municipality	3921	0.45
Gariep Dam		Kopanong Local Municipality	1325	1.76
Jagersfontein	Jagersfontein, Itumeleng and Charlesville	Kopanong Local Municipality	6630	2.03
Philippolis	Philippolis, Poding-Tse-Rolo and Bergmanshoogte	Kopanong Local Municipality	3292	0.47
Springfontein	of Springfontein, Williamsville and Maphodi	Kopanong Local Municipality	3703	1.95
Trompsburg	Trompsburg, Noordville and Madikgetla	Kopanong Local Municipality	4040	1.62

3.1.8.1 Future Water demands

All the small towns fall under the Kopanong Local Municipality except for Colesberg which falls under Umsombomvu Local Municipality. The future water demands were based on the population growth rates as obtained from the 2011 population census data, the 2016 community population survey and the 2022 population census data for the two local municipalities. The population data and percentage annual growth in population can be found in Table 3-21.

Table 3-21: Small Towns Population Growth

	2011 Population	2016 Population	2022 Population	% Annual Growth
Kopanong Local Municipality	49,171	49,999	51,832	0.48%
Umsombomvu Local Municipality	28,376	30,883	29,555	0.37%

The litres per capita per day demands were calculated using the AADD data found in the All Towns Study (DWS, 2011). A bulk conveyances loss of 10% was added to these demands to determine the GAADD. This information is summarised in

Table 3-22.

Table 3-22: Small Towns Water Demand Data (2008)

Small Town	AAD (Mℓ/a)	AADD (Mℓ/d)	AADD (l/c/d)	GAADD (l/c/d incl. 10% Bulk Conveyance Losses)
Bethulie	1,684.84	4.62	698	768
Bethanie	73.00	0.20	200	220
Colesberg	1,102.30	3.02	257	283
Fauresmith	164.25	0.45	115	126
Gariep	640.58	1.76	1325	1457
Jagersfontein	739.49	2.03	306	336
Philippolis	171.92	0.47	143	157
Springfontein	163.89	0.45	121	133
Trompsberg	167.90	0.46	114	125

Using the information provided above the future water demands were projected to 2050. It should be clarified that very little historic information on the water demands was found and therefore the confidence rating in the demand projection is low. Table 3-23 shows the projections for each of the small towns up to the year 2050.

Table 3-23: Future Water demand Projections

	Water Demand GAADD (Mℓ/d)			
	2008	**2023**	**2035**	**2050**
Bethulie	5.08	5.46	5.78	6.21
Bethanie	0.22	0.24	0.25	0.27
Colesberg	3.32	4.99	5.22	5.51
Fauresmith	0.50	0.53	0.56	0.61
Gariep Dam	1.93	2.07	2.20	2.36
Jagersfontein	2.23	2.39	2.54	2.73
Philippolis	0.52	0.56	0.59	0.63
Springfontein	2.15	2.30	2.44	2.62
Trompsburg	1.78	1.91	2.03	2.18
Total Mℓ/d	**17.72**	**20.46**	**21.60**	**23.12**
Total million m^3/a	**6.47**	**7.47**	**7.88**	**8.44**

The total projected demand of 186 million m^3/a for the overall GBWSS excludes the water demands shown in Table 3-23. These small towns have existing supplies that can satisfy their current water demands. It might, however, become more financially viable in future to connect to the Xhariep Pipeline infrastructure in which case provision for this increase in water demand must be made.

Excluding the town of Colesberg, the additional water demand by 2050 was calculated to be only 2.93 million m^3/a or a 1.6% increase in the projected water demand of 186 million m^3/a. Even with Colesberg included, it still represented an increase in water demand of only 4.5%.

As part of the stochastic system analysis undertaken (Section 3.2.2.2.2), the level of assurance was determined at which water demands of 186 million m^3/a and 194 million m^3/a can be supplied. It is further proposed that provision be made in the infrastructure planning (e.g. footprint allowed for in the WTWs to allow for future expansion should it become necessary to supply these smaller towns; this is addressed in Section 7.7.

3.2 Water Resource Analysis

The analysis involved an assessment of several augmentation and operational scenarios to confirm the potential increase in the yield of the GBWSS due to abstraction of water from Gariep Dam via the proposed Xhariep Pipeline. The water resources analysis is detailed in the Water Resource Analysis Report, but a summary is included in this report.

The outcomes of the scenario analyses informed the decision regarding the timing and optimal route of the pipeline. As described in Section 1.3, the planning horizon was 2050 with an interim development level in 2035 also included. In addition to confirming the size of the Xhariep Pipeline and associated bulk transfer infrastructure, the need for increased capacities of WTWs and bulk conveyance infrastructure within the larger GBWSS were also assessed.

As mentioned in Section 1.4, the methodology was iterative and involved sequential changes in the model after communication with the infrastructure design team. Initially the transfer volume was limited to 60 million m^3/a which was found to be insufficient to meet the 2050 demands. The water resources analysis was then optimised to meet the required transfer volume per scheme.

3.2.1 Model Configuration

3.2.1.1 Model Configuration Approach

The approach to configuring the Water Resources Yield Model (WRYM) for this study was based on the Core Base Scenario from the ongoing ORASECOM study (ORASECOM, 2020), with enhancements drawn from the 2022/23 AOA (DWS, 2023) for the Orange River System and the 2018 Mangaung Study (MMM, 2018). This amalgamation aimed to create an updated WRYM configuration tailored to the study's analysis requirements. The new datasets were adjusted to match the on-ground operational activities as at the 2023 development level and incorporated planned interventions on the GBWSS.

The WRYM configuration for this study incorporated additional elements to the (ORASECOM, 2020) WRYM such as:

- Groothoek and Rustfontein WTWs,
- A connection between Rustfontein WTW and Bloemfontein,
- A "bi-directional" pipeline intervention option. This intervention involved a pipeline between Welbedacht Dam and Knellpoort Dam, enabling abstraction from Welbedacht Dam and supporting Welbedacht WTW with de-silted water.
- Additionally, a Gariep dummy dam was included to ensure an infinite water supply for augmentation during model simulations.

Significant changes to the WRYM configuration included:

- Incorporating demands for different development levels in the GBWSS and upstream areas,
- Adjusting operating levels of some dams to maximize yield, and,
- Increasing capacities for various infrastructure elements including WTWs and pump stations, to accommodate different scenarios and interventions.

3.2.1.2 WTW Modelled Capacities

The analysis assumed an operational capacity for the WTWs equal to 80% of the Maximum Design Capacity (MDC) to allow for down time required for maintenance and repairs. However, the Current Operating Capacities (COC) of Rustfontein and Maselspoort WTWs, as configured in both the 2022/23 AOA water resources planning model and the 2018 Mangaung Study, are equivalent to the MDC. Therefore, the analysis applied the larger value of COC or 80% of MDC as the operational capacity.

For scenarios that included the “bi-directional” pipeline intervention, the maximum capacity needed to be pumped from the Welbedacht WTW. The modelled operational capacity was thus increased from 116 Mℓ/d (1.344 m^3/s; 42 million m^3/a) to 137 Mℓ/d (1.595 m^3/s; 50 million m^3/a) for the relevant scenarios.

For the scenarios where Gariep Dam supplied raw water to the GBWSS, the Rustfontein and Maselspoort WTWs were modelled to operate as close to MDC as possible. The modelled operating capacities were increased from 100 Mℓ/d (1.157 m^3/s; 36.5 million m^3/a) to 133 Mℓ/d (1.538 m^3/s; 48.5 million m^3/a) and from 110 Mℓ/d (1.273 m^3/s; 40.2 million m^3/a) to 114 Mℓ/d (1.315 m^3/s; 41.5 million m^3/a) respectively for the relevant scenarios.

3.2.1.3 Modelled Future Upstream Developments

In this analysis, future water resources developments primarily focused on the expansion of water infrastructure, i.e. dams, in the upper Caledon catchment, along with anticipated increases in urban and irrigation demands along this reach of the Caledon River. These considerations were pertinent for scenarios examining the impacts of future development levels, namely 2035 and 2050.

Planned water resources developments in the upper Caledon catchment entail the construction of Hlotse and Ngoajane dams on the Caledon River upstream of Tienfontein (see Figure 2-1). Given the typical delays associated with the planning and execution of large dam projects, Ngoajane Dam was excluded from the analysis of the 2035 development level. It was only considered in scenarios projecting to 2050. Conversely, it was assumed that Hlotse Dam would be completed by 2035 and this was included in the 2035 and 2050 projections.

In allocating water resources from Hlotse and Ngoajane dams, it was decided to assign their full yields to meet the demands projected for 2050.

3.2.2 WRYM Analysis

3.2.2.1 Scenario Definition

The water resources analyses undertaken as part of this study entailed yield analyses for various scenarios relating to the supply of bulk water to the GBWSS. As described in Sections 1.3 and 1.4, the water resource analysis and infrastructure sizing and optimisation was an iterative process. Initially, historic firm yield (HFY) analyses were carried out for the four scenarios (Figure 1-2) and the results were used to evaluate the different Xhariep Pipeline transfer schemes/options. Once the best raw transfer option was identified, this option along with the potable transfer option were taken forward into further optimisation and analyses. Finally, a stochastic yield analysis was performed on the best scheme/option to confirm the adequacy of the proposed sizing of the infrastructure in terms of meeting the projected 2050 GBWSS demand at the required 98% assurance of supply. A flowchart of the modelling process is shown in Figure 3-24, which must be read in conjunction with the flowchart shown in Figure 1-3 (Note – the economic evaluation of options is presented in Chapter 7). Phase 3 of the WYRM, which evaluates the impact of increased abstraction from Gariep Dam on the downstream users, is described only in the Water Resource Analysis Report that forms part of the study as its findings don’t influence the infrastructure sizing and design addressed in this report.

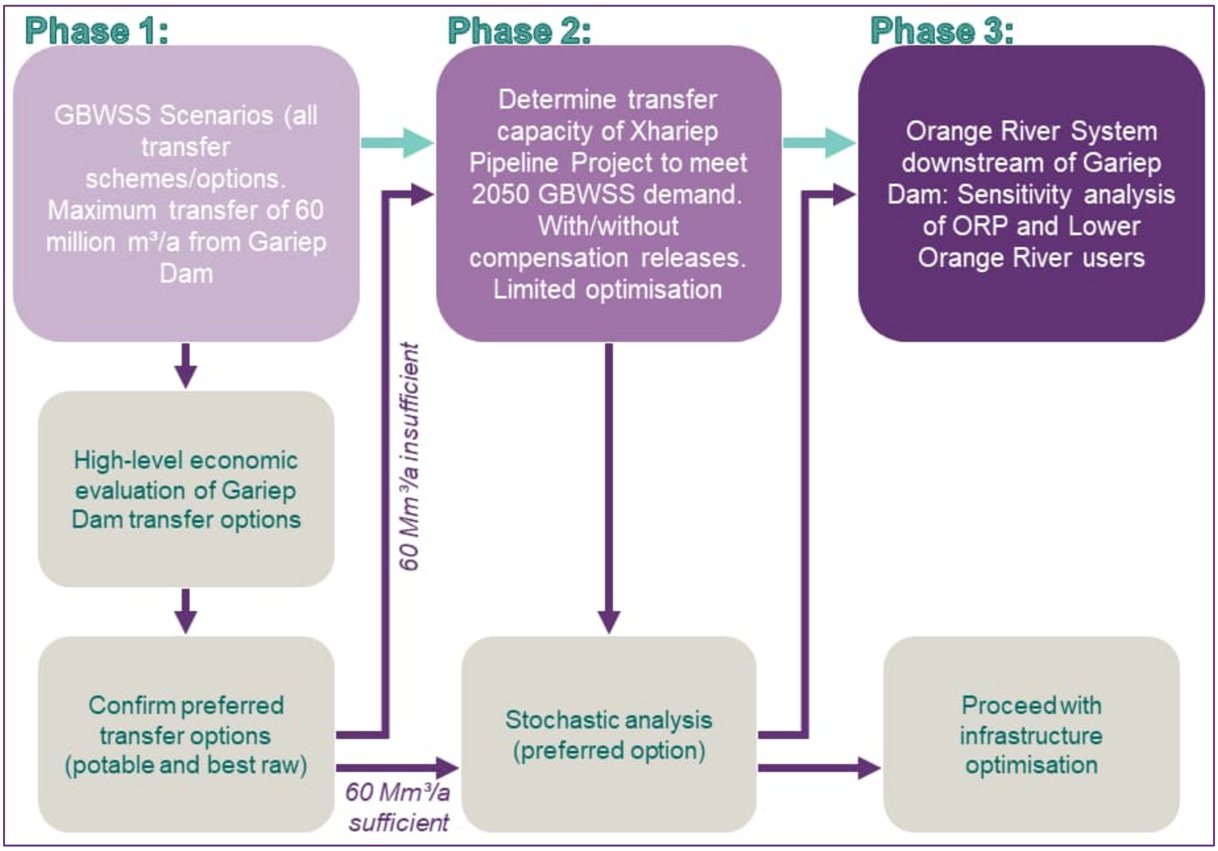


Figure 3-24: Water resource phases for analysis

3.2.2.1.1 Phase 1 Scenarios

Phase 1 scenarios evaluated the yield of the current system, the potential benefits of implementing interventions identified in the DWS Reconciliation 2012 study, the potential yield increase due to a maximum abstraction rate of 60 million m^3/a from Gariep Dam, and the impacts of planned water resources developments and increased abstractions in the upper Caledon catchment upstream of the existing Tienfontein PS abstraction to the GBWSS. The scenarios assumed that no compensation releases from the proposed Hlotse and Ngoajane Dams would be made to offset the impact that these dams would have on the yield of the downstream system/users.

Each scenario had sub-scenarios, and additional subsets of the sub-scenarios. The sub-scenarios and the subsets are detailed in the Water Resource Analysis Report. This report contains a high-level summary of the analysis, while a summary of the scenario descriptions are presented in Table 3-24.

Table 3-24: Phase 1 water resource analysis scenario description summary

Scenario	Description
Scenario A	Validate the "new" WRYM configuration specifically configured for this assignment and establish if it correlates with the recent analyses undertaken as part of the 2018 Mangaung Study.
Scenario 1	Determine historic firm yield (HFY) of the current bulk water supply system, including the raising of existing dams.
Scenario 2	Determine the impact of possible future water resources developments and increased water demands in the upper Caledon catchment on the HFY of the current GBWSS.
Scenario 3	Determine the potential increase in the current system yield through implementation of previously identified interventions as well as additional interventions.
Scenario 4	Determine further potential increase in the system yield through implementation of the Gariep Dam to Bloemfontein pipeline.

Scenario	Description
Scenario 5	Determine the impact of possible future water resource developments and increased water demands in the upper Caledon catchment on the maximum supply capacity (Yield) of the future GBWSS.

3.2.2.1.2 Phase 2 Scenarios

Following the determination of the Phase 1 scenarios, Phase 2 scenarios determined the transfer capacity from Gariep Dam, based on historic firm yield (HFY), needed to meet the 2050 demand of the GBWSS while also investigating (at a high level) the order of magnitude of and the yield sensitivity to compensation releases from the planned dams in the upper Caledon catchment. Phase 2 analysed the best raw water Gariep Dam transfer option, Scheme 4, (based on the outcome of Phase 1 infrastructure sizing and costing) along with the potable Gariep Dam transfer option, Scheme 1. Finally, the potable option (which emerged as the best transfer option) was taken forward into stochastic yield analysis to confirm the adequacy of the proposed sizing of the infrastructure to meet the projected 2050 GBWSS demand at the required assurance of supply. Phase 2 also involved optimisation of the operating rules for the preferred transfer scheme.

Like Phase 1, Phase 2 scenarios included sub-scenarios and subsets of the sub-scenarios. Table 3-25 summarises the scenarios modelled in Phase 2 of the water resource analysis.

Table 3-25: Phase 2 water resource analysis scenario description summary

Scenario	Description
Scenario 5	(continued from Phase 1) Determine the impact of possible future water resource developments / increased water demands in the upper Caledon catchment on ORP Yield.
Scenario 6	Determine the impact of both Lesotho Lowland dams on GBWSS and ORP yield. The impact should be “re-balanced” through compensation releases.
Scenario 7	Determine the increased transfer capacity (potable and raw water) when the GBWSS yield is sufficient to support the projected 2050 GBWSS demand. No compensation releases made from Lesotho Lowlands dams.
Scenario 8	Determine the increased size of both Hlotse and Ngoajane dams to increase the yield from these two dams so that they can supply the required compensation releases.
Scenario 9	Determine the impact of both Lesotho dams with compensation releases on the GBWSS and on the ORP.
Scenario 10	Determine increased transfer capacity with compensation releases from the Lesotho Lowland dams until the GBWSS yield is sufficient to support the projected 2050 GBWSS demand.
Scenario 11	Select the final Scenario to carry out a long-term stochastic yield analysis on the GBWSS.

3.2.2.2 Scenario Analysis Results

The modelling results for each scenario and its subsets are detailed in the Water Resources Analysis Report. A summary of the results is provided in the sections below.

3.2.2.2.1 Phase 1 Results

The modelling results of Phase 1 shows that the potable yield of the GBWSS continually reduces over time due to the increased domestic and irrigation developments in the upper Caledon catchment within South Africa as well as Lesotho. For comparison, the 2012 Reconciliation Strategy modelled a yield of 84 million m^3/a, the 2018 Mangaung Study modelled a yield of 73 million m^3/a, and this analysis modelled a yield of 65 million m^3/a.

Scenario 3 emphasised the need to implement the interventions as detailed in the 2012 Reconciliation Strategy as the yield will increase from 65 million m^3/a to 101 million m^3/a, a 36 million m^3/a increase. Three interventions contributed to the increase in yield, namely the bi-directional pipeline between Welbedacht WTW and Knellpoort Dam (+16 million m^3/a), the wastewater reuse and raising of Mockes Dam (+12 million m^3/a) and increasing the capacity of Tienfontein pump station to 7m^3/s (+8 million m^3/a). Demand reduction as a result of WC/WDM was considered in the demand projections and not as an intervention that would add to the potable yield of the GBWSS.

Scenarios 4 and 5 examined the impact of the four Xhariep Pipeline schemes/options, with the transfer from Gariep Dam capped at 60 million m^3/a, on the available yield after implementation and the impact of future upstream developments if all intervention options are implemented. Adding the 60 million m^3/a transfer increased the yield to 161 million m^3/a. However, this reduces due to the future upstream developments to 145 million m^3/a in 2035, and 131 million m^3/a in 2050.

The Phase 1 modelling results show:

- The 2012 Reconciliation Strategy interventions contribute to a significant increase in the GBWSS potable yield,
- A Xhariep Pipeline transfer from Gariep Dam is necessary to satisfy 2050 demands, and
- A transfer volume of 60 million m^3/a is insufficient to meet the projected 2050 demands of 186 million m^3/a as calculated in Section 3.1.

The initial infrastructure sizing and cost comparison was completed using the Phase 1 modelling results. This allowed the potential schemes/options to be narrowed down to Schemes 1 and 4, i.e. the potable and best raw water scheme, for further analysis. See Section 7.4 for a detailed description of the initial infrastructure sizing and cost comparison.

Phase 2 of the water resource analysis was needed to understand the impact of compensation releases from the Hlotse and Ngoajane dams and to determine the required transfer volume for Schemes 1 and 4 after which a stochastic analysis was performed to confirm the transfer volume at the required assurance of supply.

3.2.2.2.2 Phase 2 Results

The Phase 2 scenarios analysed the impact of the Hlotse and Ngoajane dams, that are planned in the upper Caledon River reach/Lesotho Lowlands on the GBWSS and Orange River Project (ORP) yields. The dams reduce the GBWSS yield by 14 million m^3/a and the ORP yield by 46 million m^3/a. If the dams' capacities were increased in size to accommodate compensation releases and still meet the demands imposed on these two dams, the reduction in yield on the GBWSS and ORP would be 2 million m^3/a and 36 million m^3/a respectively. The modelling showed that it was not possible to fully rebalance the impact on the yield caused by these dams, unless the demands imposed on these dams are reduced.

The next step in the Phase 2 modelling compared the required transfer volume of the potable (Schemes 1A and 1B) and best raw water option (Scheme 4B) with and without compensation releases. The results showed that the initial transfer volumes, to meet 2050 demands, of the potable schemes were 120 million m^3/a and 139 million m^3/a respectively, and 142 million m^3/a for the raw water scheme. The second round of infrastructure sizing and cost comparison was completed with these modelling results. From the comparison exercise detailed in Section 7.5, Scheme 1B was determined to be the preferred option and was taken forward for further analysis.

Further refinement of the operating rules reduced the required transfer volume of Scheme 1B to 101 million m^3/a, assuming that no compensation releases were made from the Lesotho Lowlands Dams and 95 million m^3/a with compensation releases. Assuming that all interventions are implemented, including the Xhariep Pipeline, a total transfer volume of 101 million m^3/a will increase the GBWSS yield to 186 million m^3/a, which matches the projected 2050 demands. Scheme 1B infrastructure sizing was based on no compensation releases which is a 6% higher volume than the transfer with compensation releases.

The final stage of the Phase 2 analysis was to confirm the transfer capacity to ensure at least a 98% assurance of supply. A long-term stochastic analysis was undertaken on the GBWSS including the proposed Scheme 1B, assuming no compensation releases. The analysis confirmed that Scheme 1B can meet the 2050 GBWSS demand at an assurance level higher than 1:50 year (98% assurance of supply), as indicated in Figure 3-25. Figure 3-25 shows the outcome of the stochastic analysis, which indicates that yields of approximately 220 million m^3/a and 213 million m^3/a can be delivered at 98% and 99% assurance of supply, respectively. It is recommended that the maximum transfer volume from Gariep Dam remains at 101 million m^3/a as the higher assurance of supply provides flexibility should additional towns be included in future as part of the GBWSS or to cater for any unforeseen delays experienced with the implementation of any of the 2012 Reconciliation Strategy interventions.

3.2.3 WRYM Discussion

The study confirmed that the system yield is highly sensitive to the applied operating rules, prompting a subsequent optimisation of these rules as part of the scenario analyses. These optimisations are deemed adequate for the feasibility study's objectives.

It is important to note that MMM has implemented water restrictions since 2014, which may have influenced behavioural changes among residents regarding water usage. Furthermore, the implementation of WC/WDM may lead to further behavioural adjustments. Therefore, regular updates to and sense-checking of future water demand projections are necessary.

South Africa should prioritise the negotiation of compensation releases from future Lesotho Lowland dams to prevent a significant reduction in the GBWSS yield in the future.

Immediate implementation of interventions identified in the 2012 Reconciliation Strategy is of utmost importance to increase the GBWSS yield and/or reduce system demands, considering that the current system demand already exceeds the system yield. Priority intervention options include the bi-directional pipelines between Welbedacht WTW and Knellpoort Dam, increasing Tienfontein pump station capacity to 7 m^3/s, and exploring re-use options.

In addition to the Xhariep Pipeline Project, it is crucial to consider other operational recommendations from previous studies, such as those outlined in the 2022/2023 AOA:

- Continuous effort to implement WC/WDM to reduce losses,
- Address cavitation problems of submersible pumps at Rustfontein Dam to utilise the system storage fully,
- Conduct maintenance and repairs of Tienfontein pumps to ensure maximum pumping capacity availability,
- Ensure the maximum possible transfer from Welbedacht Dam/WTW to the GBWSS with on-going and high-quality maintenance,
- Optimise the supply from Mockes Dam to reduce spills from Maselspoort Weir, and
- Rectify losses through the sluice gates at Welbedacht Dam.

Review and on-going refinement of operating rules for the final selected option should be carried out in the future. The AOA study for the ORP and GBWSS systems could serve this purpose, including the optimisation of pumping costs for selected scenarios. Additionally, addressing the intricacies of water supply at a reservoir and reticulation network level should be part of Bulk Water Master Planning undertaken by end-users, such as MMM and VCWB.

It is advisable to include a scenario in the next ORS AOA that incorporates the proposed transfer option from Gariep Dam to the GBWSS. This would provide insight into the potential benefits of this scheme regarding the necessity and severity of water restrictions, as well as assurance of supply.

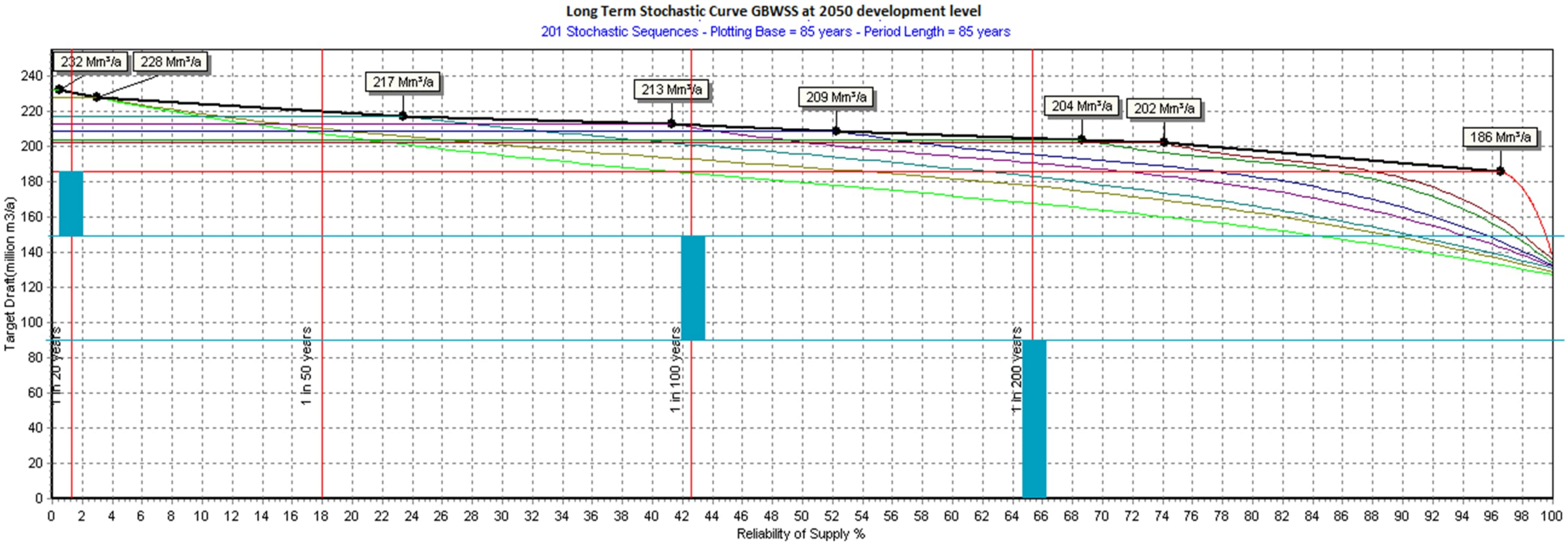


Figure 3-25: Long-term stochastic curve of the GBWSS at 2050 development level

4 Water Quality Analysis

The original objectives of the water quality assessment were to:

- Characterise the raw water quality that could potentially be abstracted at Gariep Dam, and,
- Estimate the quality of water blended from different sources at Knellpoort Dam and Rustfontein Dam, that could affect its treatability at the WTWs, if applicable.

The latter requirement is however, not further considered as part of this report. The feasibility of transferring raw water from Gariep Dam to Knellpoort Dam, the Novo Outfall or Rustfontein Dam appears to be less viable when compared to the option of transferring potable water from Gariep Dam to the GBWSS as described in Sections 3.2 and 7.6. However, in the event of this changing in future, the impact of blended water quality on the WTW located at the impacted dam can be evaluated during the detailed feasibility phase.

The discussion in this chapter is therefore focused on the raw water quality record of the Gariep Dam.

Historical raw water quality data for the Gariep Dam is available for the periods 2004 to 2017 and 2018 to 2022. The data was sourced from the DWS and VCWB respectively. The DWS data set consists of 564 samples drawn in intervals varying from hours to months. The VCWB data set consists of 75 samples drawn typically on monthly intervals. Each sample was analysed for several determinants. This resulted in each significant determinant being measured in the order of 70 to 220 times over the period. The exact count of samples per determinant is indicated in Table 4-1.

An analysis was performed on the data focusing on a comparison of the data to the South African drinking water quality specification (SANS241-1, 2015) as well as WHO guidelines (World Health Organisation, 2017). The analysis served to highlight determinants that do not meet the drinking water quality standard and that justify treatment interventions.

Based on the available water quality data only, the water can be described as of very good quality. Turbidity, microbiology, and stability are the only determinants requiring particular attention. The presence of suspended solids can inhibit the effectiveness of disinfection. Harmful micro-organisms (like viruses and bacteria) may adsorb to the surfaces of solid particles and by doing so, acquire protection from the active species involved in disinfection, which cannot reach the micro-organism to deactivate it. To facilitate effective disinfection, the total suspended solids of the water must be sufficiently reduced.

The absence of certain parameters from the available dataset is a concern. No algal bloom related data (e.g., chlorophyll-a, taste and odour, algal toxins, etc.) or data related to Contaminants of Emerging Concern (CECs), was available for the water abstracted into the WTW. A study (Venter, 2000) reported that the average chlorophyll-a concentration ranged between 0.4 and 1084 µg/l, with an average concentration of 10.8 µg/l. The high value was measured in February 1999 but was reported by the author as an exceptionally high value. The box plots in Figure 4-1 summarise the chlorophyll-a concentrations at various points along the upper Orange River catchment (up to the year 2000). The data from the Gariep Dam was of particular interest to this study. The peak levels are of concern and may require the addition of flotation to the treatment process.

Some DWS chlorophyll-a data was available for water sampled near the dam wall. The measured levels were sufficiently high at times to justify a dedicated treatment step. It however remains uncertain how much of the algal load will be drawn into the WTW. Levels of chlorophyll-a in the abstracted water must be determined as part of the feasibility study to further evaluate the treatment processes proposed in this report (Section 7.1).

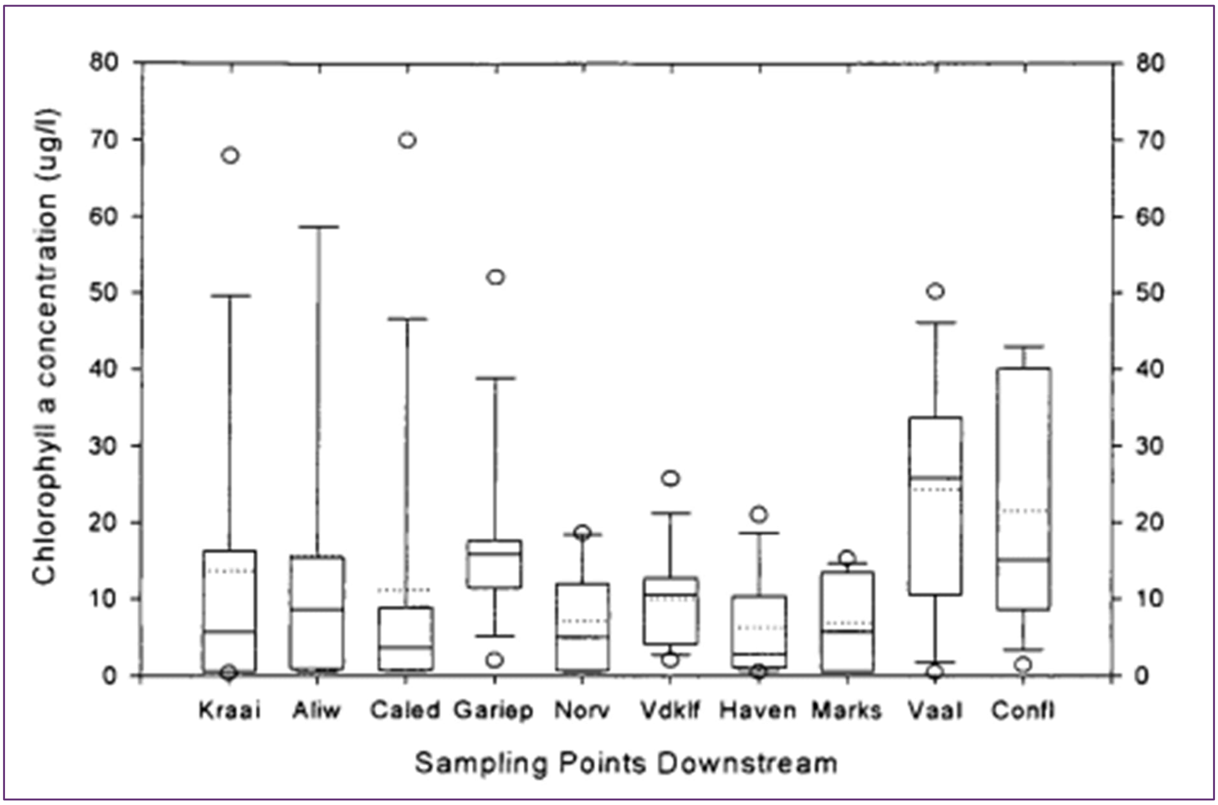


Figure 4-1: Box plots of the chlorophyll-a concentrations at different sampling sites in the Orange River system for an extended period (not specified) (Venter, 2000)

The raw water characteristics for the feed water from the Gariep Dam are provided in Table 4-1.

Table 4-1: Expected raw water characteristics

Parameter	Units	No. of analyses	5th percentile Raw Water Operational Data	50th percentile Raw Water Operational Data	95th percentile Raw Water Operational Data	SANS 241: 2015 and DWS/WHO Standards
Turbidity	NTU	50	***12.93***[1]	***32.96***	***164.20***	≤ 1
Colour [2]	mg/ℓ as Pt	12	1.00	1.00	***432.00***	≤ 15
TDS	mg/ℓ	50	97.93	124.20	150.86	≤ 1200
Conductivity	mS/m	216	13.19	16.00	21.08	≤ 170
pH	[-]	216	6.44	7.52	8.23	≥ 5 to ≤ 9.7
Total Alkalinity	mg $CaCO_3$/ℓ	216	46.78	68.70	85.53	~40-120
Fluoride	mg/ℓ	211	0.05	0.17	0.30	≤ 1.5
Ammonia	mg/ℓ	213	0.02	0.04	0.15	≤ 1.5
Potassium	mg/ℓ	213	0.99	1.44	2.52	≤ 50
Sodium	mg/ℓ	213	4.06	5.52	7.58	≤ 200
Zinc	mg/ℓ	99	0.00	0.00	0.19	≤ 5
Calcium	mg/ℓ	213	14.06	17.48	22.29	≥ 16
Iron	mg/ℓ	144	0.05	0.32	***3.83***	≤ 0.3
Manganese	mg/ℓ	145	0.00	0.02	0.05	≤ 0.1
Magnesium	mg/ℓ	213	4.72	6.20	7.59	≤ 30

Parameter	Units	No. of analyses	5th percentile Raw Water Operational Data	50th percentile Raw Water Operational Data	95th percentile Raw Water Operational Data	SANS 241: 2015 and DWS/WHO Standards
Chloride	mg/ℓ	213	1.50	3.70	5.92	≤ 300
Nitrate as NO^3 - N	mg/ℓ	215	0.16	0.50	1.00	≤ 11
Nitrite as NO_2 - N	mg/ℓ	16	0.01	0.01	0.01	≤ 0.9
Sulphate as SO_4^{2-}	mg/ℓ	213	2.00	8.90	20.92	≤ 250
Phosphate PO_4	mg/ℓ	210	0.01	0.03	0.10	
Calcium Hardness (calculated from above)	mg/ℓ as $CaCO_3$	213	35.15	44	55.73	
Magnesium Hardness (calculated from above)	mg/ℓ as $CaCO_3$	163	15.96	20.91	25.39	
Total Hardness (calculated from above)	mg/ℓ as $CaCO_3$	163	53.64	68.66	84.05	≤ 150
Langelier Index (calculated from above)	-	161	***-2.06***	***-0.60***	***-0.03***	~ 0
Ryznar Index (calculated from above)	-	161	***8.22***	***8.94***	***10.53***	6.5 – 7.0
Escherichia coli	MPN or CFU per 100 Mℓ	41	***0.00***	***3.00***	***32.60***	0
Heterotrophic plate count (HPC)	CFU	48	12.50	122.00	985.25	≤ 1000
Total coliforms	CFU	49	2.00	***101.00***	***1414.00***	≤ 10
Calcium Carbonate Precipitation Potential (CCPP) (calculated from above)	mg $CaCO_3$/ℓ		No data	No data	No data	2 to 5
TOC	mg/ℓ	49	2.62	4.01	5.73	10
DOC	mg/ℓ	92	1.81	3.73	***19.33***	10
Chlorophyll-a[3]	µg/ℓ	172	0.28	1.25	15.45	<10

Notes:

1 Figures shown in red do not meet the required standards

2 These values seem to be colour values and not true colour.

3 The sample point is near the dam wall and not in the raw water feed line to the plant. The data must therefore be used as an indicator only.

In addition to the conclusions already drawn above, the following was noted from the available data:

- There were only 12 data points for colour in the raw water. Three of these data points were approaching 200 mg/l Pt and above with the highest result being 707 mg/l Pt. The balance of the data points all equal 1 mg/l Pt. These results appear to be incorrect and require further evaluation.

- Few results were generally acceptable. There was however a number of exceptionally high values reported. It was unclear from the data reports if the numbers represent total or dissolved Fe levels. Further evaluation of this parameter is required.
- The data reflects the quality of untreated water, and the microbiological results are therefore not a significant concern. These numbers are expected to reduce significantly when the water is subjected to standard treatment protocols.
- A small number of datapoints reflected very high dissolved organic carbon levels. All the high values were however reported prior to 2003. Subsequent reports all indicated DOC levels below the national standard.

A raw water sampling and analysis programme will be undertaken as part of further design studies. A series of 12 sampling rounds at 2-week intervals are planned for the feasibility study.

Initially, a comprehensive list of parameters will be analysed as required by the water treatment team (and specified in SANS 241 of 2015), and for the remaining duration of the feasibility study, a reduced list of parameters will be analysed. It is anticipated that comprehensive analyses will be conducted on at least one summer season sample and on one winter sample. Data will be collected over one seasonal cycle to cover at least a period of thermal stratification. All sampling, and sample transport will adhere to SANS 5667 (Water Quality - Sampling) standards and guidelines. All testing of the samples will be covered by the provisional sum allowed for in the Contract.

5 Geotechnical Investigations – Desktop Level

This chapter describes the geotechnical considerations with the associated potential construction risks, based on a desktop assessment, for the four proposed pipeline schemes (Figure 1-2) under consideration. The results of which provide input into the scheme comparison.

The following information was used:

- The 1:250 000 published geological maps,
- Satellite imagery,
- Published information and previous study reports on typical geotechnical conditions associated with the various rock types,
- Zutari reports on investigations in areas with similar geology and climate, and,
- Reports compiled by others (e.g., DWS, VCWB, MMM, etc.) as part of previous studies, where readily available.

5.1 Site Characterisation

5.1.1 Site description

The proposed pipeline routes traverse the flat to gently rolling sections of the Free State province and are mostly aligned with existing roads. Location details for each of the options are summarised within Table 5-1.

Table 5-1: Summary of general pipeline route layout

Pipeline	Start position	End Position	Route length (km)	Neighbouring roads
Scheme 1	Gariep Dam 30°3'"37.60"S' 25°2'"21.35"E	Bloemfontein 29°1'"19.93"S' 26°1'"44.50"E	181	R701, N1, Ferreira
Scheme 2	Gariep Dam 30°3'"37.60"S' 25°2'"21.35"E	Novo outfall 29°3'"29.53"S' 26°°'3'1.14"E	198	R701, N1, R715, N6
Scheme 3	Gariep Dam 30°3'"37.60"S' 25°2'"21.35"E	Knellpoort Dam 29°4'"48.15"S' 26°°'3'4.80"E	190	R701
Scheme 4	Gariep Dam 30°3'"37.60"S' 25°2'"21.35"E	Rustfontein Dam 29°1'"39.53"S' 26°3'"36.33"E	214	R701, N1, R702

All the proposed pipeline options traverse existing road and rail infrastructure which will be affected during construction of the pipeline.

Elevation profiles for each of the pipeline options were created using satellite imagery. An estimation of the general topographical trends is shown in Figure 5-1.

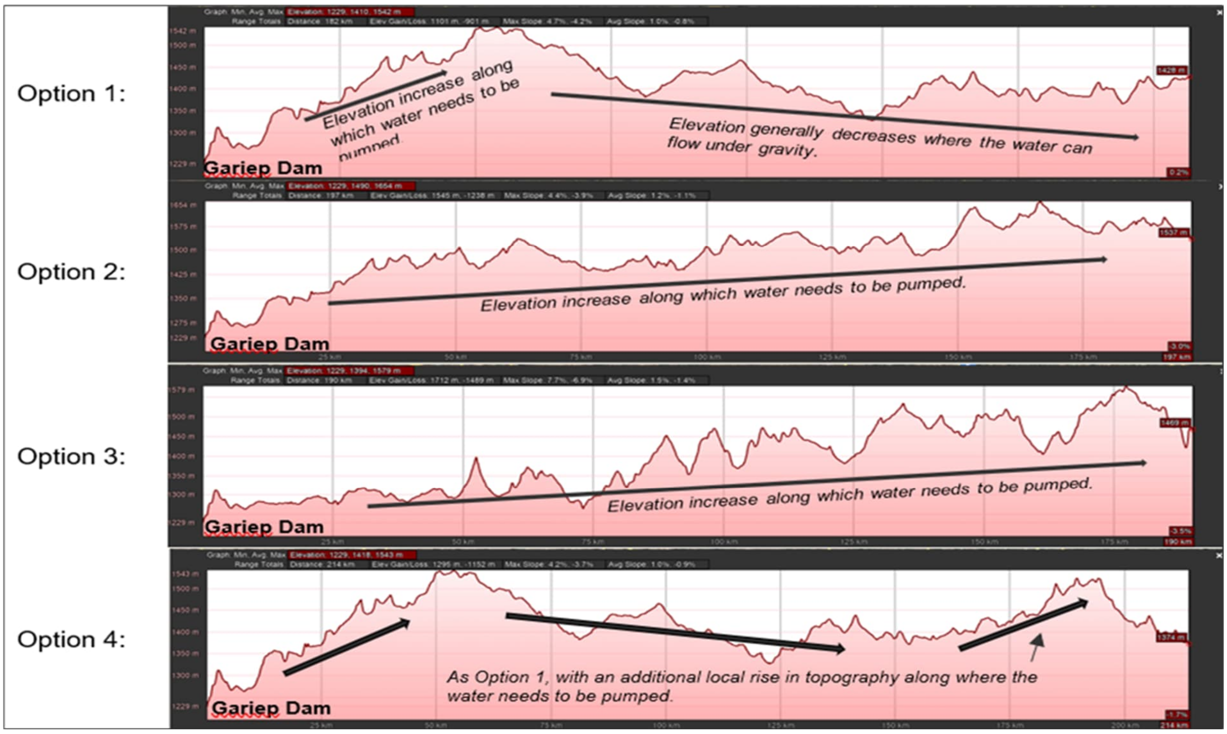


Figure 5-1: Sections of general topography along the pipeline routes

5.1.2 Regional Geology

According to the 1:250 000 geological maps, all the proposed pipelines are underlain by interbedded mudstone and sandstone of either the Adelaide or Tarkastad Subgroups within the Beaufort Group, Karoo Supergroup. According to Brink (1983) the Beaufort Group sandstone is generally thin and poorly sorted. Additionally, free swell ranging from 0.01% to 7.0% was reported by Oliver (1976) for samples of fresh Beaufort mudstone.

The Karoo rocks have been extensively intruded by dolerite dykes and sills. Most dolerite intrusions in the Karoo Supergroup are horizontal (sills) with a thickness that varies from 1 m to about 300 m. Solid unweathered dolerite is hard to extremely hard and generally requires blasting for excavation. Dolerite varies in weathering resulting in boulders, gravels, granular (sugar) dolerite and eventually to residual dolerite soil.

The regional geology is shown in Figure 5-2.

5.2 Seismicity

According to the seismic hazard map in SANS 10160-4:2017, the proposed schemes are within Zone 2 that experiences seismicity with a peak ground acceleration in the order of 0.1g - 0.15g due to mining activities and natural movement. There is a 10% probability that the expected ground acceleration could be exceeded within a 50-year period (Figure 5-3). Note that this is based on published SANS data and no site-specific studies have been carried out.

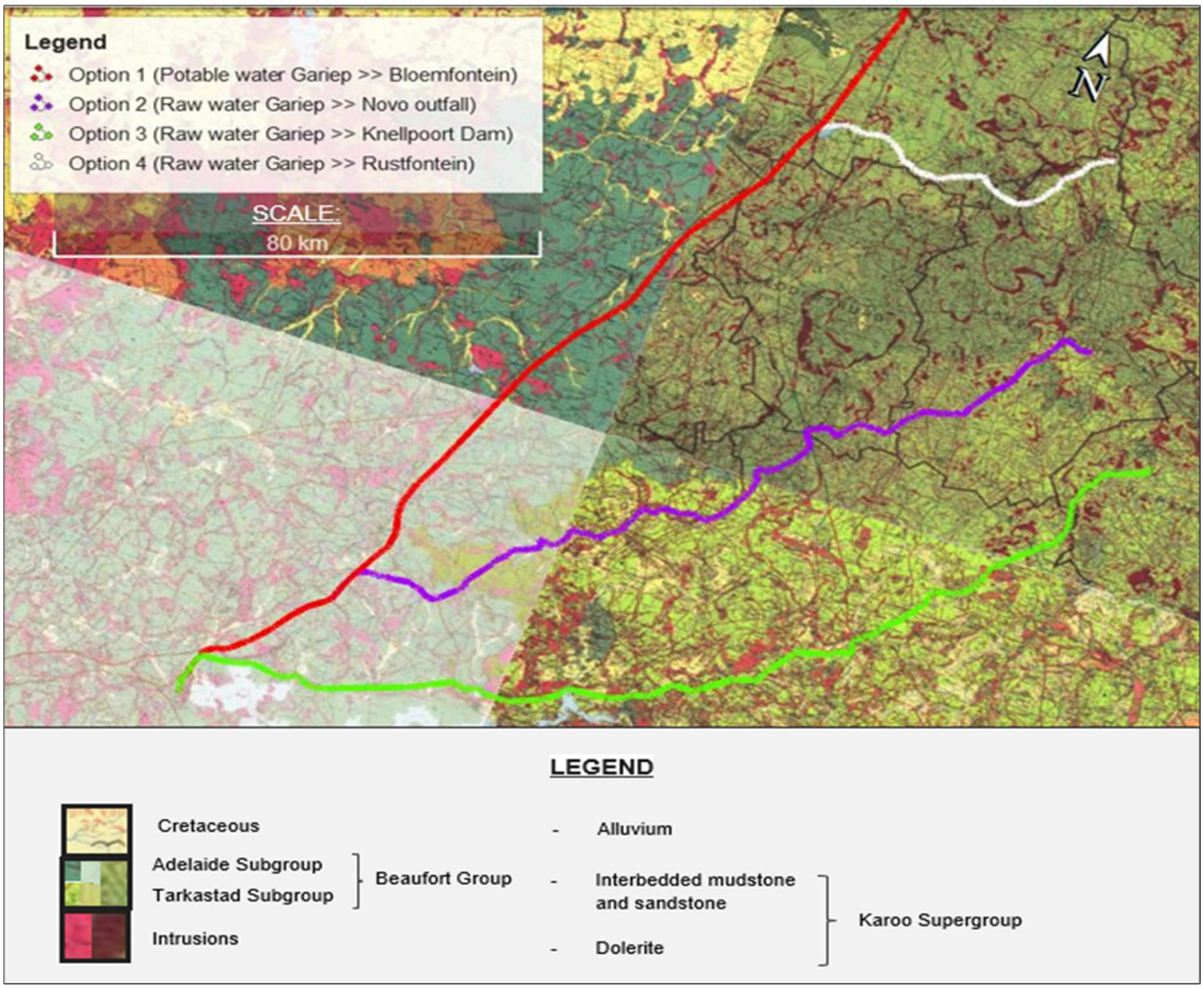


Figure 5-2: Regional Geology of the site (1:250 000 Geological maps as listed, published by the Council for Geoscience)

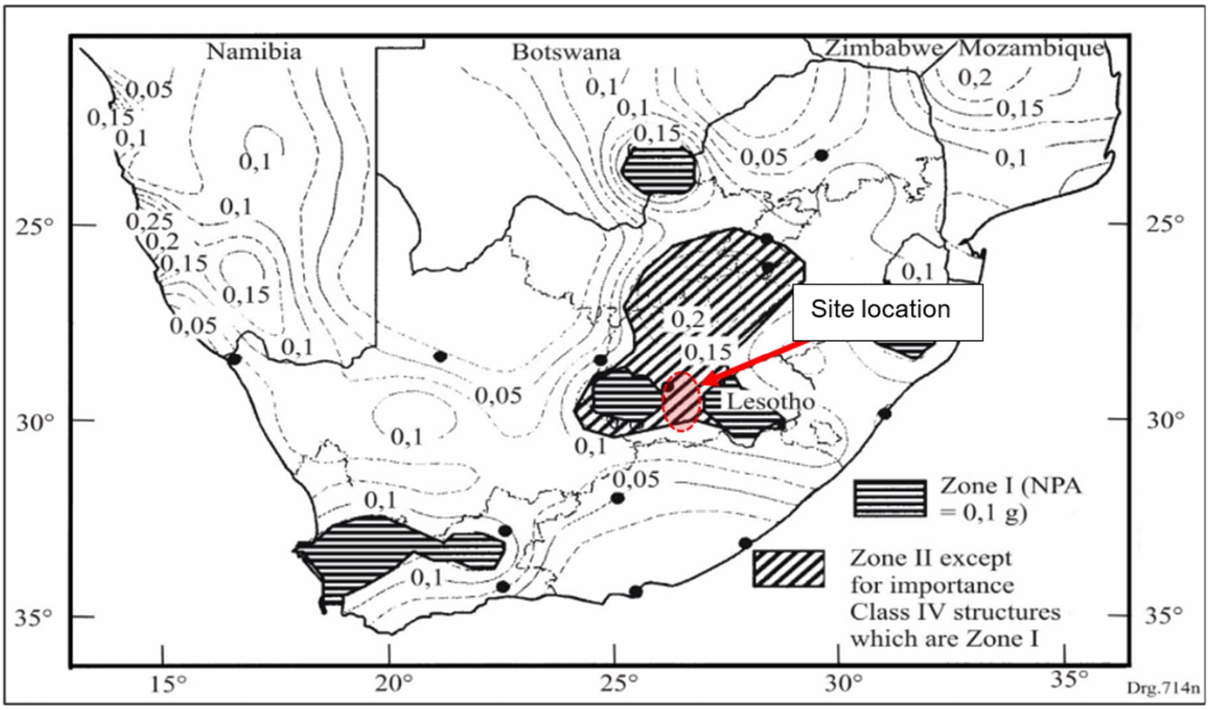


Figure 5-3: Seismic hazard map showing peak ground acceleration (g) with 10% probability of being exceeded in a 50-year period (SANS 10160-4: 2017)

5.3 Weathering and Soils

The climatic conditions largely determine the rate and mechanism of weathering of natural rock and thus the resulting residual soil profile. The Weinert N-value groups climatic regions to areas of similar moisture patterns, and the likely weathering mechanisms that can be expected. Figure 5-4 shows the project location in relation to the macro-climatic regions of Southern Africa. The project area is situated in moderate to dry conditions, with the Weinert N-value between 2 and 5 in the east, becoming more than 5 towards the west. A combination of mechanical disintegration and chemical alteration as rock weathering mode is expected over the area. The schemes thus traverse thicker well-developed residual soil profiles along the east, becoming shallow and weakly developed towards the west.

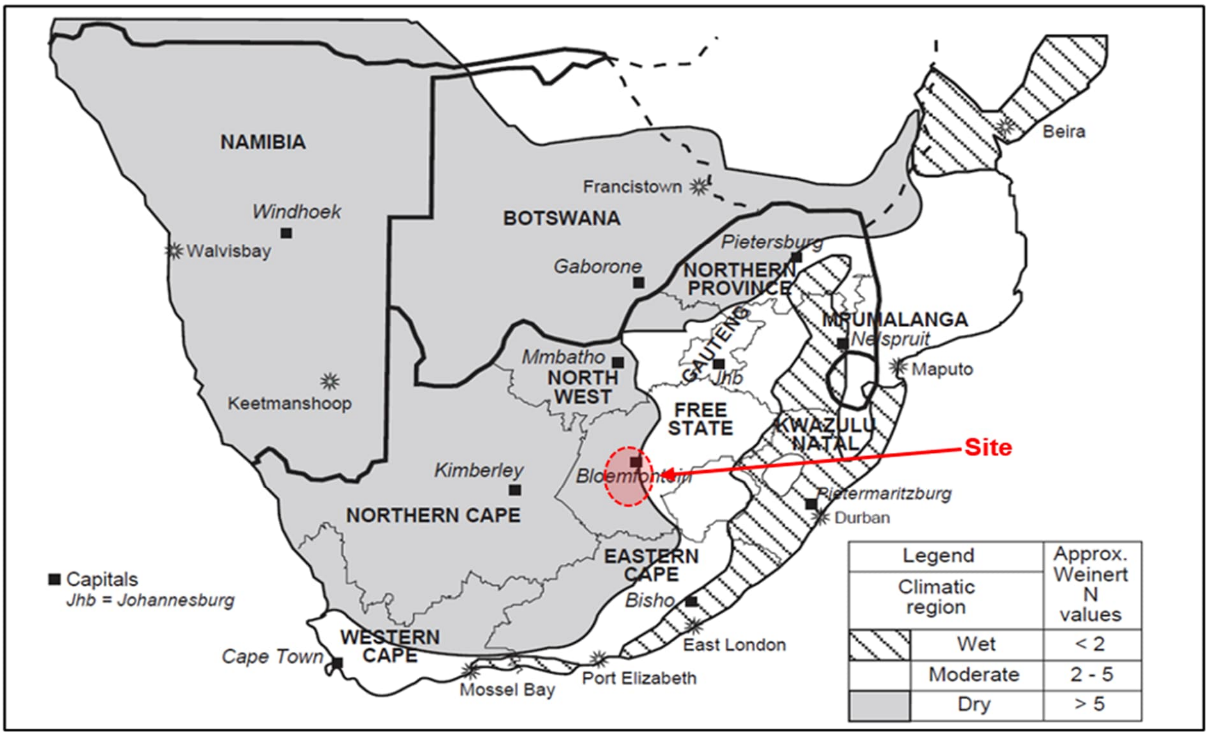


Figure 5-4: Site location with respect to macro-climatic regions of Southern Africa (after Weinert, 1980)

5.4 Geotechnical Considerations

5.4.1 Excavation Conditions

Soft to intermediate excavation conditions (in line with SANS 1200 DA-1998) to depths exceeding 2 m are anticipated in the east that become shallower towards Gariep Dam. Localised areas that are underlain by dolerite or traversed by rivers may have boulders within the soil profile that may require "boulder excavation". An excavator with power tools will be required for the excavation of Intermediate material.

Additionally, there is a possibility that shallow refusal could be encountered on hard to very hard rock along sections underlain by dolerite. Blasting may be required to reach the pipeline invert level.

5.4.2 Expansive Soil and Rock

Laboratory test results of residual mudstone of the Beaufort Group and alluvium samples within the area generally showed the material to be low to medium expansive according to Van der Merwe's method (Van der Merwe, 1964). The mudstone bedrock itself is also known to be expansive.

The magnitude of cyclic movement due to wetting (heave) and drying (shrink) that will affect the pipeline decreases with increasing depth below surface up to a level where no volumetric change occurs (the moisture – stable zone). The depth of this zone varies depending on the climatic pattern of the region.

The moisture equilibrium depth in the area is generally between 1 m and 1.5 m, potentially reducing the effects if the pipeline invert is deeper than 1.5 m below surface. However, should shallower excavations be required over mudstone or within alluvium, cyclic movement due to heave and shrinkage is anticipated to be the main geotechnical concern. Here, consideration should be given to undercutting the pipeline to a depth of 1.5 m below surface (up to the moisture-stable zone) and replacing the in-situ material with non-expansive material that is compacted to at least 93% Mod AASHTO density. It is also recommended that the depth of the moisture-stable zone along the selected pipeline route be verified by means of test pitting.

5.4.3 Corrosivity

Samples taken of residual mudstone and dolerite soil on other projects generally tested as weakly aggressive to aggressive for steel. Locally, the aggressiveness of the environment will also be influenced by stray currents of the electrified rail infrastructure.

Allowance should be made to provide protection for the proposed pipeline due to the risk of an aggressive environment. The corrosivity of the soil along the pipeline will be confirmed during the detailed feasibility geotechnical investigation.

5.4.4 Stability of Excavations

Test pits excavated on other sites within the area were generally described as stable. It should however be noted that some of these pits were shallow, and all the pits were only open for a limited time. Additionally, shallow groundwater seepage may be encountered in areas along rivers and dams.

Should deep excavations be required, shoring, or battering of excavation faces must be considered to ensure safety during construction, especially if these excavations are created during the rainy season. It is recommended that the excavated face be inspected by a competent engineering geologist or geotechnical engineer to confirm the stability of excavations.

5.4.5 Crossing of Streams and Areas with Alluvial Material

There is a risk that flowing streams and seasonal flooding will cause the bedding and cradle material to be washed away and thus cause problems with the structural support. Encasing the pipeline in properly supported concrete across the span of these areas should be considered.

5.4.6 Crossing of Roads and Railway Lines

Pipe-jacking will be required at railway lines and at major road crossings. Excavation and replacement of the road (with temporary traffic diversion) may be used at other local road crossings, which will depend on traffic demand and local authority guidelines.

There is a risk that shallow hard rock dolerite may be found that will make jacking very difficult. Micro-tunnelling can be considered depending on the pipe diameter.

5.4.7 Bedding Materials

In addition to the expansive nature of the soil along the route covering Karoo rocks, it is anticipated to encounter more than the recommended amount of fines in line with SANS 1200 Section LB. This material would most likely not be suitable for either cradle or bedding material. Material will need to be imported to the site and commercial sources will therefore need to be considered.

5.5 Geotechnical Conclusions

This chapter contains the results of a desktop assessment undertaken for the routes of the four proposed pipeline schemes between Gariep Dam to points within the GBWSS.

The findings of the study include:

- The regional geology is similar for all the proposed r5-6ncl, with the exception of a slight variation in the proportion of the route that traverse dolerite intrusions.
- Irrespective of the chosen route, sourcing of material is likely to be problematic in terms of haulage distance and availability. This may have cost implications.
- Due to local climatic conditions, the residual soil profile is expected to be thicker in the east, becoming thinner and weakly developed towards Gariep Dam. This will affect excavatability along the route.
- Irrespective of the chosen route, it is unlikely that the in-situ material would be suitable for bedding and commercial sources should be considered.

The summarised geotechnical considerations and risks are listed in Table 5-2.

Table 5-2: Conditions and considerations for each of the proposed pipeline options

Pipeline option	Pipeline length (km)	Geology (estimated % route underlain by dolerite)	Cross large rivers	Road crossing	Railway crossing	Unique characteristics
Scheme 1 (Red in Figure 5-1)	181	10 (Most of the route potentially characterised by potentially expansive soils)	Multiple crossings	pipe-jacking /excavation and replacement	pipe-jacking	
Scheme 2 (Purple in Figure 5-1)	198	5 (Mostly potentially expansive)	Multiple crossings	pipe-jacking /excavation and replacement	pipe-jacking	
Scheme 3 (Green in Figure 5-1)	190	20 (Mostly potentially expansive)	Multiple crossings	Excavation and replacement generally possible	pipe-jacking	Locally along a border of a dam
Scheme 4 (White in Figure 5-1)	214	10 (Mostly potentially expansive)	Multiple crossings	pipe-jacking /excavation and replacement	pipe-jacking	Locally along a border of a dam

Based on the comparison contained in Table 5-2, none of the pipeline routes under consideration is deemed fundamentally flawed from a geotechnical perspective. Given the higher percentage of dolerite expected along Scheme 3, the other three routes might be slightly more economical in terms of excavation costs.

A detailed geotechnical investigation, comprising the drilling of boreholes, excavation of test pits and laboratory testing will be undertaken during the detailed feasibility study for the preferred scheme to verify the desktop findings.

6 Topographical Survey – Available Survey Information

The South African 1:50 000 dataset, comprising 20m contour data, and the National Geo-Spatial Information (NGI) dataset, comprising 5m contour data, was used to build a Digital Elevation Model (DEM) for the pre-feasibility design. The DEM was of sufficient accuracy to compare different route options/schemes. Figure 6-1 shows the extent of project area for which NGI contour data was available.

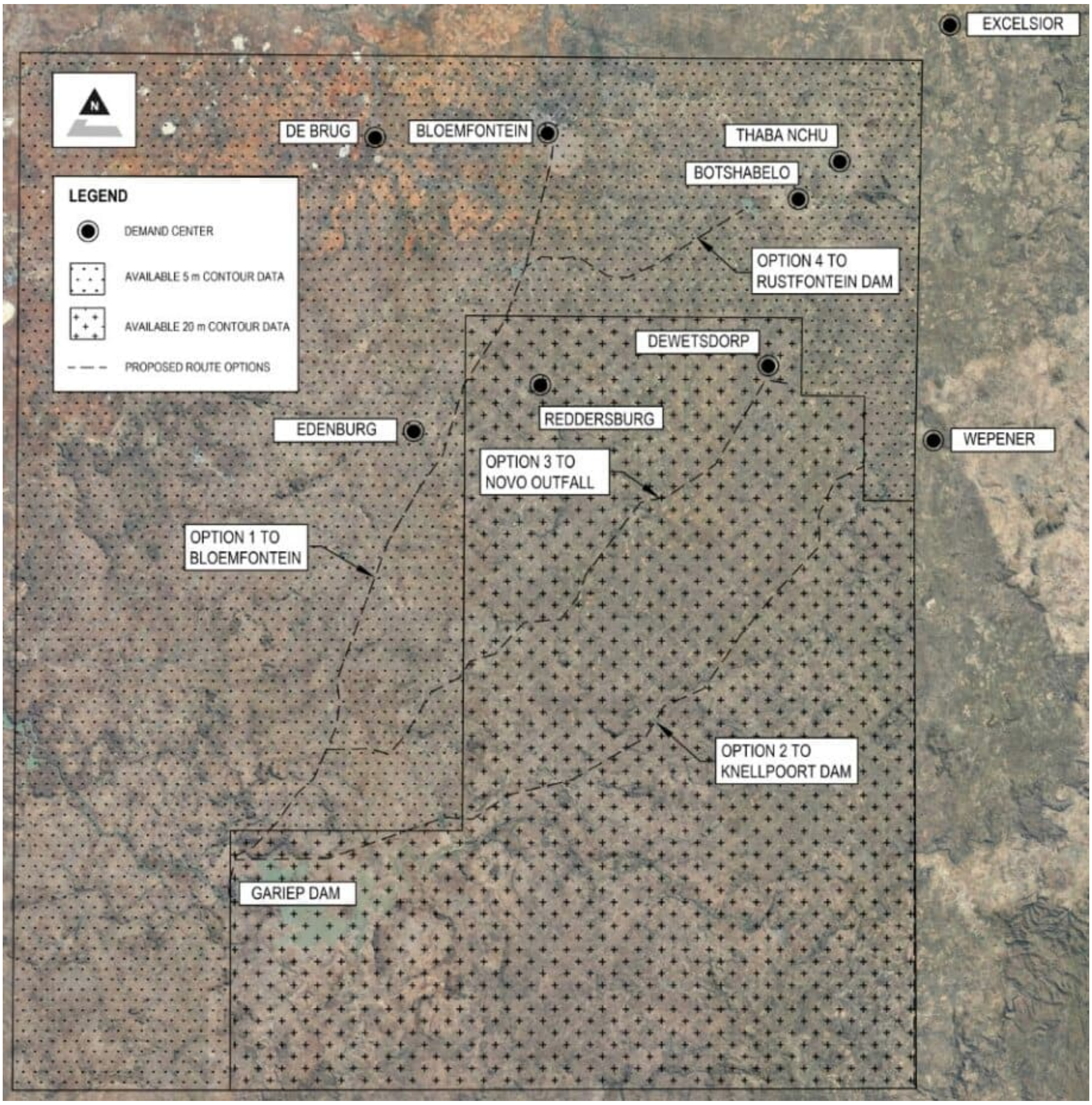


Figure 6-1: Extent of NGI contour data

In preparation for the feasibility study, documentation was prepared for the procurement of a Light Detection and Ranging (LiDAR) service provider. The specifications for the LiDAR survey included:

- An airborne survey of the project area for the preferred option. In areas of uncertainty, e.g., where the position of a pump station must still be finalised, the survey area would be increased so that minor deviations could be accommodated without requiring an additional survey afterwards. The airborne survey will include the classified data points in LiDAR Aerial Survey (LAS) file format (X;Y;Z format) with an aerial photo backdrop.

- Installation of ground control points and benchmarks that can be used as reference points during construction.
- Line mapping of the project areas.
- The service provider, appointed by the Professional Service Provider, must also collect all the relevant cadastral data to show existing servitudes and property/farm boundaries.

During the pre-feasibility phase, data on the existing survey control along the N1 road was obtained from SANRAL. This information will be made available to the service provider for integration with the benchmark data.

The survey, which will be based on the World Geodetic System (WGS 84), straddles more than one longitude of origin (Lo) system. The service provider will agree with DWS and Zutari the preferred Lo system to be used for the project prior to commencing with the topographical survey.

7 Pre-feasibility Design, Infrastructure Sizing and Cost Estimation

7.1 Scheme Development and Basis of Design

The study methodology is detailed in Section 1.3 and explains the iterative process between the water resources modelling and infrastructure design tasks. It also details how all four schemes (refer to Figure 1-3) were initially compared to each other, whereafter the preferred raw water scheme and potable scheme were evaluated in further detail. Understanding how the proposed infrastructure will be integrated with the existing GBWSS infrastructure, was considered key for the optimal performance of the overall system.

The approach to the infrastructure investigated in this pre-feasibility study was as follows:

- Evaluated the capacity of the existing infrastructure and determined the upgrades required to the existing infrastructure from a regional perspective that would satisfy the 2050 water demands (Section 7.2),
- Established the design parameters required for the bulk infrastructure design, e.g. horizontal and vertical pipeline alignments, design flows, hydraulic parameters, pump sizing and storage capacities (Section 7.3),
- Determined the infrastructure requirements for Schemes 1 to 4 based on a maximum transfer volume of 60 million m^3/a, excluding raw water losses, from Gariep Dam, and selected the preferred raw water and potable water schemes (Section 7.4),
- Determined the infrastructure requirements for Schemes 1A, 1B (both potable schemes) and 4B (preferred raw water scheme) based on the increased abstraction required from Gariep Dam to satisfy the 2050 demands, and selected the preferred scheme for implementation (Section 7.5),
- Undertook the design optimisation of Scheme 1B, which is the preferred scheme for implementation (Section 7.6), and
- Evaluated the process requirements for the proposed WTW required as part of Scheme 1B (Section 7.7).

7.2 Existing Infrastructure and Common Upgrades

To determine the infrastructure requirements for the proposed schemes it was important to develop an understanding of the existing pipeline, pump station and WTWs capacities. The systems characteristics and capacities were obtained from existing drawings and reconciled with site and desktop observations as well as data received from the DWS and VCWB. A schematic of the existing system infrastructure was developed. The schematic (Drawing No. 1002533-000-DRG-CC-0002) is included in Appendix A.

The information used and assumptions made in determining the existing system capacities and the required upgrades are as follows:

- Known maximum operating velocities in existing pipelines were used where available,
- A maximum permissible velocity of 1.7 m/s was assumed for pipelines where flow data was not available,
- A design velocity of 1.5 m/s was used for proposed pipeline upgrades,
- Sizing of the 'common infrastructure', i.e. infrastructure that will be required irrespective of which scheme/option is implemented, was based on fulfilling the 2050 demand projections and no peak week factor was applied, and,
- Rustfontein WTW will supply the northern towns of Botshabelo and Thaba Nchu to their maximum demand before supplying Bloemfontein. This is based on the operating rules and penalty system applied in the WRYM.

The existing infrastructure capacities were compared to the 2050 demands for each demand centre to determine where additional infrastructure will be required to satisfy the 2050 demands. The calculations and results of this comparison are presented in Table 7-1. All bulk transfer pipelines with diameters exceeding 600 mm were assumed to be steel pipelines and those with smaller diameters were assumed to be high-density polyethylene (HDPE) or polyvinyl chloride (PVC) pipelines.

Additionally, various interventions were proposed in previous studies (e.g. 2012 Reconciliation Strategy) as described in the Data Collection, Review and Analysis Report (P WMA 06/D00/00/3423/4). **The interventions required for the successful implementation of a Xhariep Pipeline infrastructure were included in the water resources yield modelling and must be implemented, irrespective of the scheme/option selected.**

The infrastructure upgrades that are required across all the proposed schemes/options were termed 'common' infrastructure. These 'common' requirements were included to provide a holistic representation of each scheme but were excluded from the financial assessment of this project. The intention of showing the overall infrastructure requirements for the GBWSS was also to sensitise all stakeholders to infrastructure that must be developed by them to satisfy the 2050 water demands.

The additional 'common' infrastructure upgrades and interventions that are not shown in Table 7-1 are listed below:

- 10 Mℓ/d upgrade to Maselspoort WTW and pump station (2012 Reconciliation Strategy),
- 89 Mℓ/d upgrade to Rustfontein WTW and pump station (based on 2050 demands),
- 276.5 Mℓ/d (3.2 m^3/s) upgrade to Tienfontein pump station (2012 Reconciliation Strategy),
- 224.6 Mℓ/d (2.6 m^3/s) upgrade to Novo pump station (2012 Reconciliation Strategy), and
- New bi-directional transfer scheme between Welbedacht Dam and Knellpoort Dam including a 172.8 Mℓ/d (2 m^3/s) pump station, 172.8 Mℓ/d (2 m^3/s) rising main and 57.5 Mℓ/d (0.665 m^3/s) gravity main (2012 Reconciliation Strategy).

All 'common' infrastructure required as a basis for each scheme is illustrated on the schematic with drawing No. 1002533-000-DRG-CC-0003 included in Appendix A. The infrastructure designs presented in the subsequent sections assume that all common upgrades and interventions are implemented.

It is noted that the 2050 demands are AADD. In the water resources yield modelling, peak month factors were applied to the AADD when evaluating whether the demands can be supplied on a month-to-month basis while taking into consideration fluctuating dam levels. It is important to note that potable bulk water infrastructure should be designed based on peak week demands. As the 'common' upgrades are intended to be largely indicative across all schemes (potable and raw), no peak week demands were applied during the infrastructure sizing of the 'common' infrastructure. The institutions responsible for implementing the 'common' infrastructure must therefore adjust the already applied monthly peak factors to a peak week factor when undertaking the design of the potable water infrastructure.

Table 7-1: Summary of existing bulk water pipelines and proposed common upgrades

Pipeline description	Known/ assumed max velocity	Nominal diameter existing pipe	Existing pipeline capacity	Demand flow 2050	Additional capacity required	Design velocity	Estimated Diameter
Calculation parameters	v_1	D_1	$Q_1 = v_1 \times \frac{D_1}{1000}$	Q_2	$Q_3 = Q_2 - Q_1$	v_2	$D_2 = 1000 \times \sqrt{\frac{4 \times Q_3}{\pi \times v_2}}$
Units	m/s	mm	m^3/s	m^3/s	m^3/s	m/s	mm (material)
Welbedacht pipeline	1.70	1170	1.828	Pipe capacity sufficient - No upgrades to Welbedacht WTW			
Offtake to Wepener	1.70	219	0.064	0.051	Pipe capacity sufficient		
Offtake to De Wetsdorp	1.70	200	0.055	0.086	0.031	1.5	161 (HDPE DN200)
Offtake to Reddersburg and Edenburg	1.70	200	0.054	0.084	0.030	1.5	159 (HDPE DN160)
Reddersburg to Edenburg	1.70	150	0.030	0.037	0.007	1.5	79 (HDPE DN110)
Novo transfer pipeline	1.95	1200	2.200	4.800	2.600	1.5	1486 (Steel DN1500)
Rustfontein WTW to Botshabelo (Lesaku)	2.25	648	0.742	2.186	1.444	1.5	1107 (Steel DN1100)
Botshabelo to Thaba Nchu pipeline	1.70	406	0.220	0.887	0.667	1.5	752 (Steel DN750)
Groothoek WTW to Thaba Nchu	1.70	406	0.221	Pipe capacity sufficient – No upgrades to Groothoek WTW			
Maselspoort WTW to Bloemfontein	1.70	2 x 765	1.563	Pipeline capacity sufficient			

7.3 Design Parameters for Bulk Water infrastructure

This section describes the design parameters adopted for the planning and sizing of the bulk water infrastructure.

7.3.1 Horizontal and Vertical Pipe Alignments

The horizontal alignments proposed in previous studies were reviewed and found to be suitable for high level planning purposes but required refinement for this pre-feasibility investigation. The revised horizontal alignments followed existing services corridors (e.g. roads, pipelines, powerlines, etc.) as far as practically possible to minimise the potential impact on landowners and to enable easier access for construction and future maintenance.

Where following existing roads added impractically long sections of pipe or where the topography was unfavourable (e.g. high points that could increase pumping costs or increase the risk of excessive surge pressures), alternative routes which deviated from the road edge were evaluated and if feasible, these alternative routes were selected. The available contour data was used to obtain the vertical profiles of the alignments.

Where there was not a well-defined high point on the original alignment, nearby peaks in the topography were identified and the horizontal alignment was adjusted to pass through or near them. This facilitated the placement of in-line reservoirs, which was required to simplify the control of the scheme. All the proposed schemes' alignments underwent this optimisation process to ensure they promoted feasible pipeline configurations that would not require significant adjustments in the detailed feasibility design stage.

A summary of the alignment characteristics of each scheme is provided in Table 7-2. A plan layout of the proposed horizontal alignments along with their associated vertical profiles for each scheme is shown in Figure 7-1.

Table 7-2: Summary of bulk water transfer pipeline characteristics used for design

Scheme No.	Pipeline length (km)	Water type	Termination point	Terminal elevation (masl)
1	181.2	Potable	Longridge Reservoir	1475.0
2	190.4	Raw	Knellpoort Dam	1456.0
3	197.8	Raw	Novo Outfall	1539.0
4	203.9	Raw	Rustfontein Dam	1377.0

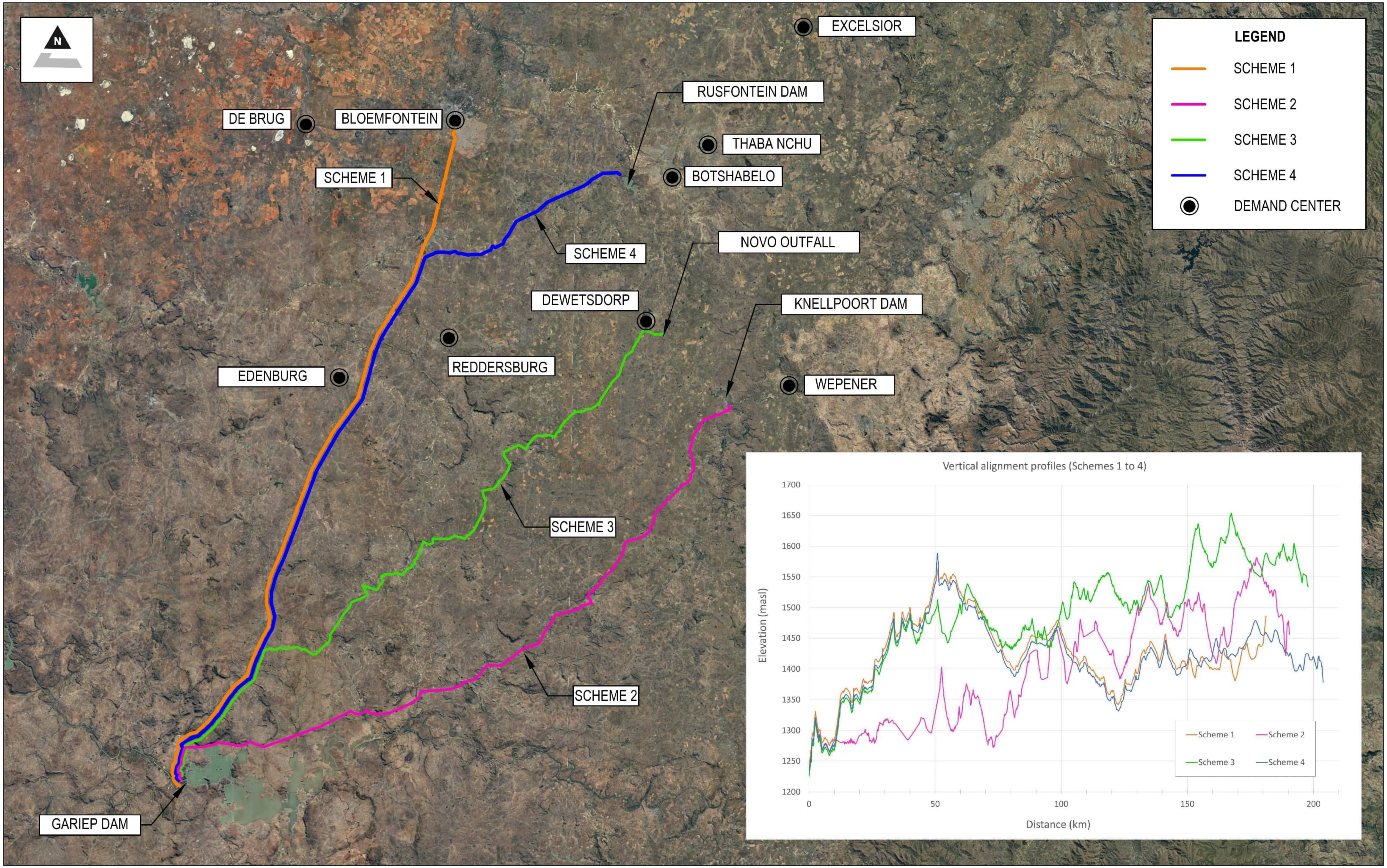


Figure 7-1: Plan layout of bulk water pipeline routes and associated vertical profiles for original four schemes

7.3.2 Design Flow Rates and Pipe Sizing

The four schemes (refer to Figure 7-1) comprise one potable water scheme (Scheme 1) and three raw water schemes (Schemes 2, 3 & 4). The first phase of the water resources yield modelling considered a maximum transfer volume of 60 million m³/a from Gariep Dam. Table 7-3 summarises the HFY of each scheme, the average and maximum monthly flows from each WTW and the percentage of the 2050 demand that can be supplied for each scheme. The red text indicates that the scheme does not meet the 2050 demands and the green text indicates that the demands are met.

The following observations are made from the information presented in Table 7-3:

- The HFY for Scheme 4 is the highest of the three raw water schemes when transferring 60 million m^3/a from Gariep Dam, which justifies including Scheme 4 for further consideration,
- None of the four schemes can satisfy the 2050 demands of the large demand centres, i.e. Bloemfontein, Botshabelo and Thaba Nchu,
- Scheme 1 (potable) has a slight bias towards supplying Bloemfontein (compared to the other schemes), whereas Schemes 2 and 3 have a bias towards supplying Botshabelo and Thaba Nchu, and,
- The average flows from the WTWs differ from scheme to scheme, e.g. the average flow from Rustfontein WTW is 60 Mℓ/d for Scheme 1 and 166.1 Mℓ/d for Scheme 2. Similarly, the average flow from Welbedacht WTW is 111.7 Mℓ/d for Scheme 1 and 83.2 Mℓ/d for Scheme 2.

Based on the above observations, it is evident that:

- The transfer volume from Gariep Dam needs to be increased to more than 60 million m^3/a, and,
- Operating costs should be calculated based on the total flows treated and pumped at all the WTWs and pump stations in the GBWSS to account for the difference in average flows between the various schemes/options.

The flow rates shown in Table 7-3 includes a monthly peak factor and were accepted for the infrastructure sizing of the raw water transfer schemes. However, potable water infrastructure sizing is based on peak week demands and must account for water treatment losses. Therefore, the flow rates used for the infrastructure sizing of the potable schemes were factored as follows:

- The low-lift pump station (LLPS) and abstraction pipeline were sized to allow for 5% raw water loss at the WTW at maximum flow rates, and,
- A 1.13 factor was applied to the peak month factor for high-lift pump station (HLPS) and pipeline sizing for a peak week factor of 1.30.

The flow rates used in the design of the bulk water transfer infrastructure from Gariep Dam to the termination point for each of the four schemes are summarised in Table 7-4.

Table 7-3: Table of WTW outflows of schemes assessed for this study

Scheme No. / Description (maximum transfer flowrate)	Historic firm yield (Mm³/a)	Rustfontein WTW		Maselspoort WTW		Welbedacht WTW		Proposed new Xhariep WTW		Bloem-fontein	Botshabelo & Thaba Nchu
		Avg Flow (Mℓ/d)	Max flow (Mℓ/d)	Avg Flow (Mℓ/d)	Max flow (Mℓ/d)	Avg Flow (Mℓ/d)	Max flow (Mℓ/d)	Avg Flow (Mℓ/d)	Max flow (Mℓ/d)	2050 Demands met (%)	2050 Demands met (%)
Scheme 1: Potable water to Bloemfontein (60 Mm³/a)	131	60.0	189.2	71.3	110.0	111.7	137.9	122.1	164.2	59.1	84.3
Scheme 2: Raw water to Knellpoort Dam (60 Mm³/a)	119	166.1	188.8	66.6	184.0	83.2	137.9	-	-	44.3	92.6
Scheme 3: Raw water to Novo Outfall (60 Mm³/a)	120	173.1	188.8	60.9	183.9	85.1	137.9	-	-	43.1	96.2
Scheme 4: Raw water to Rustfontein Dam (60 Mm³/a)	134	174.2	189.2	77.5	184.0	105.3	137.9	-	-	55.2	97.1

Table 7-4: Design flow rates for bulk water transfer infrastructure from Gariep Dam to scheme termination

Scheme No.	Maximum transfer flow rate		Average transfer flow rate		LLPS and abstraction		HLPS and bulk transfer pipeline	
Calculation	Q_{max}		Q_{avg}		$Q_{abs} = 1.05 \times Q_{max}$		$Q_{des} = 1.13 \times Q_{max}$	
Unit	m³/s	Mℓ/d	m³/s	Mℓ/d	m³/s	Mℓ/d	m³/s	Mℓ/d
1	1.901	164	1.411	122	1.996	172	2.148	186
2	1.901	164	1.035	89	N/A	N/A	1.901	164
3	1.901	164	1.645	142	N/A	N/A	1.901	164
4	1.901	164	1.431	124	N/A	N/A	1.901	164

The pipe diameters required for each scheme were calculated based on Q_{abs} and Q_{des} for abstraction and delivery pipelines. At the initial stage of the investigation, there was still uncertainty about the location of the low-lift pump station for the potable options. Therefore, the abstraction pipelines were modelled as an extension of the main delivery line to the connection point downstream of the Gariep Dam wall. This abstraction point remained the same across all schemes for comparison purposes.

The initial pipe sizing was completed using the principles of continuity. The continuity equation can be expressed as:

$$Q = vA$$

Where:

Q = flow rate (m³/s)
v = velocity (m/s)
A = flow area or internal diameter of pipe (m²)

The flow rates and an initial velocity of 1.5 m/s were used to determine the initial flow area and related internal pipe diameters required. The resulting diameters were used as the starting input for the hydraulic analysis. The diameters were then optimised using the hydraulic analysis and costing model described in Section 7.3.3. In some cases, larger diameters and lower velocities resulted in lower pumping costs which reduced the overall cost or net present value (NPV) of the scheme. The final diameters selected for each scheme are presented in Table 7-7.

7.3.3 Pipeline Hydraulics

The Darcy-Weisbach equation was used to calculate the frictional losses in the pipeline. The Darcy-Weisbach equation can be expressed as:

$$h_f = \frac{fLv^2}{2gD}$$

Where:

h_f = friction head loss (m)
f = friction coefficient (Colebrook-White friction factor as a function of pipe roughness, k)
L = length of pipe (m)
v = velocity (m/s)
g = gravitational acceleration (9.81 m/s²)
D = internal diameter of pipe (m)

The typical pipe roughness values (k) for new and aged cement mortar lined steel pipes are 0.15 mm and 0.60 mm, respectively. In comparison, the pipe roughness (k) for a new and aged epoxy lined steel pipe is 0.03 mm and 0.15 mm, respectively. Although the proposed pipelines would be newly installed for this project, a more conservative pipe roughness of 0.60 mm was selected to calculate the maximum anticipated working pressures to be considered for pump sizing and calculating the pipe wall thickness.

The decision on the preferred pipe lining (i.e. epoxy or cement mortar) will be taken during the detailed design stage of the project.

The hydraulic gradelines were used to inform the sizing of the gravity pipelines and determine the maximum and average working pressures required by the high-lift and booster pump stations. The hydraulic gradelines calculated for all sub-options of the schemes are included in Appendix B. As an example, Figure 7-2 and Figure 7-3 shows the hydraulic gradelines for the maximum and average transfer flow rates for Scheme 1, Sub-Option 3.

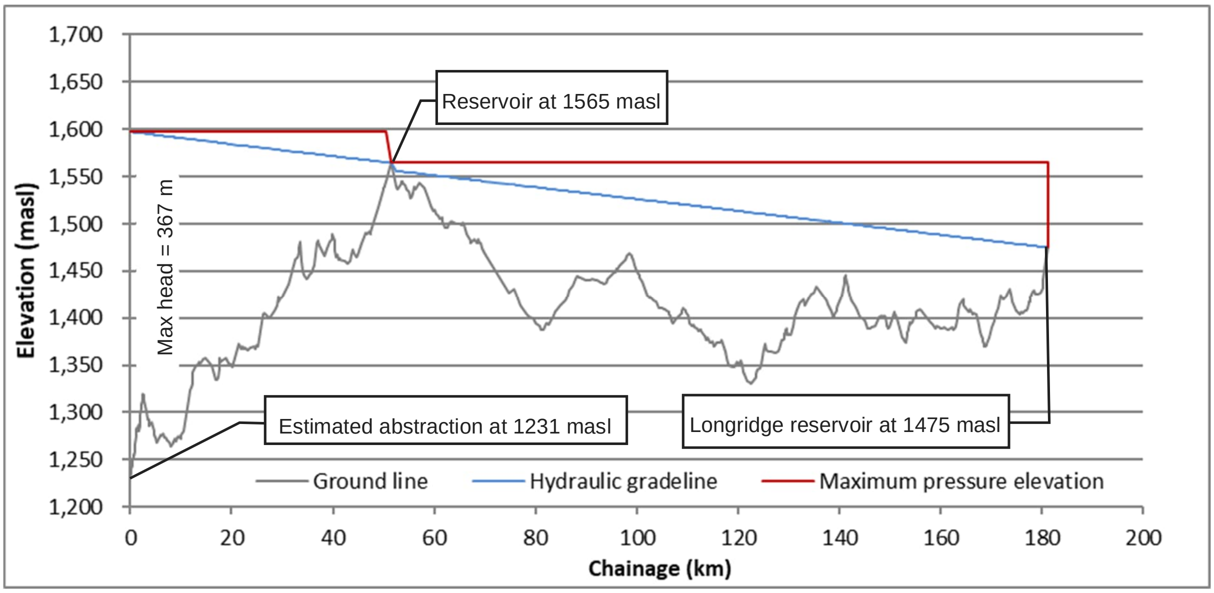


Figure 7-2: Hydraulic gradeline of Scheme 1 (Sub-Option 3) for maximum flow from Gariep Dam to Bloemfontein

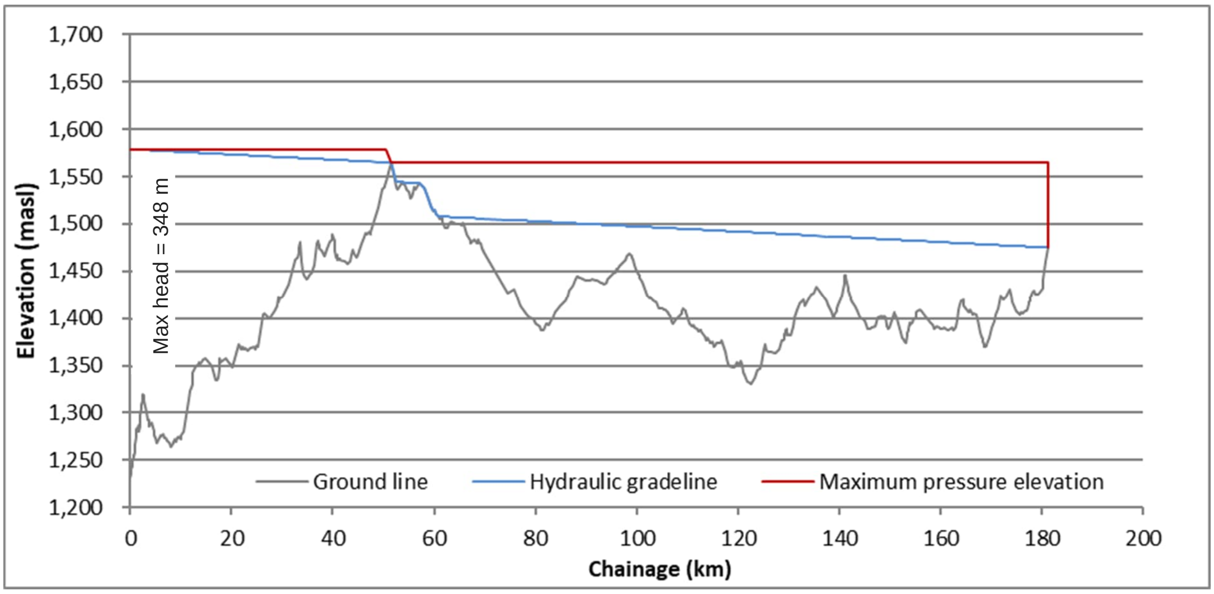


Figure 7-3: Hydraulic gradeline of Scheme 1 (Sub-Option 3) for average flow from Gariep Dam to Bloemfontein

The gradeline calculated from the maximum flow rate was used to find the peak duty point (flow rate and pumping head) to size the pumping infrastructure and calculate the peak power requirements. It was also used to determine the pipe pressure classes required across the full length of the pipeline (maximum hydraulic gradeline or static pressure, whichever is the higher, minus the ground elevation).

7.3.4 Pump Sizing and Power Requirements

The pumping infrastructure required for each scheme was determined from the hydraulic gradeline calculations for maximum flows as described in Section 7.3.3. The power requirement was calculated based on the maximum and average flows, and the corresponding pressure head at each pump station in the configuration. The power requirements were calculated using the hydraulic power equation expressed as follows:

$$\boldsymbol{P} = \frac{\boldsymbol{pgQ}}{\boldsymbol{n}}$$

Where:

P = power (kW)
p = maximum pressure head (m)
g = gravitational acceleration (9.81 m/s²)
Q = design flow rate (m³/s)
n = hydraulic pump efficiency (estimated at 70%, which is conservative)

The power requirements calculated based on the average flows were used to determine the operating and NPV costs.

7.3.5 Reservoirs

A minimum of one reservoir was included along the bulk water transfer pipeline of each scheme to improve the operability and maintainability of the infrastructure system. The reservoir(s) were sized to allow for six hours of storage at the peak flow rates. This storage will reduce system downtime during repair or maintenance activities (e.g. depending on the section of pipeline isolated, some smaller demands can still be supplied from the reservoir), and will assist with re-filling the pipeline after a repair (i.e. having additional storage will allow subsequent filling to commence immediately after the repair is made), as well as to simplify the control and operation of the pumping system (e.g. having a larger storage capacity will reduce the number of pump starts and stops).

The location of these reservoirs and pump stations were optimised by testing various configurations (sub-options) and analysing their hydraulic gradelines and cost.

It is noted that these balancing reservoirs are conservatively sized when compared to the Technical Guidelines for DWS Infrastructure (DWS, 2004), as the guidelines propose a minimum of 30 minutes storage for break pressure tanks and control reservoirs. However, given the advantages associated with the larger storage capacities and the small cost of these larger reservoirs relative to the overall project cost, the proposed 6-hour storage was considered appropriate and necessary.

7.4 Proposed Bulk Water Infrastructure (Schemes 1 to 4)

7.4.1 Scheme and Sub-option Development

A hydraulic analysis and costing model was developed to allow for the testing of multiple sub-options per scheme. These sub-options consisted of different infrastructure configurations to ensure that the cost of each scheme was optimised and the comparative cost analysis between each scheme was fair. The main technical input parameters that were used in the model are summarised as follows:

- Pipeline chainage and elevation data taken from optimised horizontal and vertical alignments,
- Reservoir and pump station locations specified for each sub-option at selected chainages,
- Pipe material, pipe diameters and friction roughness coefficients were selected,
- Peak factors were applied to determine the maximum required flow rate (used for pipe and pump sizing and to determine capital costs), and,

- Average flow rates were applied to determine operational costs.

One of the technical outputs of the model was the hydraulic gradeline of the pipeline. An iterative process was followed, whereby initial infrastructure configurations were set up and the resulting gradelines analysed. The reservoir and pump station locations along with the pipe diameters were adjusted until a technically suitable hydraulic gradeline was achieved. Thereafter, additional diameters were tested using the same reservoir and pump station configuration. As part of the iterative process, the following parameters were calculated:

- **Maximum velocities** – it is preferable to limit maximum velocities in long pumping mains to below 2.5 m/s to mitigate excessive friction losses and to reduce the risk of excessive surge pressures,
- **Working pressure along pipeline** – it is preferable to limit working pressures to 400 m pressure head as fittings (e.g. valves) with a PN40 pressure rating are more readily available that PN63 fittings, and,
- **Pumping head** – it is preferable to limit the maximum pumping pressure head to 400 m as the pump station fittings could then be PN40 rated. It is, however, feasible to consider PN63 rated fittings for the pump station as the number of fittings is low compared to those along a 200 km long pipeline.

Each viable diameter change created a new sub-option which was used to optimise the cost of that configuration. A flow diagram illustrating the main sub-option inputs and the iterative process is shown in Figure 7-4 and the hydraulic gradelines for the sub-options assessed for Scheme 1 are shown in Figure 7-5. Table 7-5 summarises all the sub-options considered for Scheme 1.

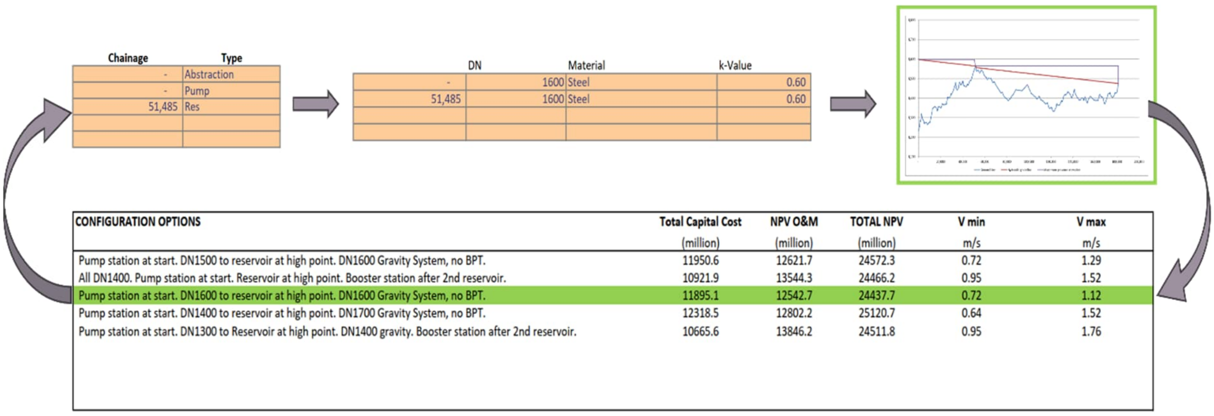
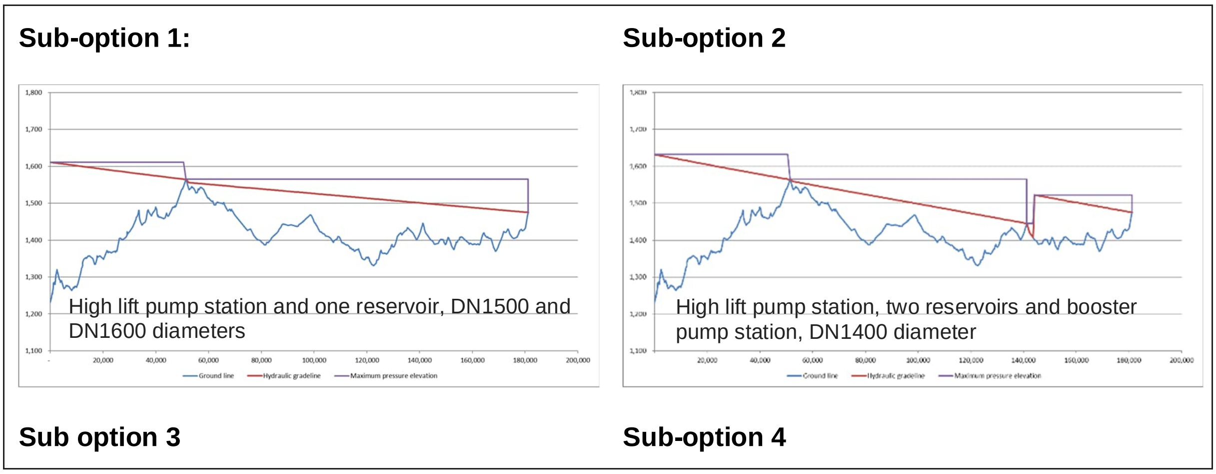
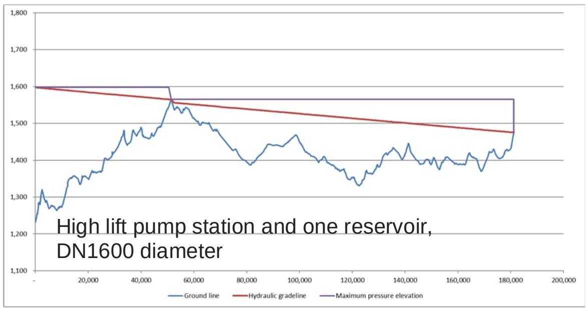
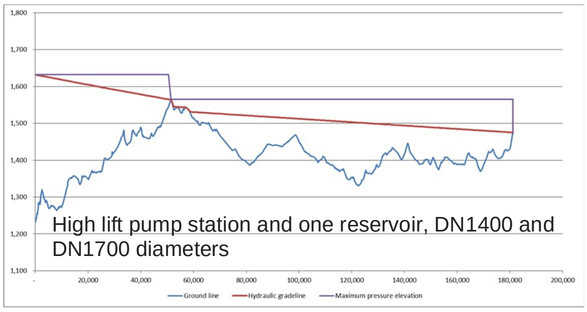


Figure 7-4: Diagram illustrating iterative sub-option selection process for Scheme 1







Sub-option 5

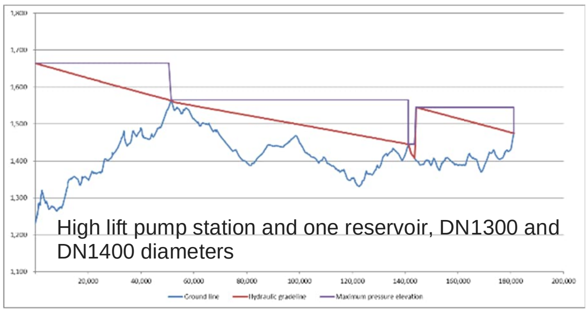


Figure 7-5: Hydraulic gradelines of sub-options assessed for Scheme 1

Table 7-5: Sub-options considered for Scheme 1

Scheme 1 Sub-option	Nominal diameters	Description of configuration	Minimum velocity (m/s)	Maximum velocity (m/s)
1	1500, 1600	High-Lift pump station (HLPS) and rising main to reservoir (~CH51485 m). Gravity line to Longridge reservoir.	0.72	1.29
2	1400	HLPS and rising main to first reservoir at high point (~CH51485 m). Gravity line to second reservoir (~CH141156 m) and booster pump station to Longridge reservoir.	0.95	1.52
3	1600	High-Lift pump station (HLPS) and rising main to reservoir (~CH51485 m). Gravity line to Longridge reservoir.	0.72	1.12
4	1400, 1700	High-Lift pump station (HLPS) and rising main to reservoir (~CH51485 m). Gravity line to Longridge reservoir.	0.64	1.52
5	1300, 1400	HLPS and rising main to first reservoir (~CH51485 m). Gravity line to second reservoir (~CH141156 m) and booster pump station to Longridge reservoir.	0.95	1.76

The process noted above was followed for all subsequent schemes investigated in this study with five technically feasible sub-options developed for each scheme. A table describing all configurations (sub-options) tested for each scheme is presented in Appendix B.

7.4.2 Selection of Preferred Sub-options

The aim when developing the sub-options was to find the configuration that will result in the lowest NPV, while also ensuring that the sub-option was a practical and technically sound solution. The most technically and financially beneficial sub-option was then selected to represent each scheme in the options analysis.

The main criteria used to assess the sub-options were as follows:

- Velocities within acceptable range of 0.7 – 2.5 m/s,
- Pumping pressure heads to be preferably lower than 400 m,
- Operating pressure heads in pipelines to be kept to below 400 m,
- Cost of sub-option, and,
- Ease of operation and maintenance.

The costing model was set up to calculate the capital expenditure (CAPEX), operational expenditure (OPEX), the NPV and the unit reference value (URV) for each scheme which allows the comparison of the schemes. Details on the financial assessment are provided in Section 9.

The most economical sub-option was selected based on the NPV and URV. In the cases where the cost was within 5% between competing sub-options and the velocities were both within range, preference was given to the sub-option with fewer pump stations to simplify maintenance and operation requirements. The sub-option selection tables with the preferred sub-option highlighted in green is provided in Table 7-6. Descriptions of the infrastructure components for the recommended sub-options for Schemes 1, 2, 3 and 4 are provided in Table 7-7. The selected sub-options were used in the financial assessment of the schemes detailed in Section 9.

Table 7-6: NPV and URV summary of sub-options (Schemes 1 to 4)

Scheme 1: Preferred sub-option 3

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
Pump station at start. DN1500 to reservoir at high point. DN1600 Gravity System, no BPT.	11950.6	12621.7	24572.3	6.4	12.37	OPTION 1	0.72	1.29
All DN1400. Pump station at start. Reservoir at high point. Booster station after 2nd reservoir.	10921.9	13544.3	24466.2	6.8	12.32	OPTION 2	0.95	1.52
Pump station at start. DN1600 to reservoir at high point. DN1600 Gravity System, no BPT.	11895.1	12542.7	24437.7	6.3	12.31	OPTION 3	0.72	1.12
Pump station at start. DN1400 to reservoir at high point. DN1700 Gravity System, no BPT.	12318.5	12802.2	25120.7	6.4	12.65	OPTION 4	0.64	1.52
Pump station at start. DN1300 to Reservoir at high point. DN1400 gravity. Booster station after 2nd reservoir.	10665.6	13846.2	24511.8	7.0	12.34	OPTION 5	0.95	1.76

Scheme 2: Preferred sub-option 3

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
DN1300 rising main, two reservoirs at high points, DN1600 stretch to booster pump station.	12885.0	12021.8	24906.8	6.7	13.9	OPTION 1	0.53	1.57
DN1300 Pump station at start. One reservoir at high point.	15448.2	12518.0	27966.1	7.0	15.6	OPTION 2	0.81	1.57
DN1300 rising main, two reservoirs and booster pump station	9948.2	11648.4	21596.6	6.5	12.02	OPTION 3	0.81	1.57
DN1400 Pump station at start. One reservoir at high point.	14824.3	12066.3	26890.6	6.7	15.0	OPTION 4	0.70	1.34
DN1400 rising main, two reservoirs and booster pump station	10288.9	11308.8	21597.7	6.3	12.02	OPTION 5	0.70	1.31

Scheme 3: Preferred sub-option 4

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
DN1300 Pump station at start. Two reservoirs at high points with booster pump station inbetween.	8833.0	14373.4	23206.4	8.0	12.8	OPTION 1	1.29	1.57
DN1300 Pump station at start. Two reservoirs at high points with booster pump station inbetween (2)	10233.2	14446.2	24679.4	8.0	13.7	OPTION 2	1.29	1.57
DN1400 Pump station at start. Two reservoirs at high points with booster pump station inbetween.	9117.7	13854.6	22972.3	7.7	12.7	OPTION 3	1.11	1.31
DN1400 and DN1300. Pump station at start. Two reservoirs at high points with booster pump station.	8917.9	13811.1	22729.0	7.6	12.6	OPTION 4	1.11	1.49
DN1200 rising main, two reservoirs at high points, DN1300 gravity end	9339.2	15921.4	25260.7	8.8	14.0	OPTION 5	1.29	1.87

Scheme 4: Preferred sub-option 1

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
All DN1300. Pump station at start. Two reservoirs at high points with booster pump station inbetween.	8473.2	12718.1	21191.3	6.5	10.84	OPTION 1	1.12	1.57
DN1400/DN1500 Pump station at start. Two reservoirs at high points with booster pump station inbetween.	9209.3	12558.1	21767.4	6.4	11.1	OPTION 2	0.83	1.31
DN1200, DN1300 & DN1500 Pump station at start. Two reservoirs at high points with booster pump station inbetwee	9939.7	12364.2	22303.9	6.3	11.4	OPTION 3	0.73	1.85
DN1300 to high point reservoir. DN1500 gravity thereafter.	9430.4	12009.5	21440.0	6.1	11.0	OPTION 4	0.83	1.57
DN1300 to high point DN1500 to second high point reservoir. DN1400 gravity thereafter.	9254.1	11976.1	21230.1	6.1	10.86	OPTION 5	0.83	1.76

Table 7-7: Infrastructure details of recommended scheme sub-options (Schemes 1 to 4)

Scheme No.	Sub-option	Nominal diameters	Description of configuration	Minimum velocity (m/s)	Maximum velocity (m/s)
1	3	1600	High-Lift pump station (HLPS) and rising main to reservoir (~CH51485 m). Gravity line to Longridge reservoir.	0.72	1.12
2	3	1300	HLPS and rising main to first reservoir at local high point (~CH107482 m). Booster pump station and rising main to second reservoir (~177698 m). Gravity to Knellpoort Dam.	0.81	1.57
3	4	1300 & 1400	HLPS and rising main to first reservoir (~CH62920 m). Gravity line to booster pump station (~98778 m) and rising main to second reservoir (~CH167427 m). Gravity to Novo outfall.	1.11	1.49
4	1	1300	HLPS and rising main to first reservoir (~CH51485 m). Gravity line to booster pump station (~135446 m) and rising main to second reservoir (~CH178024 m). Gravity to Rustfontein Dam.	1.12	1.57

Table 7-8 provides a summary of the duty points based on average and maximum flows for each of the recommended sub-options per scheme. The power required per pump station, excluding any provision for stand-by capacity, is also shown.

Table 7-8: Summary of proposed new pump station operating requirements for each scheme

Scheme No.	HLPS Average duty point	HLPS Maximum duty point	HLPS power required	Booster Average duty point	Booster Maximum duty point	Booster power required	Annual transfer Volume
	ℓ/s | m	ℓ/S | m	kW	ℓ/s | m	ℓ/s | m	kW	Million m³/a
1	1,411 | 348	2,148 | 367	11,034	-	-	-	44,5
2	1,035 | 298	1,901 | 418	11,129	1,035 | 132	1,901 | 207	5,504	32,6
3	1,645 | 361	1,901 | 377	10,036	1,645 | 231	1,901 | 248	6,607	51,9
4	1,431 | 378	1,901 | 412	10,984	1,431 | 78	1,901 | 104	2,782	45,1

A summary of the balancing reservoir sizing calculations and quantity provided for the recommended sub-option for each scheme is provided in Table 7-9.

Table 7-9: Summary of reservoir requirements for each scheme

Scheme No.	Number of reservoirs	Design transfer flowrate	Storage volume required for 6 hours	Design storage volume
Calculation	-	Q_{des}	$S = 6 \times 3600 \times Q_{des}$	S_{des}
Unit	No.	m³/s	m³	Mℓ
1	1	2.148	46,400	47
2	2	1.901	41,062	42
3	2	1.901	41,062	42
4	2	1.901	41,062	42

7.4.3 Schematics

Schematics of the final scheme configurations were developed. These schematics consist of the proposed new infrastructure to be priced for each scheme and the ‘common’ upgrades which were excluded in the cost estimates. A summary of all schematics developed for the pre-feasibility investigation is presented in Table 7-10 and included in Appendix A.

Table 7-10: Summary of schematics developed for pre-feasibility investigation

Scheme No.	Drawing No.	Drawing Title
All	1002533-0000-DRG-CC-0002	Schematic of existing infrastructure
All	1002533-0000-DRG-CC-0003	Schematic of common upgrades to all options
1	1002533-0000-DRG-CC-0004	Schematic of Scheme 1, sub-option 3 [Sc5b] potable water to Bloemfontein (Historic firm yield 137 Mm³/a)
2	1002533-0000-DRG-CC-0005	Schematic of Scheme 2, sub-option 3 [Sc4b(ii)] raw water to Knellpoort Dam (Historic firm yield 119 Mm³/a)
3	1002533-0000-DRG-CC-0006	Schematic of Scheme 3, sub-option 4 [Sc4c(i)] raw water to Novo outlet (Historic firm yield 120 Mm³/a)
4	1002533-0000-DRG-CC-0007	Schematic of Scheme 4, sub-option 1 [Sc5f] raw water to Rustfontein Dam (Historic firm yield 134 Mm³/a)

7.4.4 Additional Infrastructure and Operational Considerations

The utilisation of the existing infrastructure varies between schemes and the recommended sub-options. Therefore, when considering the capital and operating cost of each scheme, it was also necessary to quantify the flows pumped and treated at all of the pump stations and WTW in the GBWSS, and to account for any additional infrastructure that will be required to supply the 2050 demands at the respective demand centres.

A summary of the additional pipeline and pump station infrastructure upgrades, as well as the average pumping duty, that needed to be included in the costing of each scheme is provided in Table 7-11.

Table 7-11: Summary of additional pipeline infrastructure and pumping operations associated with each scheme

Scheme No.	Description	Pipeline length	Pipeline diameter	Operating pump duty	Maximum pump duty
		km	mm	ℓ/s | m	ℓ/s | m
1	Rustfontein pump operation	-	-	1,434 | 199	-
	Welbedacht pump operation	-	-	1,293 | 247	-
	Novo transfer operation	-	-	734 |136	-
2	Maselspoort new pipeline and pump station upgrades	33.5	800	257 | 135	570 | 210
	Rustfontein pump operation	-	-	1,297 | 194	-
	Welbedacht pump operation	-	-	963 | 244	-
	Novo transfer operation	-	-	2,438 | 185	-
3	Maselspoort new pipeline and pump station upgrades	33.5	800	235 | 133	570 | 210
	Rustfontein pump operation	-	-	1,373 | 197	-
	Welbedacht pump operation	-	-	985 | 244	-
	Novo transfer operation	-	-	1,054 | 141	-
4	Maselspoort new pipeline and pump station upgrades	33.5	800	299 | 140	570 | 210
	Rustfontein pump operation	-	-	1,390 | 197	-

Scheme No.	Description	Pipeline length km	Pipeline diameter mm	Operating pump duty ℓ/s | m	Maximum pump duty ℓ/s | m
	Welbedacht pump operation	-	-	1,219 | 246	-
	Novo transfer operation	-	-	863 | 138	-

The capital and operating costs for the infrastructure listed above were quantified and priced using the same hydraulic analysis and costing model developed for the main bulk transfer pipeline. The precise horizontal alignments and vertical profiles of the existing pipelines were not available and were developed based on satellite imagery, available topographic data, and the terminal reservoir locations. The starting and terminal elevations were obtained from schematics provided in previous studies. These calculations were completed for comparative costing purposes only.

A summary of the new WTW infrastructure and WTW upgrades as well as the total treated volume required for each scheme is provided in Table 7-12.

Table 7-12: Summary of water treatment infrastructure and treatment volumes included in each scheme

Scheme No.	Description of scheme specific WTW upgrades required*	Scheme specific capacity increase (Mℓ/d)	Total annual volume treated at all WTWs (Million m³/a)
1	Proposed new Xhariep WTW	165	133.2
2	Upgrade of 64 Mℓ/d at Maselspoort WTW	64	115.3
3	Upgrade of 64 Mℓ/d at Maselspoort WTW	64	116.5
4	Upgrade of 64 Mℓ/d at Maselspoort WTW	64	130.3

*All capacity upgrades listed are over and above the 10 Mℓ/d upgrade to Maselspoort WTW and 89 Mℓ/d upgrade to Rustfontein WTW which are common to all schemes (See Section 7.2).

The scheme specific capacity increases represent the total additional treatment volume required by each scheme and was used to calculate the capital cost of the new or upgraded WTWs. The total annual treated volume is a summation of the average outflows from the new, upgraded and existing WTWs in the GBWSS over a one-year period. This was used to calculate the total treatment cost required by each scheme.

7.4.5 Comparison of Schemes 1 to 4

Table 9-6, repeated below as Table 7-13 for ease as reference, shows the financial comparison of Schemes 1 to 4 based on transferring 60 million m^3/a from Gariep Dam. It is evident from Table 7-13 that:

- Schemes 1 (direct potable supply to Bloemfontein) and 4 (raw water supply to Rustfontein Dam) are the two most economical schemes, and
- Scheme 4 is the most economical raw water scheme when compared to Scheme 2 (raw water supply to Knellpoort Dam) and Scheme 3 (raw water supply to Novo Outfall Structure).

Based on the financial comparison, it was recommended that further water resources yield modelling be undertaken for Schemes 1 and 4 to determine the transfer volumes required that will satisfy the 2050 water demands.

Table 7-13: Results of financial comparison for original schemes 1 to 4 with maximum annual transfer of 60 Mm³/a

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Present Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme comparison of original transfer at 60 Mm³ / annum	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**Mℓ**	**Mm³/annum**	**Mm³/annum**	**Mm³**	**Million Rands**	**(Million Rands)**	**(Million Rands)**	**R/m³**	**R/m³**	**%**
Scheme 1 [Sc5b] Potable water from Gariep Dam to Bloemfontein	181.2	DN1600	1411 | 348	2148 | 367	-	-	165	133.2	44.5	1986	11895	12543	24438	6.32	12.31	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1434 | 199	-	-	-	-	-	45.2	1986	0	1384	1384	0.70	0.70	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1293 | 247	-	-	-	-	-	40.8	1986	0	1549	1549	0.78	0.78	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	734 | 136	-	-	-	-	-	23.2	1986	0	485	485	0.24	0.24	-
										Total	**11895**	**15962**	**27857**	**8.04**	**14.03**	**+7.5**
Scheme 2 [Sc4b(ii)] Raw water from Gariep Dam to Knellpoort Dam	190.4	DN1300	1035 | 298	1901 | 418	1035 | 132	1901 | 207	64	115.3	32.6	1797	9948	11648	21597	6.48	12.02	-
+ Maselspoort pipeline and PS upgrades	33.5	DN800	257 | 135	570 | 210	-	-	Incl above	Incl above	8.1	1797	498	423	921	0.24	0.51	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1297 | 194	-	-	-	-	-	40.9	1797	0	1224	1224	0.68	0.68	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	963 | 244	-	-	-	-	-	30.4	1797	0	1142	1142	0.64	0.64	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	2438 | 185	-	-	-	-	-	76.9	1797	0	2194	2194	1.22	1.22	-
										Total	**10446**	**16632**	**27078**	**9**	**15.07**	**+15.4**
Scheme 3 [Sc4c(i)] Raw water from Gariep Dam to Novo outfall	197.8	DN1300 and DN1400	1645 | 361	1901 | 377	1645 | 231	1901 | 248	64	116.5	51.9	1808	8918	13811	22729	7.64	12.57	-
+ Maselspoort upgrades	33.5	DN800	235 | 133	570 | 210	-	-	Incl above	Incl above	7.4	1808	498	405	903	0.22	0.50	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1373 | 197	-	-	-	-	-	43.3	1808	0	1312	1312	0.73	0.73	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	985 | 244	-	-	-	-	-	31.1	1808	0	1169	1169	0.65	0.65	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	1054 | 141	-	-	-	-	-	33.2	1808	0	723	723	0.40	0.40	-
										Total	**9416**	**17421**	**26836**	**10**	**14.85**	**+13.7**
Scheme 4 [Sc5f] Raw water from Gariep Dam to Rustfontein Dam	203.9	DN1300	1431 | 378	1901 | 412	1431 | 78	1901 | 104	64	130.3	45.1	1955	8473	12718	21191	6.51	10.84	-
+ Maselspoort upgrades	33.5	DN800	299 | 140	570 | 210	-	-	Incl above	Incl above	9.4	1955	498	459	957	0.24	0.49	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1390 | 197	-	-	-	-	-	43.8	1955	0	1332	1332	0.68	0.68	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1219 | 246	-	-	-	-	-	38.4	1955	0	1457	1457	0.75	0.75	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	863 | 138	-	-	-	-	-	27.2	1955	0	578	578	0.30	0.30	-
										Total	**8971**	**16544**	**25515**	**8**	**13.05**	**100**

7.4.6 Stakeholder feedback on Schemes 1 to 4

A technical meeting was held on 2 November 2023 where the results for Schemes 1 to 4, based on transferring 60 million m^3/a from Gariep Dam, were presented to DWS, MMM and VCWB. In addition, the meeting was an opportunity to discuss any other operational matters to be considered during the detailed feasibility phase of the project. The following specific matters were raised at the meeting:

- Botshabelo and Thaba Nchu were experiencing higher levels of restriction compared to other towns within the GBWSS, mainly because these two towns are limited to supply from Rustfontein WTW alone whereas Bloemfontein can receive water from Welbedacht, Rustfontein and Maselspoort WTWs,
- VCWB preferred Scheme 2 (raw water supply to Knellpoort Dam) due to greater operational flexibility, e.g. raw water can be supplied from Knellpoort Dam to Welbedacht Dam as well as to Rustfontein and Maselspoort WTWs,
- The supply of potable water to towns located along the proposed pipeline route remains a priority from a regional water supply perspective,
- Scheme 1 is the only potable scheme under consideration, but it can only supply Bloemfontein and the towns along the pipeline route, i.e. it would not resolve the challenges experienced at Botshabelo and Thaba Nchu, and,
- All parties agreed that Schemes 1 and 4 had limitations in terms of overall flexibility and on improving the resilience of the GBWSS.

7.5 Proposed Bulk Water Infrastructure (Schemes 1A, 1B and 4B)

7.5.1 Development of Scheme 1B

Based on the feedback received at the technical meeting held on 2 November 2023, it was evident that a scheme had to be developed that could satisfy the following criteria:

- Supply potable water to the towns along the proposed pipeline route,
- Supply water to the major demand centres of Botshabelo, Thaba Nchu and Bloemfontein, and,
- Improve the resilience of the GBWSS, i.e. should downtime be experienced at any of the WTWs, the overall system should still be able to satisfy the majority of the 2050 water demands.

This led to the development of Scheme 1B (also referred to as the “hybrid” scheme since the pipeline route is a combination of the routes for Schemes 1 and 4) as shown in Figure 7-6 and Figure 7-7; where potable water is supplied from Gariep Dam to a command reservoir located between Bloemfontein and Rustfontein WTW. Water from the command reservoir can gravitate to Bloemfontein and Rustfontein WTW. The elevation of the command reservoir is such that water can be gravitated directly to Botshabelo, but the battery limit for this project is at the Rustfontein WTW with VCWB responsible for the infrastructure required from Rustfontein WTW to Botshabelo and Thaba Nchu. Similarly, the command reservoir will supply Longridge Reservoir in Bloemfontein with MMM being responsible for the further distribution of water to their end-users.

A summary of the bulk water transfer pipeline characteristics used for the design is provided in Table 7-14.

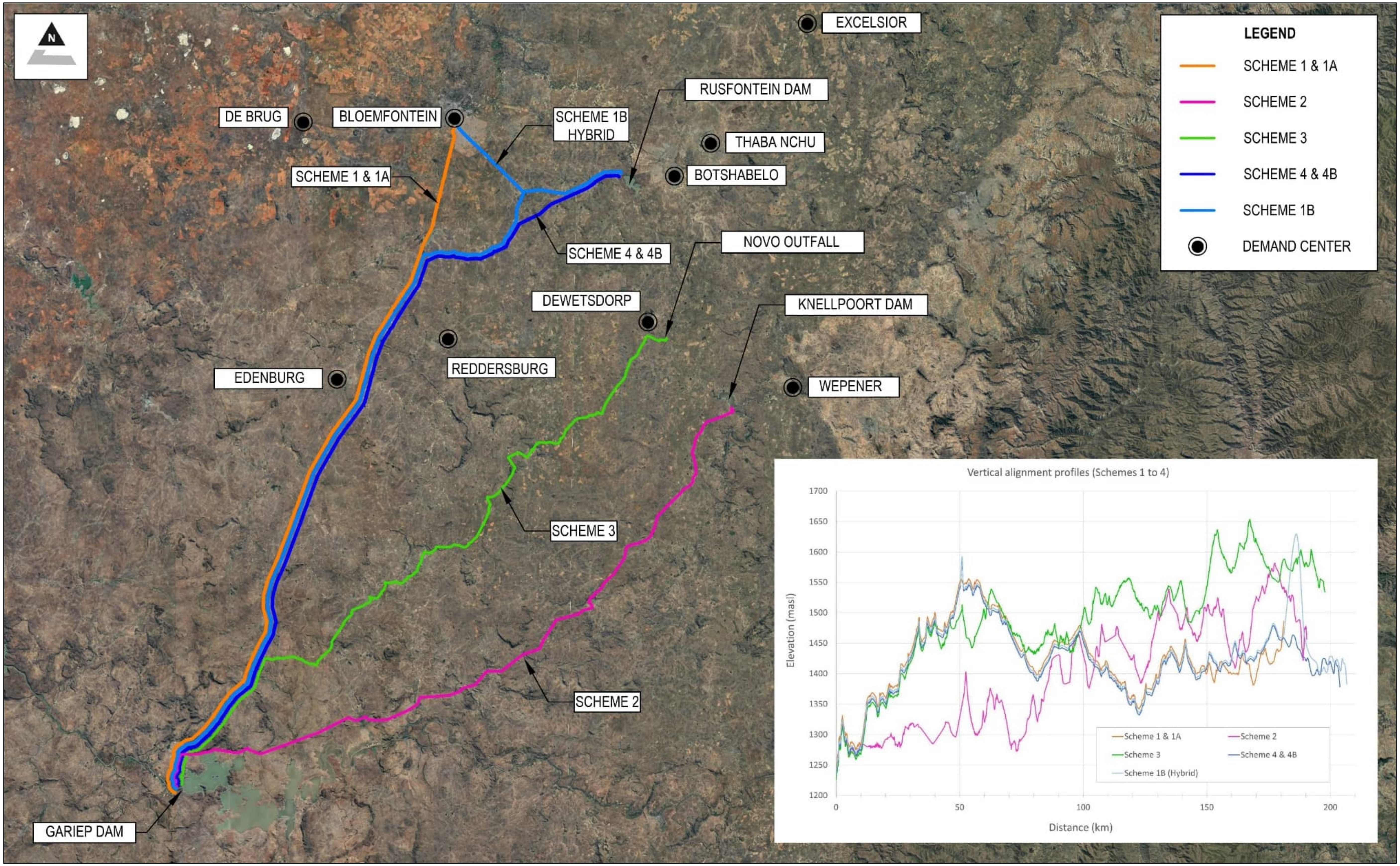


Figure 7-6: Plan layout of bulk water pipeline routes and associated vertical profiles for all schemes

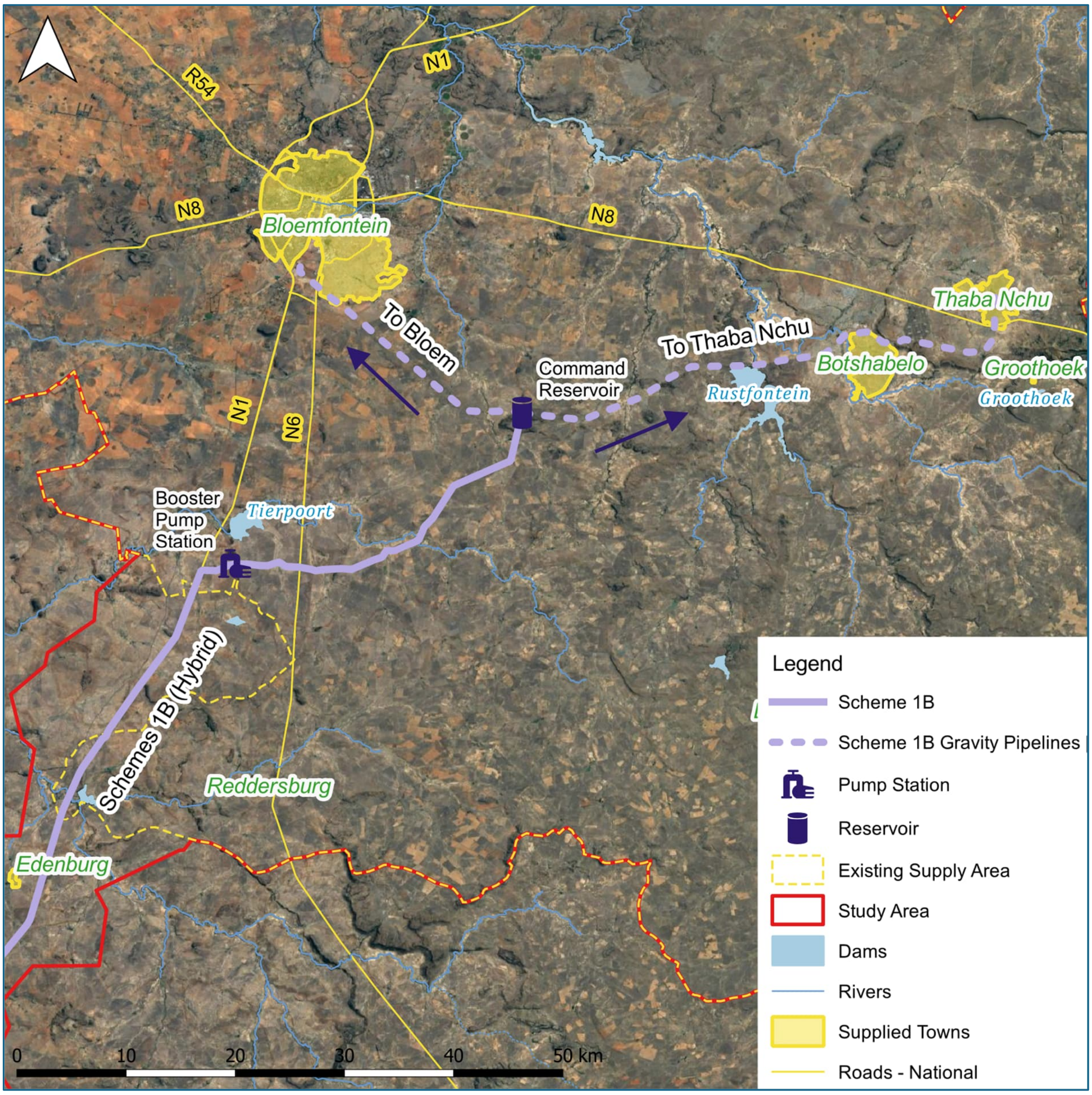


Figure 7-7: Scheme 1B supply to Bloemfontein and Rustfontein WTW

Table 7-14: Summary of bulk water transfer pipeline characteristics used for design (Schemes 1A, 1B and 4B)

Scheme No.	Pipeline length (km)	Water type	Termination point	Terminal elevation (masl)
1 and 1A	181.2	Potable	Longridge Reservoir	1,475.0
4 and 4B	203.9	Raw	Rustfontein Dam	1,377.0
1B (hybrid)	186.1	Potable	New command reservoir	1,625.0*
-1B (Rustfontein)	25.7	Potable	Rustfontein PS	1,385.9*
-1B (Longridge)	26.0	Potable	Longridge Reservoir	1,475.0*

* Preliminary elevations which were costed for comparison but further optimized after selection (Section 7.6)

7.5.2 Design Flow Rates

The first phase of the water resources yield modelling considered a maximum transfer volume of 60 million m^3/a from Gariep Dam, whereas the second phase of the modelling was aimed to determine the actual maximum transfer volume required from Gariep Dam for Schemes 1A, 1B and 4B (refer to Figure 7-6).

In terms of naming convention, and to distinguish between the original and increased transfer capacities, the schemes were renumbered as follows:

- Scheme 1 (60 million m^3/a) = Scheme 1A (increased transfer capacity)
- Scheme 4 (60 million m^3/a) = Scheme 4B (increased transfer capacity), and,
- Scheme 1B (increased transfer capacity)

Table 7-15 summarises the HFY of each scheme, the average and maximum monthly flows from each WTW and the percentage of the 2050 demand that can be supplied for each scheme. For comparative purposes, the information for Schemes 1 to 4 (i.e. transferring 60 million m^3/a) are also presented in Table 7-15 together with that for Schemes 1A, 1B and 4B.

The red text indicates that the scheme does not meet the 2050 demands and the green text indicates that the demands are met. The following observations are made from the information presented in Table 7-15:

- A total volume of 120 million m^3/a must be transferred from Gariep Dam for Scheme 1A and 1B to match the 2050 water demand of 186 million m^3/a, whereas 142 million m^3/a must be transferred for Scheme 4B to satisfy the 2050 demands,
- Scheme 1A can only satisfy 84% of the demands for Botshabelo and Thaba Nchu,
- Scheme 4B can supply 99.6% of the demands for Botshabelo and Thaba Nchu, and,
- Scheme 1B can supply 100% of the demands for the three large demand centres, i.e. Bloemfontein, Botshabelo and Thaba Nchu.

The flow rates shown in Table 7-15 includes a monthly peak factor and were accepted for the infrastructure sizing of the raw water transfer schemes. However, potable water infrastructure is based on peak week demands and must account for water treatment losses. Therefore, the flow rates used for the infrastructure sizing of the potable schemes were factored as follows:

- The LLPS and abstraction pipeline were sized to allow for 5% raw water loss at the WTW at maximum flow rates, and,
- A 1.13 factor was applied to the peak month factor for HLPS and pipeline sizing for a peak week factor of 1.30.

The flow rates used in the design of the bulk water transfer infrastructure from Gariep Dam to the scheme's termination for Schemes 1A, 1B and 4B are summarized in Table 7-16. For comparative purposes, the flow rates for Schemes 1 and 4 are also shown in Table 7-16.

Table 7-15: Table of WTW outflows of schemes assessed for this study

Scheme No.		Rustfontein WTW		Maselspoort WTW		Welbedacht WTW		Proposed new Xhariep WTW		Bloem-fontein	Botshabelo & Thaba Nchu
Description (maximum transfer flowrate)	**Historic firm yield (Mm³/a)**	**Avg Flow (Mℓ/d)**	**Max flow (Mℓ/d)**	**Avg Flow (Mℓ/d)**	**Max flow (Mℓ/d)**	**Avg Flow (Mℓ/d)**	**Max flow (Mℓ/d)**	**Avg Flow (Mℓ/d)**	**Max flow (Mℓ/d)**	**2050 Demands met (%)**	**2050 Demands met (%)**
Scheme 1: Potable water to Bloemfontein (60 Mm³/a)	131	60.0	189.2	71.3	110.0	111.7	137.9	122.1	164.2	59.1	84.3
Scheme 2: Raw water to Knellpoort Dam (60 Mm³/a)	119	166.1	188.8	66.6	184.0	83.2	137.9	-	-	44.3	92.6
Scheme 3: Raw water to Novo Outfall (60 Mm³/a)	120	173.1	188.8	60.9	183.9	85.1	137.9	-	-	43.1	96.2
Scheme 4: Raw water to Rustfontein Dam (60 Mm³/a)	134	174.2	189.2	77.5	184.0	105.3	137.9	-	-	55.2	97.1
Scheme 1A: Potable water to Bloemfontein (120 Mm³/a)	186	145.5	189.2	48.0	110.0	62.8	137.9	253.0	388.8	100.0	84.4
Scheme 4B: Raw water to Rustfontein Dam (142 Mm³/a)	186	184.2	313.4	182.9	184.0	132.1	137.9	0	0	100.0	99.6
Scheme 1B (hybrid): Potable water to Command res. (120 Mm³/a)	186	50.0	189.2	80.1	110.0	129.0	137.9	240.1	328.3	100.0	100.0

Table 7-16: Design flow rates for bulk water transfer infrastructure from Gariep Dam to scheme termination

Scheme No.	Maximum transfer flow rate	Average transfer flow rate	LLPS and abstraction	HLPS and bulk transfer pipeline
Calculation	Q_{max}	Q_{avg}	$Q_{abs} = 1.05 \times Q_{max}$	$Q_{des} = 1.13 \times Q_{max}$
Unit	m³/s	m³/s	m³/s	m³/s
Scheme 1	1.901	1.411	1.996	2.148
Scheme 4	1.901	1.431	NA	Q_{max} = 1.901
Scheme 1A	4.500	2.925	4.725	5.085
Scheme 4B	4.500	3.016	NA	Q_{max} = 4.500
Scheme 1B (hybrid)	3.800	2.776	3.990	4.294
-1B (Rustfontein)	2.186	1.607	NA	Q_{avg} = 1.607
-1B (Longridge)	2.193	1.489	NA	Q_{max} = 2.193

Scheme 1B requires the flow to split from the proposed command reservoir into two pipelines, one to Longridge Reservoir and one to the Rustfontein WTW. The average (normal operating) flow to the Rustfontein WTW was calculated by subtracting the average yield model flow from Rustfontein WTW (0.579 m³/s) from the 2050 demand flow rate required for Botshabelo and Thaba Nchu (2.186 m³/s) resulting in a flow rate of 1.607 m³/s (note that the 1.607 m³/s + 1.489 m³/s is slightly higher than the total average flow from Xhariep of 2.776 m³/s as the average flow to Rustfontein accounts for contributions from Rustfontein and Groothoek WTWs). The average flow rate was used as the design flow rate because the full demand was not intended to be supplied from the command reservoir.

The average flow to Longridge Reservoir, obtained from average monthly flow from the yield model, was 1.489 m³/s. The maximum flow to Longridge Reservoir was as assumed to be the remainder of the 3.800 m³/s maximum transfer flow rate when the Rustfontein gravity line is operating at its average flow rate of 1.607 m³/s, equating to 2.193 m³/s (189.5 Mℓ/d). Therefore, in conjunction with either the existing Welbedacht WTW supply (145 Mℓ/d) or upgraded Maselspoort WTW supply (120 Mℓ/d), the total Bloemfontein 2050 demand of 301 Mℓ/d will be satisfied.

7.5.3 Further Development of Preferred Sub-options

The hydraulic gradelines and infrastructure sizing for Scheme 1A (sub-option 3), Scheme 4B (sub-option 1) and Scheme 1B were updated for the increased flows as shown in Table 7-16. A table describing all configurations (sub-options) tested for Schemes 1A and 4B is presented in Appendix B.

Descriptions of the infrastructure components for the recommended sub-options for Schemes 1A, 1B and 4B are provided in Table 7-17. The selected sub-options shown in Table 7-17 were used in the financial assessment of the schemes detailed in Section 9.

Table 7-17: Infrastructure details of recommended scheme sub-options (Schemes 1A, 1B and 4B)

Scheme No.	Sub-option	Nominal diameters	Description of configuration	Minimum velocity (m/s)	Maximum velocity (m/s)
1A	3	1500, 1700	High-Lift pump station (HLPS) and rising main to reservoir (~CH51485 m). Gravity line to Longridge reservoir.	0.66	1.52

Scheme No.	Sub-option	Nominal diameters	Description of configuration	Minimum velocity (m/s)	Maximum velocity (m/s)
1B	-	Main 1400 Longridge 1200 Rustfontein 1100	HLPS and rising main to first reservoir (~CH50925 m). Gravity line to booster pump station (~151005 m) and rising main to command reservoir (~CH186119 m). Gravity to Longridge Reservoir and Rustfontein WTW.	0.93 2.03 1.77	1.52 2.03 1.81
4B	4	1400	HLPS and rising main to first reservoir (~CH51485 m). Gravity line to booster pump station (~135446 m) and rising main to second reservoir (~CH178024 m). Gravity to Rietfontein Dam.	1.01	1.59

Table 7-18 provides a summary of the duty points based on average and maximum flows for each of the recommended sub-options per scheme. The power required per pump station, excluding any provision for stand-by capacity, is also shown.

Table 7-18: Summary of proposed new pump station operating requirements for Schemes 1A, 1B and 4B

Scheme No.	HLPS Average duty point	HLPS Maximum duty point	HLPS power required	Booster Average duty point	Booster Maximum duty point	Booster power required	Annual transfer Volume
	ℓ/s | m	ℓ/S | m	kW	ℓ/s | m	ℓ/s | m	kW	Million m³/a
1A*	2925 | 356	5085 | 399	28,440	2925 | 89	5085 | 119	-	92,246,584
4B*	3016 | 367	4500 | 408	25,731	-	-	-	95,127,325
1B*	2776 | 362	4294 | 401	24,106	2776 | 210	4294 | 233	13,278	87,560,917

*Schemes 1A, 4B and 1B were modelled with dual pipelines in parallel. Therefore, the pumping head shown represents the head required through each pipe in the configuration.

A summary of the balancing reservoir sizing calculations and quantity provided for the recommended sub-option for each scheme is provided in Table 7-19.

Table 7-19: Summary of reservoir requirements for Schemes 1A, 1B and 4B

Scheme No.	Number of reservoirs	Design transfer flowrate	Storage volume required for 6 hours	Design storage volume
Calculation	-	Q_{des}	$S = 6 \times 3600 \times Q_{des}$	S_{des}
Unit	No.	m³/s	m³	Mℓ
1A	1	5.085	109,836	110
4B	2	4.500	97,200	98
1B (hybrid)	2	4.294	92,750	95

7.5.4 Schematics

Schematics of the final scheme configurations for Schemes 1A, 1B and 4B were developed. These schematics consist of the proposed new infrastructure to be priced for each scheme and also the 'common' upgrades which were not included in the cost estimates. A summary of all schematics developed for the pre-feasibility investigation is presented in Table 7-20 and included in Appendix A.

Table 7-20: Summary of schematics developed for pre-feasibility investigation

Scheme No.	Drawing No.	Drawing Title
1A	1002533-0000-DRG-CC-0008	Schematic of option 1A [Sc5b] potable water to Bloemfontein (Historic firm yield 186 Mm³/a)

Scheme No.	Drawing No.	Drawing Title
1B	1002533-0000-DRG-CC-0009	Schematic of option 1B [Sc5f] potable water to command reservoir (Historic firm yield 186 Mm³/a)
4B	1002533-0000-DRG-CC-0010	Schematic of option 4B [Sc5f] raw water to Rustfontein Dam (Historic firm yield 186 Mm³/a)

7.5.5 Additional Infrastructure and Operational Considerations

The utilisation of the existing infrastructure varied between schemes and the recommended sub-options. Therefore, when considering the capital and operating cost of each scheme, it was necessary to quantity the flows pumped and treated at all of the pump stations and WTW in the GBWSS, and to account for any additional infrastructure that will be required to supply the 2050 demands at the respective demand centres.

A summary of the additional pipeline and pump station infrastructure upgrades, as well as the average pumping duty, that need to be included in the costing of each scheme is provided in Table 7-21.

Table 7-21: Summary of additional pipe infrastructure and pumping operations associated with Schemes 1A, 1B and 4B

Scheme No.	Description	Pipeline length	Pipeline diameter	Operating pump duty	Maximum pump duty
	Units	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**
1A	Rustfontein pump operation	-	-	941 | 185	-
	Welbedacht pump operation	-	-	726 | 243	-
	Novo transfer operation	-	-	1,926 | 165	-
4B	Maselspoort new pipeline and pump station upgrades	33.5	800	706 | 217	570 | 210
	New pipeline from Rustfontein to Bloemfontein	50.2	1,000	63 | 98	920 | 242
	Rustfontein pump station upgrade and operation to Bloemfontein	-	-	63 | 98	1,440 | 156
	Rustfontein pump operation to Botshabelo	-	-	1,445 | 199	-
	Welbedacht pump operation	-	-	1,529 | 249	-
	Novo transfer operation	-	-	475 | 133	-
1B	Gravity pipeline to Longridge from command reservoir	26.0	1,200	-	-
	Gravity pipeline to Rustfontein from command reservoir	25.7	1,100	-	-
	Welbedacht pump operation	-	-	1,493 | 249	-
	Novo transfer operation	-	-	575 | 134	-

The capital and operating costs for the infrastructure listed above were quantified and priced using the same hydraulic analysis and costing model developed for the main bulk transfer pipeline. The precise horizontal alignments and vertical profiles of the existing pipelines were not available and were developed based on satellite imagery, available topographic data, and the terminal reservoir locations. The starting and terminal elevations were obtained from schematics provided in previous studies. These calculations were completed for comparative costing purposes only.

A summary of the new WTW infrastructure and WTW upgrades as well as the total treated volume required for Schemes 1A, 1B and 4B is provided in Table 7-22.

Table 7-22: Summary of water treatment infrastructure and treatment volumes included in Schemes 1A, 1B and 4B

Scheme No.	Description of scheme specific WTW upgrades required*	Scheme specific capacity increase (Mℓ/d)	Total annual volume treated at all WTWs (Million m³/a)
1A	Proposed new water treatment works	390	185.9
4B	Upgrade of 125 Mℓ/d at Rustfontein WTW and 64 Mℓ/d at Maselspoort WTW	189	182.2
1B	Proposed new water treatment works	330	182.2

*All capacity upgrades listed are over and above the 10 Mℓ/d upgrade to Maselspoort WTW and 89 Mℓ/d upgrade to Rustfontein WTW which are common to all schemes (See Section 7.2).

The scheme specific capacity increases represent the total additional treatment volume required by each scheme and was used to calculate the capital cost of the new or upgraded WTWs. The total annual treated volume is a summation of the average outflows from the new, upgraded and existing WTWs in the GBWSS over a one-year period. This was used to calculate the total treatment cost required by each scheme.

7.5.6 Comparison of Schemes 1A, 1B and 4B

Table 9-7, repeated below as Table 7-23 for ease as reference, shows the financial comparison of Schemes 1A, 1B and 4B based on transferring 120 million m^3/a, 120 million m^3/a and 142 million m^3/a from Gariep Dam, respectively. It is evident from Table 7-23 that:

- The net present values (NPVs) and unit reference values (URVs) of the three schemes are very similar (i.e. within 3% of each other),
- Scheme 4B (raw water supply to Rustfontein Dam) is marginally cheaper than Schemes 1A and 1B, but cannot supply potable water to the towns along the proposed pipeline route, and
- Scheme 1B is marginally cheaper than Scheme 1A.

Given the similar costs between the three schemes, and taking cognisance of the operational requirements noted in the technical meeting held on 2 November 2023 (refer to Section 7.4.6), Scheme 1B is recommended for implementation as it can satisfy 100% of the demands of the large demand centres (refer to Table 7-15).

Table 7-23: Results of financial comparison for additional schemes 1A, 4B and 1B (Hybrid) with maximum annual transfer of 142 Mm³/a

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Present Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme comparison of increased transfer at 142 Mm³ / annum	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**Mℓ**	**Mm³/annum**	**Mm³/annum**	**Mm³**	**(Million Rands)**	**(Million Rands)**	**(Million Rands)**	**R/m³**	**R/m³**	**%**
Scheme 1A [Sc5b] Potable water from Gariep Dam to Bloemfontein	181.2	2 x DN1500	2925 | 356	5085 | 399	2925 | 89	5085 | 119	390	185.9	92.2	2208	25120	21983	47103	9.96	21.33	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	941 | 185	-	-	-	-	-	29.7	2208	0	844	844	0.38	0.38	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	726 | 243	-	-	-	-	-	22.9	2208	0	856	856	0.39	0.39	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	1926 | 165	-	-	-	-	-	60.7	2208	0	1544	1544	0.70	0.70	-
										Total	**25120**	**25226**	**50347**	**11.42**	**22.80**	**+3.1**
Scheme 4B [Sc5f] Raw water from Gariep Dam to Rustfontein Dam	203.9	2 x DN1400 & DN1600	3016 | 367	4500 | 408	-	-	189	182.2	95.1	2208	21317	20941	42258	9.48	19.14	-
+ Maselspoort upgrades	33.5	DN800	706 | 217	570 | 210	-	-	Incl above	Incl above	22.3	2208	505	1034	1539	0.47	0.70	-
+ New pipeline from Rustfontein to Bloemfontein	50.2	DN1000	63 | 98	920 | 242	-	-	-	-	-	2208	914	199	1112	0.09	0.50	-
+ Rustfontein pump upgrades + operating cost (to Bloemfontein)	Varies	Equivalent DN1400	63 | 98	1440 | 156	-	-	-	-	2.0	2208	119	265	384	0.12	0.17	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1529 | 249	-	-	-	-	-	48.2	2208	0	1849	1849	0.84	0.84	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	475 | 133	-	-	-	-	-	15.0	2208	0	307	307	0.14	0.14	-
										Total	**22855**	**25992**	**48847**	**12**	**22.12**	**100**
Scheme 1B [Sc5b] Potable water from Gariep Dam to Rustfontein	186.1	2 x DN1400	2776 | 362	4294 | 401	2776 | 210	4294 | 233	330	182.2	87.5	2208	21846	24675	46521	11.17	21.07	-
+ Gravity pipeline to Longridge reservoir from command reservoir	26.0	DN1200	-	-	-	-	-	-	69.2	2208	663	144	808	0.07	0.37	-
+ Gravity pipeline to Rustfontein from command reservoir	25.7	DN1100	-	-	-	-	-	-	50.7	2208	590	128	718	0.06	0.33	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	-
										Total	**23099**	**27125**	**50224**	**12.28**	**22.75**	**+2.8**

7.5.7 Stakeholder Feedback on Schemes 1A, 1B and 4B

The results for Schemes 1A, 1B and 4B were presented at Project Steering Committee (PSC) Meeting No 3 held on 20 February 2024. Stakeholders were afforded the opportunity to comment on the findings and the recommendation to proceed with Scheme 1B. The PSC members supported the recommendation as Scheme 1B significantly increases the resilience of the GBWSS and can satisfy the 2050 demands of the three largest demand centres.

7.6 Design Optimisation of Scheme 1B

This section of the report considers the design optimisation of the various infrastructure components of Scheme 1B based on transferring a flow of 120 million m^3/a from Gariep Dam.

In parallel to the design optimisation, the water resources yield modelling was further refined for Scheme 1B and the stochastic yield analysis undertaken. The optimised Scheme 1B infrastructure will ultimately be refined based on the optimised yield to be transferred. Section 7.6.9 below presents the refined infrastructure to be taken forward to the detailed feasibility phase.

7.6.1 Supply to Low-Lift Pump Station

As-built drawings were obtained from DWS that show the approximate location of the existing DN2100 outlet pipeline from Gariep Dam to which this proposed scheme needs to connect. No coordinates were provided on the as-built drawings for the connection point. The exact tie-in point will therefore need to be determined once construction commences.

From this connection point, the pipeline will be extended to the site of the LLPS, from where the raw water will be pumped to the proposed Xhariep WTW. The location of the LLPS was investigated during the site inspection conducted by Zutari and DWS staff between 20 and 22 February 2023 as detailed in the Site Visit Report (DWS Report No. P WMA 06/D00/00/3423/2). The indicative footprint of the initial LLPS site (dashed blue), identified as part of the MMM investigations, and proposed new LLPS site (solid blue), identified during the February 2023 site visit, is shown in Figure 7-8.

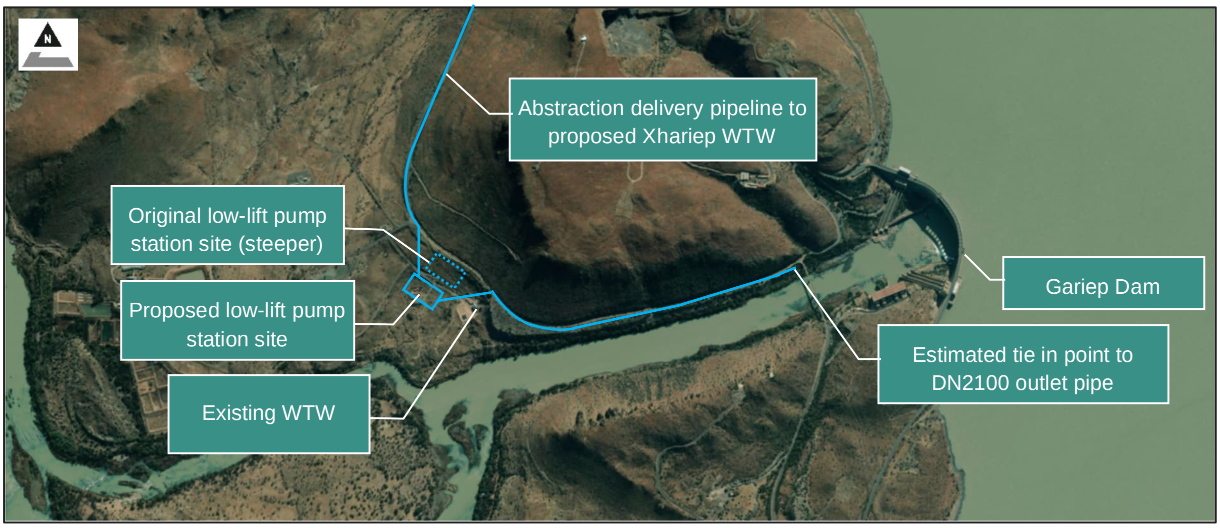


Figure 7-8: Indicative footprint of proposed Gariep Dam connection point and low-lift pump station sites

The proposed LLPS is located just north of the existing Gariep WTW. No floodline information could be obtained for the area downstream of Gariep Dam. It was, however, assumed that the preferred LLPS site lies above the 1:50 year flood line based on the similar elevation of the nearby Gariep WTW.

The initial LLPS site sits on a steep slope just below the road with a high point of roughly 1242 masl. The average elevation of this LLPS site is 1235 masl. An assessment of the historic water levels (refer to Figure 7-9) at the Gariep Dam, which were available for the period of December 1971 to August 2023, was undertaken to ascertain the expected pressure heads at the LLPS.

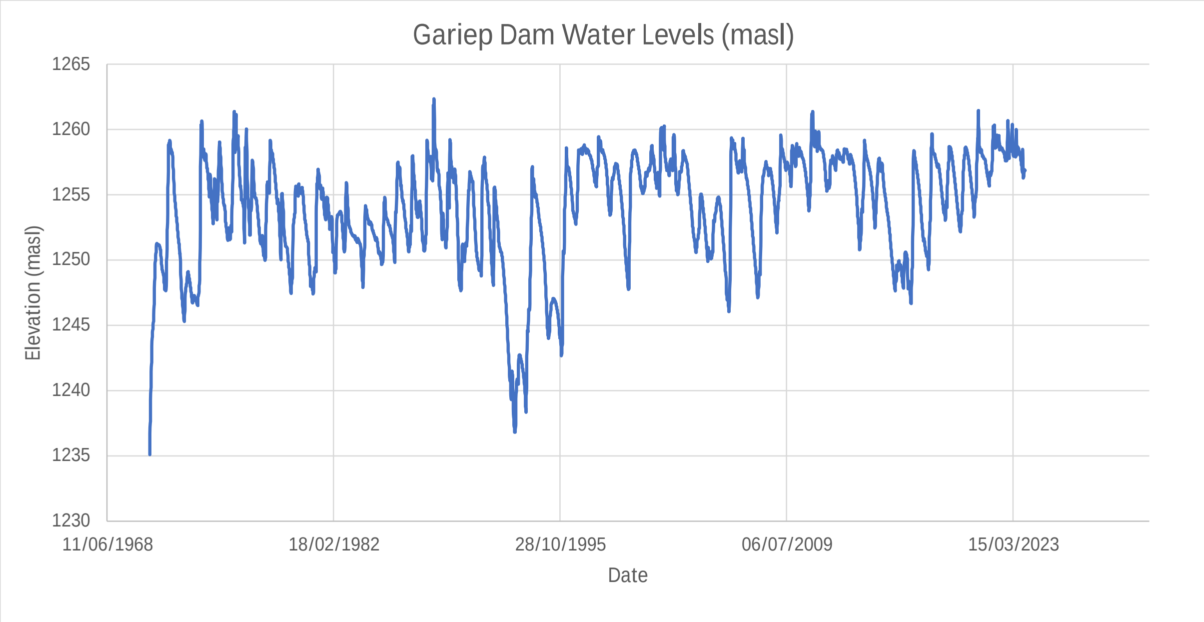


Figure 7-9: Historic water levels in Gariep Dam

The data shows that the water level has dropped below 1240 meter above sea level (masl) once since 1971 to a minimum level of 1236.8 masl in 1993. The water level typically ranges between 1246.7 masl and the full supply at level at 1258.7 masl with an average water level of 1254 masl. A maximum flood water level of 1262.4 masl was recorded at the dam in 1988.

Based on the historic water level information and considering the steep slope of the LLPS site identified in previous studies, a flatter site was identified just south of the initial LLPS site with an approximate elevation of 1215 masl.

The pipeline from the tie in point to the proposed LLPS is roughly 1.6 km long and its diameter will be in the range of DN1800 and DN2100 with a pressure class of PN10. These diameters are estimated using the preliminary design flow rate for the raw water abstraction pipeline of 3.99 m³/s (Q_{abs}) as shown in Table 7-16.

7.6.2 Optimisation of Scheme 1B configuration

The original configuration of Scheme 1B used in the options analysis to compare Schemes 1A, 1B and 4B, was based on a high point on a small mountain outcrop with an elevation of 1625 masl for the proposed command reservoir.

As part of the optimisation process for Scheme 1B, the reservoir site was further evaluated in terms of accessibility and constructability. It was concluded that this site does pose risks and that alternative command reservoir locations need to be identified and considered. The elevation of the original location was beneficial as it provided enough head to gravity feed both the Longridge Reservoir and Botshabelo. Two alternative configurations were investigated, namely Scheme 1B1 and Scheme 1B2 as shown in Figure 7-10.

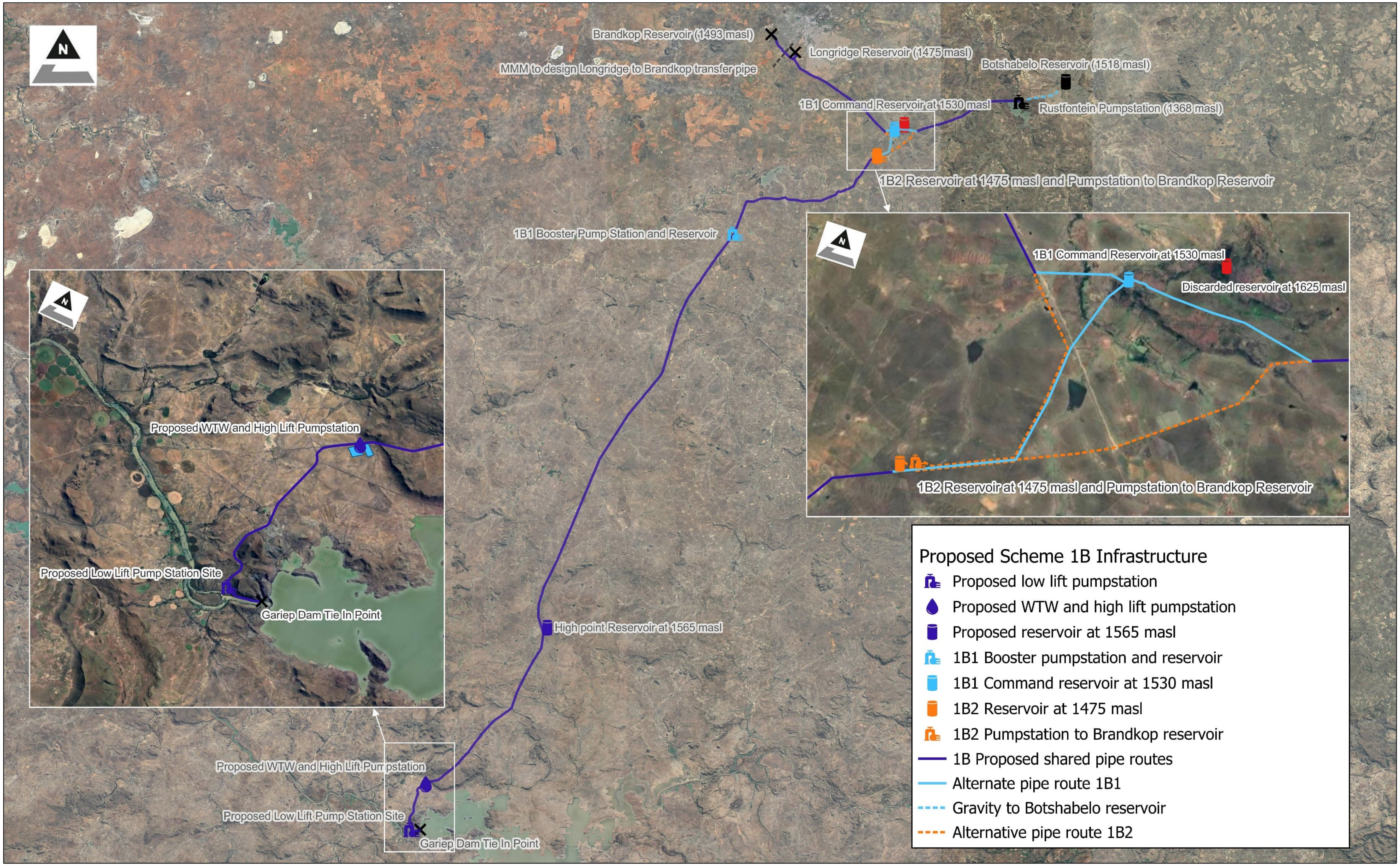


Figure 7-10: Map of alternative infrastructure configurations for hybrid scheme 1B

7.6.3 Updated Termination Points and Design Flow Rates

The previous scheme configurations included in the options analysis were based on the Bloemfontein termination point located at Longridge Reservoir with an elevation of 1475 masl, which is the battery limit of this project. During the optimisation of Scheme 1B, cognisance was taken of the elevation of Brandkop Reservoir (1493 masl) as well, as this will provide MMM with the option to extend the bulk infrastructure from Longridge Reservoir to Brandkop Reservoir, which could improve the operational flexibility and distribution of the additional water delivered to Bloemfontein.

The previous scheme configuration also had sufficient head with the command reservoir located at 1625 masl to feed directly to Botshabelo reservoir located at 1518 masl. The updated reservoir locations will require a booster pump station at Rustfontein WTW to pump the water to the Botshabelo reservoir.

The average flows to Bloemfontein and Botshabelo / Thaba Nchu (via Rustfontein) of 1.489 m^3/s and 1.607 m^3/s were obtained from the water resources yield model. Scheme 1B will supply 2.776 m^3/s of these average flows with the balance to Botshabelo and Thaba Nchu made up from Groothoek and Rustfontein WTW. The design flows used for the optimisation of Scheme 1B are shown in Table 7-24.

Table 7-24: Updated design flow rates for scheme 1B optimisation

Scheme No.	Maximum transfer flow rate	Average transfer flow rate	LLPS and abstraction	HLPS and bulk transfer pipeline
Calculation	Q_{max}	Q_{avg}	$Q_{abs} = 1.05 \times Q_{max}$	$Q_{des} = 1.13 \times Q_{max}$
Unit	m^3/s	m^3/s	m^3/s	m^3/s
Updated design flows for Scheme 1B optimisation				
1B (hybrid)	3.800	2.776	3.990	4.294
-1B (Rustfontein)	2.186	1.607	NA	Q_{max} = 2.186
-1B (Longridge/Brandkop)	2.193	1.489	NA	Q_{max} = 2.193

7.6.4 Alternative Configurations and Pipeline Routes

The infrastructure from the Gariep Dam connection to where the pipeline deviates away from the N1 road remained the same for all Scheme 1B configurations. This included the location of the LLPS, the proposed WTW and the first command reservoir with an elevation of 1565 masl. However, the two revised reservoir locations considered for the second command reservoir required different infrastructure configurations. Therefore, on approaching the regional road R702 in the northern reaches of the pipeline, two pipe routes were investigated to accommodate the alternative command reservoir locations. The pipe routes (not necessarily diameters) and bulk water infrastructure that is common to both Scheme 1B alternatives is shown in purple in Figure 7-10. A description of the infrastructure requirements for each alternative configurations is provided in Sections 7.6.4.1 and 7.6.4.2 below.

7.6.4.1 Scheme 1B Configuration 1 (1B1): Alternatives A & B

The first configuration shown in light blue on Figure 7-10 (labelled 1B1) makes use of a local high point with an elevation of 1530 masl for the command reservoir on the same mountain outcrop as the discarded reservoir location (shown in red). This configuration also required a reservoir and booster pump station adjacent to the N1 at approximate chainage of 144.2 km. From the second command reservoir water can gravity feed to Brandkop Reservoir at 1493 masl (Bloemfontein) via a 31.4 km DN1600 pipeline. There are two alternative configurations to supply Botshabelo and Thaba Nchu from the second command reservoir which are summarized as follows:

- **Alternative A**: 24.5 km DN1400 gravity pipeline from the command reservoir at 1530 masl to the proposed Rustfontein pump station (at 1368 masl) and pump water to Botshabelo via the existing DN648 and proposed 'common' DN1100 pipelines (Note – the DN1100 pipelines will be required as VCWB plans to upgrade the capacity of the Rustfontein WTW).
- **Alternative B**: 30.3 km DN2000 gravity pipeline from the command reservoir at 1530 masl to the Botshabelo reservoir at 1518 masl (additional dashed blue line on Figure 7-10).

Alternative A would rely on the Rustfontein pump station for distribution to the northern towns, whereas Alternative B would provide an independent supply line to Botshabelo. It will, however, with Alternative B still be necessary to install a third pipeline that can accommodate the increased flow from the planned Rustfontein WTW upgrade.

7.6.4.2 Scheme 1B Configuration 2 (1B2)

The second configuration shown in orange on Figure 7-10 (labelled 1B2) makes use of a local high point with an elevation of 1475 masl for the second command reservoir just south of the R702. Water would need to be pumped from the second command reservoir to Brandkop Reservoir at 1493 masl (Bloemfontein) via a 36.4 km DN1500 pipeline. Given the lower elevation of the second command reservoir, the supply line to the northern towns can only gravitate as far as the Rustfontein pump station via a 29.3 km DN1600 pipeline. Therefore, the supply to Botshabelo reservoir would need to be pumped from the Rustfontein pump station via the existing DN648 and proposed 'common' DN1100 pipelines as per Scheme 1B1 Alternative A.

7.6.5 Pipeline Hydraulics and Bulk Infrastructure Sizing

The method of hydraulic analysis and infrastructure sizing described in Section 7.3.3 was completed for each alternative scheme 1B configuration.

7.6.5.1 Scheme 1B Configuration 1 (1B1): Alternatives A & B

The hydraulic gradeline for the bulk water transfer pipeline from Gariep Dam to the proposed command reservoir for configuration 1B1 at maximum flow of 4.294 m^3/s (2.147 m^3/s per pipeline in parallel) is shown in Figure 7-11. The system consisted of a rising main to the first command reservoir at CH 52.7 km after which the water gravitates to a suction reservoir and booster pump station at CH 144.2 km from where a final rising main delivers the water to the second command reservoir at CH 186.7 km. It was shown that two DN1400 pipes in parallel could supply the required flow rates with a maximum velocity at peak flows of 1.52 m/s and minimum velocity at average flows of 0.93 m/s.

The high-lift pump station required for configuration 1B1 would need to fulfil a peak duty point of 4.294 m^3/s at 403 m of head and an average operating duty point of 2.776 m^3/s at 363 m of head. The booster pump station would require a peak duty point of 4.294 m^3/s at 138 m of head and an average operating duty point of 2.776 m^3/s at 106 m of head.

The maximum pressure class of the pipe would be PN63 for a short length at the start of the pipeline whereafter it reduces to PN40 and lower. The pressure classes from the first command reservoir to the second command reservoir varies from PN10 to PN25.

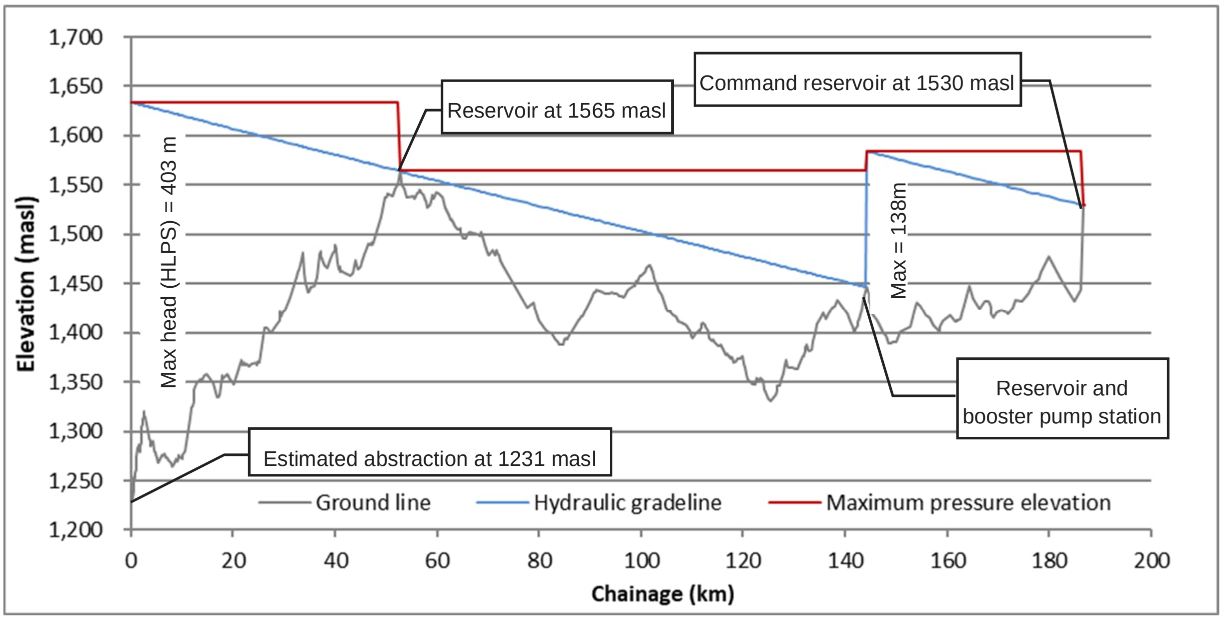


Figure 7-11: Hydraulic gradeline of alternative configuration 1B1 from Gariep Dam to proposed command reservoir at maximum flow (4.294 m³/s)

The hydraulic gradeline for the bulk water pipeline from the second command reservoir to Brandkop Reservoir at maximum flow is shown in Figure 7-12. In this configuration the water could gravitate via a 31.4 km DN1600 pipeline with a maximum velocity of 1.12 m/s at the peak flow of 2.193 m³/s and minimum velocity of 0.76 m/s at the average flow of 1.489 m³/s.

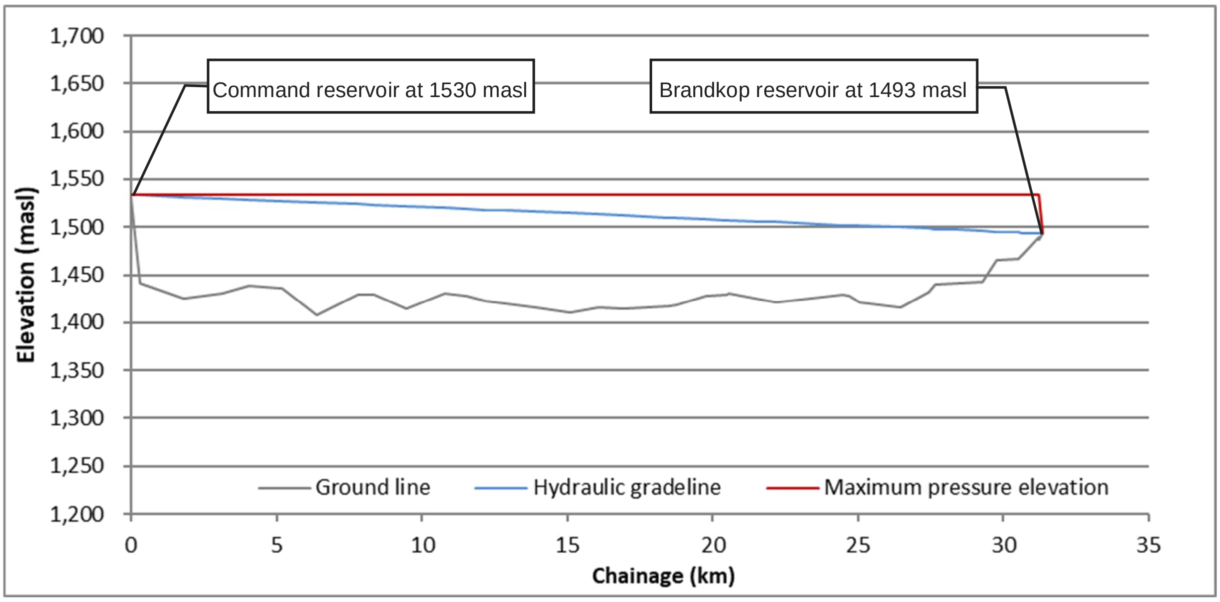


Figure 7-12: Hydraulic gradeline of alternative configuration 1B1 from proposed command reservoir to Brandkop reservoir (gravity) at maximum flow (2.193 m³/s)

The hydraulic gradeline for the bulk water pipeline from the second command reservoir to Rustfontein pump station at maximum flows is shown in Figure 7-13. In this configuration (alternative A) the water could gravitate via a 24.5 km DN1400 pipeline with a maximum velocity of 1.48 m/s at the peak flow of 2.186 m³/s and minimum velocity of 1.08 m/s at the average flow of 1.607 m³/s.

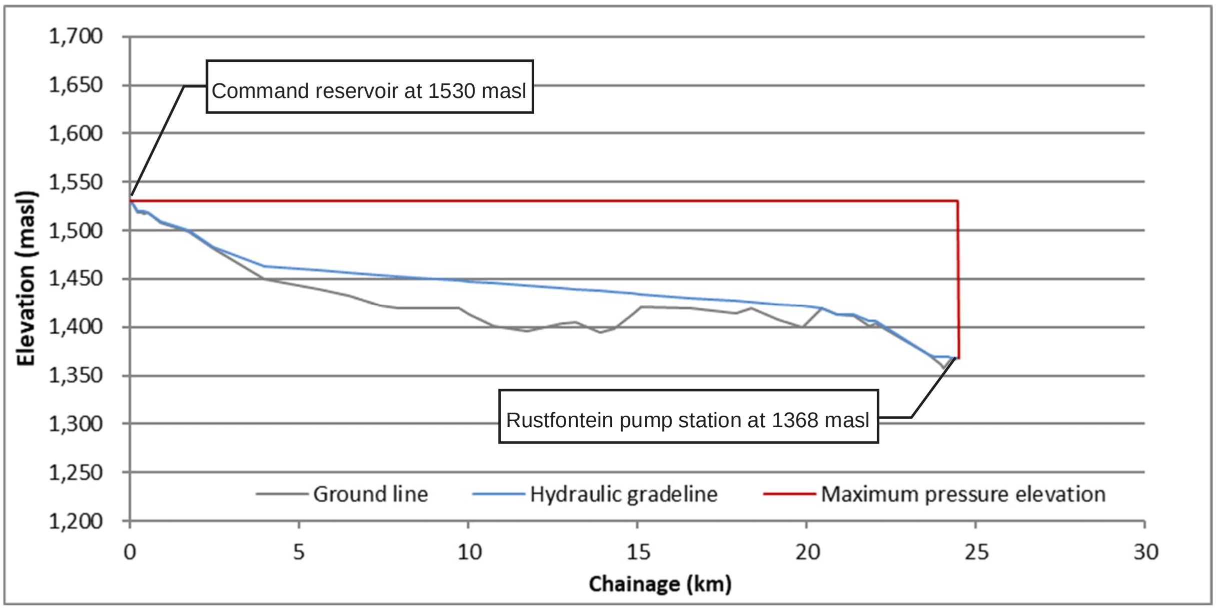


Figure 7-13: Hydraulic gradeline of alternative configuration 1B(A) from proposed command reservoir to Rustfontein pump station at maximum flow (2.186 m³/s)

The hydraulic gradeline for the bulk water pipeline from the second command reservoir to Botshabelo reservoir at maximum flows is shown in Figure 7-14. In this configuration (alternative B) the water could gravitate via a 30.3 km DN2000 pipeline with a maximum velocity of 0.71 m/s at the peak flow of 2.186 m³/s and minimum velocity of 0.52 m/s at the average flow of 1.607 m³/s.

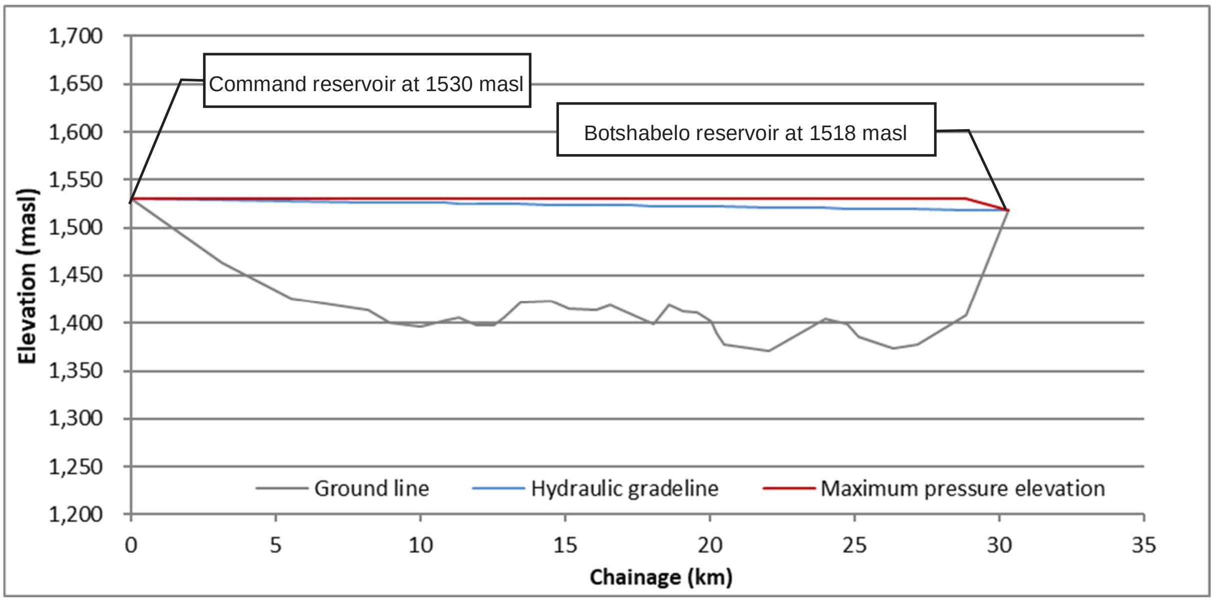


Figure 7-14: Hydraulic gradeline of alternative configuration 1B1(B) from proposed command reservoir to Botshabelo reservoir at maximum flow (2.186 m³/s)

It is important to note that the small elevation difference between the second command reservoir and the Botshabelo reservoir makes the pipeline sensitive to any additional head losses (such as minor losses) which may compromise the pipeline's ability to achieve the design flow rates.

7.6.5.2 Scheme 1B Configuration 2 (1B2)

The hydraulic gradeline for the bulk water transfer pipeline from Gariep Dam to the second command reservoir for Scheme 1B2 at maximum flow of 4.294 m³/s (2.147 m³/s per pipeline in parallel) is shown in Figure 7-15. The system consisted of a rising main to the first command reservoir at CH 52.7 km after which the water gravitates to the second command reservoir at CH 180.2 km. It was shown that two DN1400 pipes in parallel for the rising main and two DN1600 pipes in parallel for the gravity main could supply the required flow rates with a maximum velocity at peak flows of 1.52 m/s and minimum velocity at average flows of 0.71 m/s.

The high-lift pump station required for Scheme 1B2 would need to fulfil a peak duty point of 4.294 m³/s at 403 m of head and an average operating duty point of 2.776 m³/s at 363 m of head. The maximum pressure class of the pipe would be PN63 for a short length at the start of the pipeline whereafter it reduces to PN40 and lower. The pressure classes from the first command reservoir to the second command reservoir varies from PN10 to PN25.

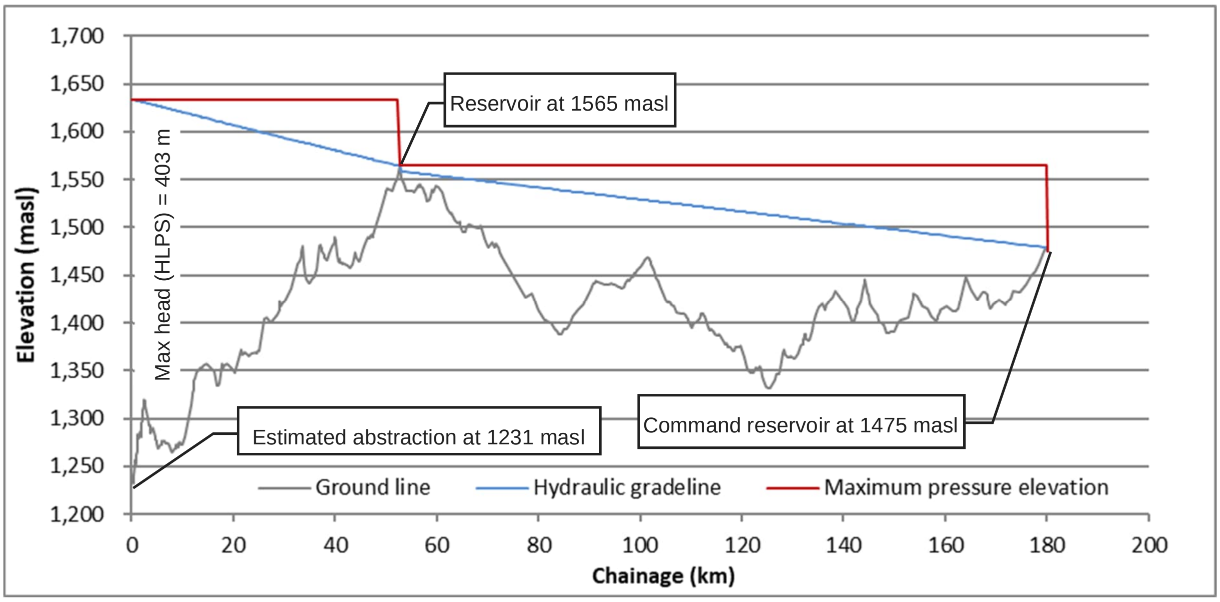


Figure 7-15: Hydraulic gradeline of alternative configuration 1B2 from Gariep Dam to proposed command reservoir at maximum flow (4.294 m³/s)

The hydraulic gradeline for the bulk water pipeline from the second command reservoir to Brandkop Reservoir at maximum flow is shown in Figure 7-16. In this configuration the water would be pumped via a 36.4 km DN1500 pipeline with a maximum velocity of 1.28 m/s at the peak flow of 2.193 m³/s and minimum velocity of 0.87 m/s at the average flow of 1.489 m³/s.

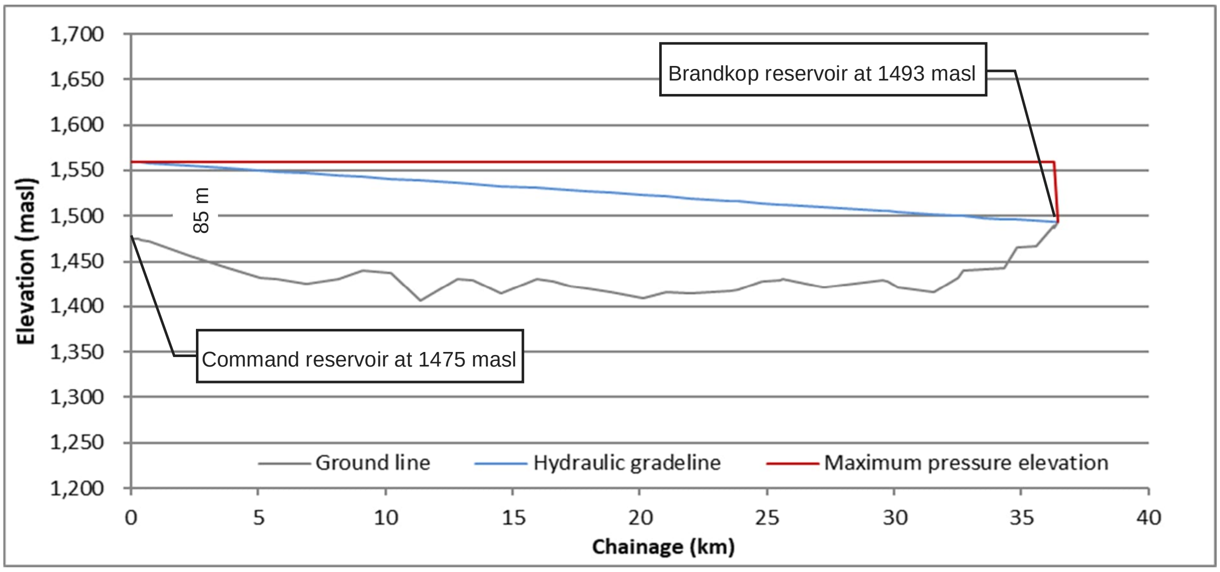


Figure 7-16: Hydraulic gradeline of alternative configuration 1B2 from proposed command reservoir to Brandkop reservoir (pumped) at maximum flow (2.193 m³/s)

The hydraulic gradeline for the bulk water pipeline from the second command reservoir to Rustfontein pump station at maximum flows is shown in Figure 7-17. In this configuration the water could gravitate via a 29.3 km DN1600 pipeline with a maximum velocity of 1.12 m/s at the peak flow of 2.186 m³/s and minimum velocity of 0.82 m/s at the average flow of 1.607 m³/s.

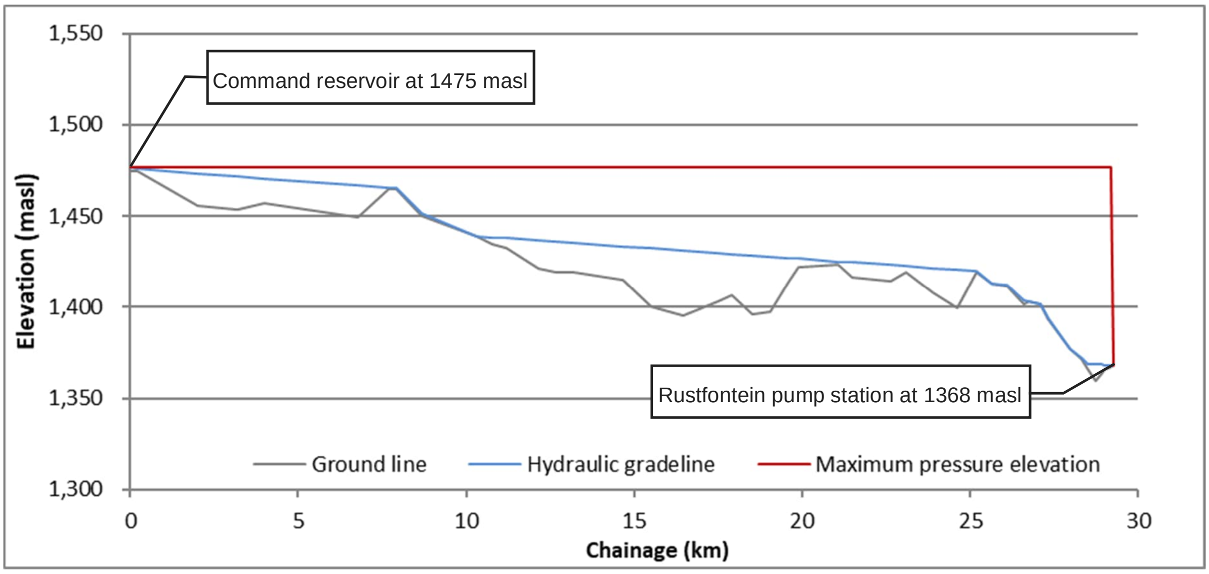


Figure 7-17: Hydraulic gradeline of alternative configuration 1B2 from proposed command reservoir to Rustfontein pump station at maximum flow (2.186 m³/s)

The gradelines at average flows of each branch of the scheme 1B2 configuration were calculated (not shown) and used to determine the expected average operating duty point of the pumps where rising mains are concerned.

7.6.6 Summary of Infrastructure Requirements

The infrastructure requirements were calculated and priced using the same hydraulic analysis and costing model developed for the initial options analysis. The pumping infrastructure required for each scheme was determined from the hydraulic gradeline calculations at maximum flows presented in Section 7.6.5. A summary of the pumping requirements for each configuration of Scheme 1B is provided in Table 7-25.

Table 7-25: Summary of pumping and power requirements calculated for Scheme 1B configurations

Scheme No.	HLPS Average duty point ℓ/s | m	HLPS Maximum duty point ℓ/S | m	HLPS power required kW	Booster Average duty point ℓ/s | m	Booster Maximum duty point ℓ/s | m	Booster power required kW	Annual transfer Volume m³
1B1	2776 | 363	4294 | 403	24,257	2776 | 106	4294 | 138	8,276	87,560,917
1B2	2776 | 363	4294 | 403	24,257	1489 | 49*	2193 | 85*	2,605*	69,158,448*

*These values are associated with the pump station to Brandkop Reservoir.

The gradelines at average flows of each branch of the Scheme 1B1 configuration were calculated (not shown) and used to determine the expected average operating duty point of the pumps on rising main branches as well as the 'minimum' operating velocities.

A summary of the pipeline and pump station infrastructure upgrades, as well as the operating duties included in the costing of each Scheme 1B configuration, is provided in Table 7-26.

Table 7-26: Summary of pipeline and pump station infrastructure and pumping operations associated with each configuration of Scheme 1B

Scheme No.	Description	Pipeline length	Pipeline diameter	Operating pump duty	Maximum pump duty
	Units	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**
1B1(A)	Bulk transfer pipeline from Gariep Dam to command	186.9	2 x 1400	Table 7-25	Table 7-25
	Gravity pipeline to Brandkop from command reservoir	31.4	1600	-	-
	Gravity pipeline to Rustfontein from command reservoir	24.5	1400	-	-
	Welbedacht pump operation	-		1,493 | 249	-
	Novo transfer operation	-	-	575 | 134	-
	Rustfontein pump operation to Thaba Nchu	-	-	1,445 | 199	
1B1(B)	Bulk transfer pipeline from Gariep Dam to command	186.9	2 x 1400	Table 7-25	Table 7-25
	Gravity pipeline to Brandkop from command reservoir	31.4	1600	-	-
	Gravity pipeline to Botshabelo from command reservoir	30.3	2000	-	-
	Welbedacht pump operation	-	-	1,493 | 249	-
	Novo transfer operation	-	-	575 | 134	-
	Rustfontein pump operation to Thaba Nchu	-	-	579 | 180	-
1B12	Bulk transfer pipeline from Gariep Dam to command	180.2	2 pipes of 1400 2 pipes of 1600	Table 7-25	Table 7-25
	Pumped pipeline to Brandkop from command reservoir	36.4	1500	1,489 | 49	2,193 | 85
	Gravity pipeline to Rustfontein from command reservoir	29.3	1600	-	-
	Welbedacht pump operation	-	-	1,493 | 249	-
	Novo transfer operation	-	-	575 | 134	-
	Rustfontein pump operation to Thaba Nchu	-	-	1,445 | 199	-

The following infrastructure is also included in the cost of each Scheme 1B configuration:

- Proposed new 330 Mℓ/d Xhariep WTW with total annual volume of water treated of 182.2 million m^3/a.
- Proposed 95 Mℓ reservoirs along the bulk transfer line (Three for scheme 1B1 and two for 1B2)
- Low lift pump station at abstraction works (maximum flow of 3.990 m^3/s)

Schematics were developed for the optimised scheme alternatives as listed in Table 7-27 and included in Appendix A.

Table 7-27: Summary of schematics developed for scheme 1B configurations

Scheme No.	Drawing No.	Drawing Title
1B1(A)	1002533-0000-DRG-CC-0011	Schematic of option 1B1A [Sc5f] potable water to 1530 masl command reservoir (Historic firm yield 186 Mm^3/a)

Scheme No.	Drawing No.	Drawing Title
1B1(B)	1002533-0000-DRG-CC-0012	Schematic of option 1B1B [Sc5f] potable water to 1530 masl command reservoir (Historic firm yield 186 Mm^3/a)
1B2	1002533-0000-DRG-CC-0013	Schematic of option 1B2 [Sc5f] potable water to 1475 masl command reservoir (Historic firm yield 186 Mm^3/a)

These schematics show the new bulk water infrastructure that was priced for each configuration investigated in the optimisation of scheme 1B.

7.6.7 Site Inspection of Scheme 1B Configurations

Apart from financial considerations, a site inspection was undertaken in January 2024 of the main infrastructure components of the two Scheme 1B configurations to evaluate aspects such as topography, access and constructability. The findings from the inspection can be summarized as follows:

1B (all): Low-Lift Pump Station Site

- The site originally proposed in the MMM study has a considerable slope and will require substantial hard rock excavation,
- The original site has a telephone line crossing it (unclear if still in use) and there is a small power line following the road on the northern border,
- An alternative site was identified to the west of the original site. Existing buildings are located on this site that may be impacted by the proposed pump station,
- The alternative site has a flatter slope that will be better suited for the proposed pump station, and,
- The alternative site has large overhead power lines located to the east that need to be considered.

Figure 7-18 shows the locations of the originally proposed and alternative sites for the low-lift pump station. Figure 7-19 is a photo of the two sites. The overhead powerlines are evident in the photo, as well as the existing structures located on the alternative site.

1B (all): Water Treatment Works and High-Lift Pump Station Site

- The site straddles two properties with different owners (WTW sludge ponds on southern property and WTW and HLPS on northern property) with a clearly defined fence line between them,
- The site to the north (WTW and high-lift pump station) was considerably more uneven with erosion channels and dongas. South of the fence is relatively even ground,
- Just north on the outside of the WTW site is an old dilapidated rectangular stone kraal packed with stone that must be checked for heritage significance,
- Two large power lines cross the middle of both the sludge ponds and WTW sites (66 kV and presumably 132 kV),
- It may be worth considering moving both sites to the south for them to fall on one property and to avoid the stormwater erosion evident on the northern property, and
- The property owner to the south was aware of the project and was very accommodating.

Figure 7-20 shows the location of the proposed WTW and HLPS, as well as the cadastral boundaries of the two properties under consideration.

Figure 7-21 and Figure 7-22 are photos of the northern and southern sites being considered for the WTW and HLPS.

The option to locate the WTW, HLPS and sludge lagoons/ponds on the southern property will be evaluated once the topographical information from the Lidar survey is received.

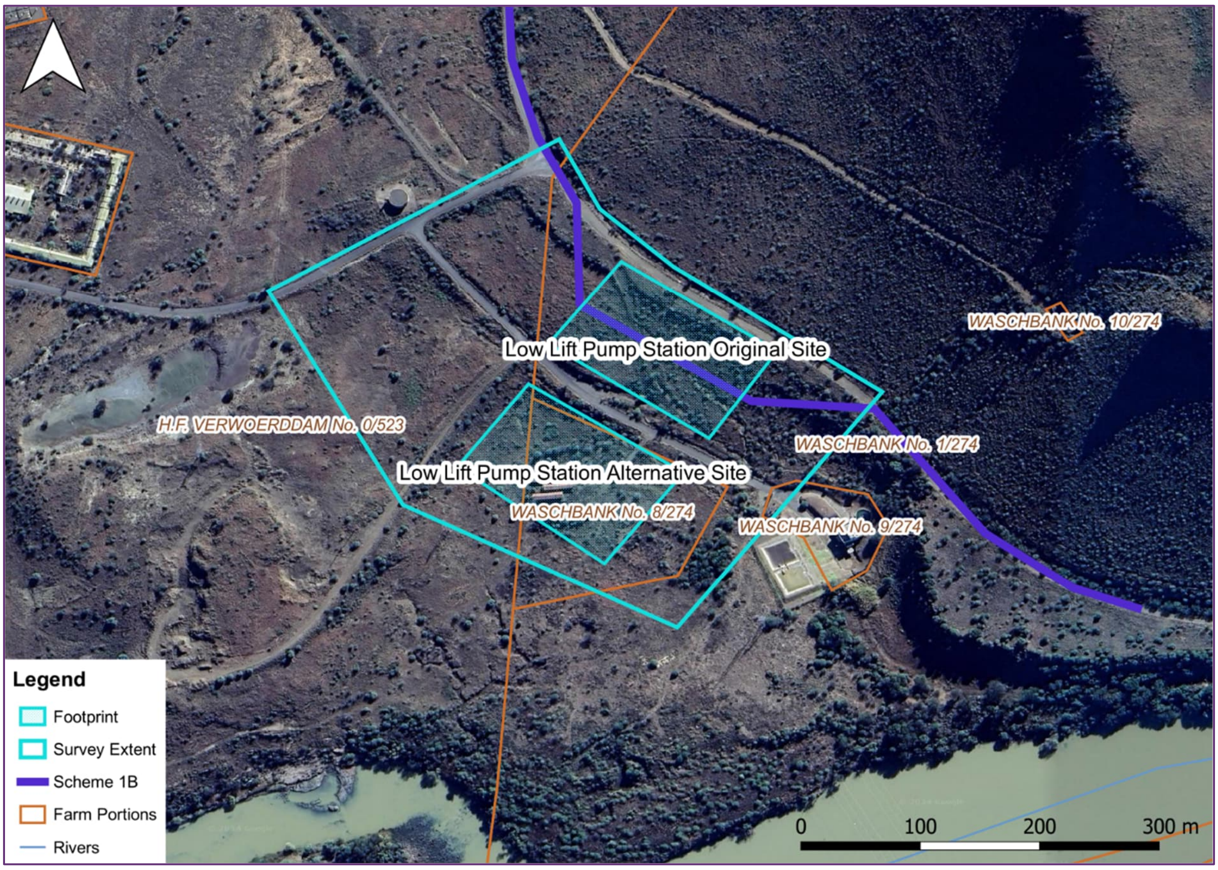


Figure 7-18: Location of originally proposed and alternative sites for low-lift pump station

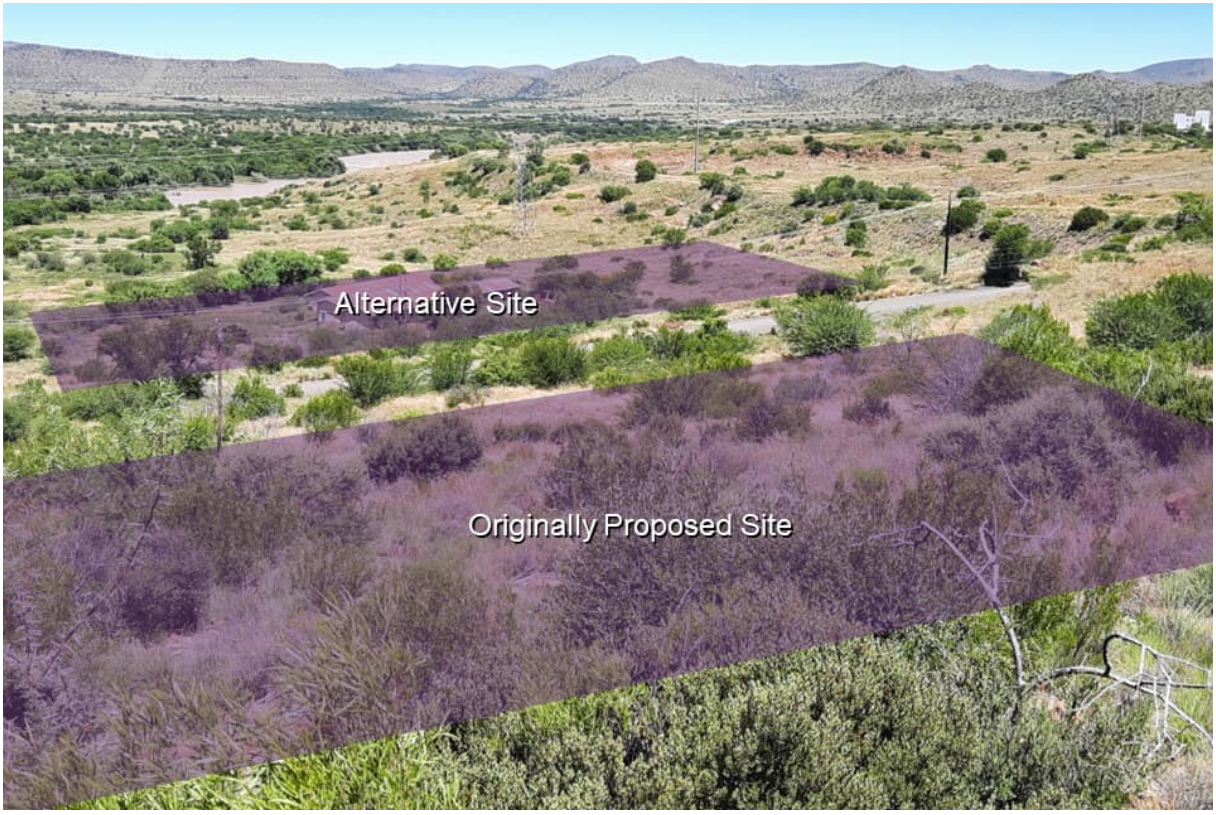


Figure 7-19: Photo of proposed and alternative sites for low-lift pump station

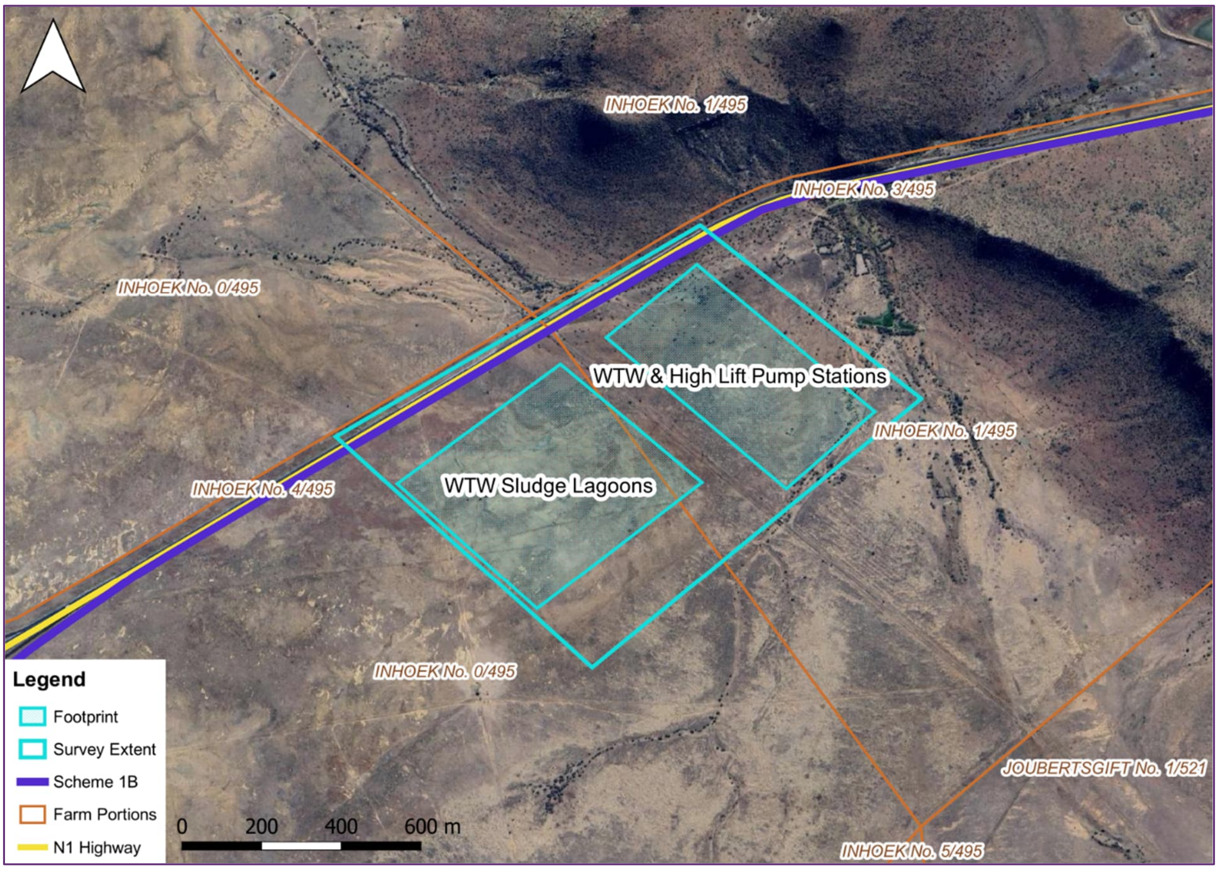


Figure 7-20: Location of proposed WTW and high-lift pump station site

Figure 7-21: Northern property considered for WTW and high-lift pump station site

Figure 7-22: Southern property considered for WTW and high-lift pump station site

1B (all): First Command Reservoir at 1565 masl

- No problems in terms of access or topography with this site were identified, and,
- Numerous succulents were encountered further up the hill. The environmental specialists will review this as part of their specialist studies, but these succulents were located above the proposed reservoir site.

Figure 7-23 shows the proposed location of the first command reservoir. Figure 7-24 shows a photo of the proposed location, as well as the gentle slope of the site, which is preferable when constructing an earth embankment type reservoir.

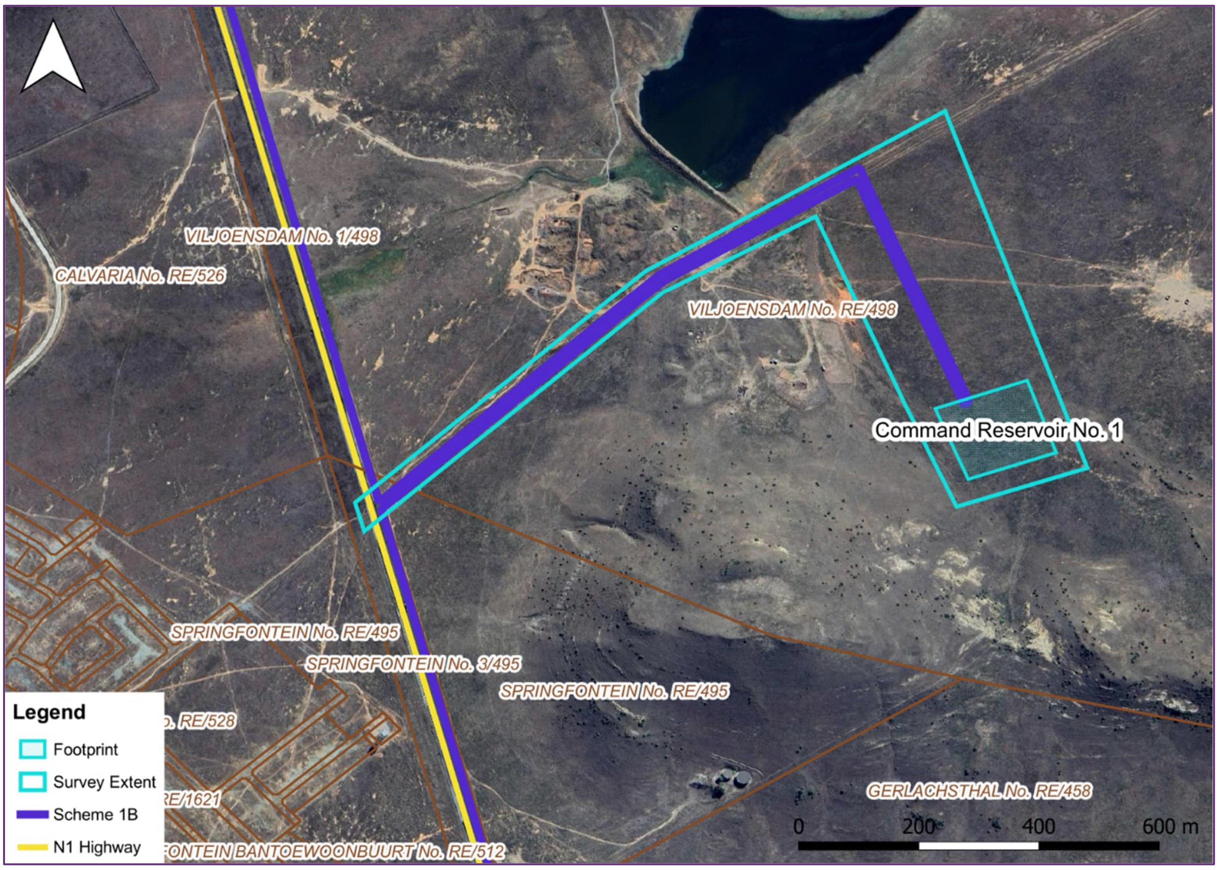


Figure 7-23: Proposed location of first command reservoir

Figure 7-24: Photo of proposed location for first command reservoir

1B1: Booster Pump Station (Adjacent to N1)

- No problems with this site were identified in terms of topography,
- Road access might need to be a distance from the proposed location since it is located on a localised high point with a blind rise from either side along the N1, and,
- There is a large power line approximately 500 m to the north of the site.

Figure 7-25 shows the proposed location of the booster pump station, with Figure 7-26 being a photo of the proposed site.

The option to locate the pump station slightly to the north or south of the currently proposed site will be evaluated once the topographical information from the Lidar survey becomes available.

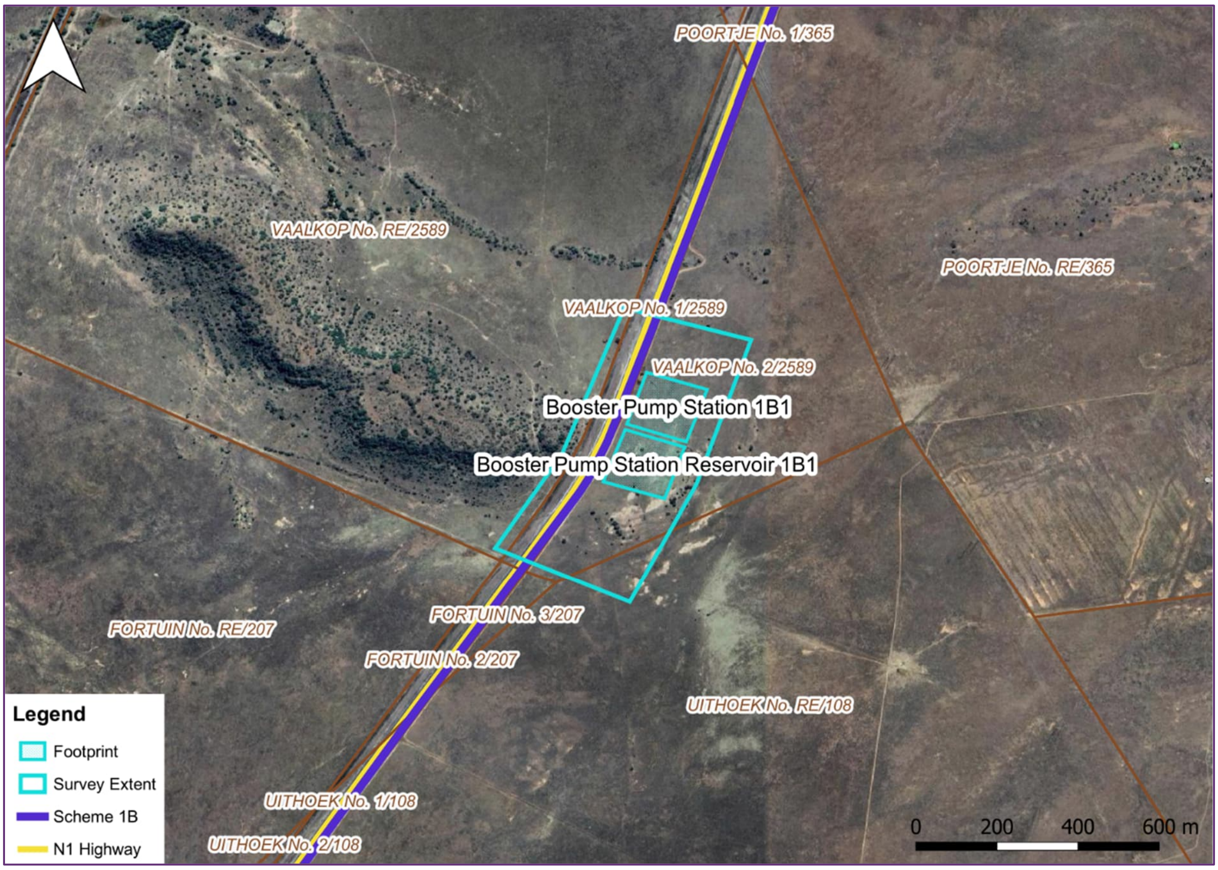


Figure 7-25: Proposed location of Booster Pump Station 1B1

Figure 7-26: Photo of proposed location for Booster Pump Station 1B1

1B1: Command Reservoir at 1530 masl

- The elevation of about 1530 masl will be achievable at the proposed site,
- The hillock has a sharp ridge towards the north-west but flattens out at the top,
- The access road will most likely need to be from the east which would require a longer servitude, and
- The farmer's house is located approximately 1 km from the proposed reservoir position, which will need to be considered in the design of the reservoir and the landowner engagements.

Figure 7-27 shows the proposed location of the second command reservoir. Figure 7-28 shows a photo of the proposed command reservoir site.

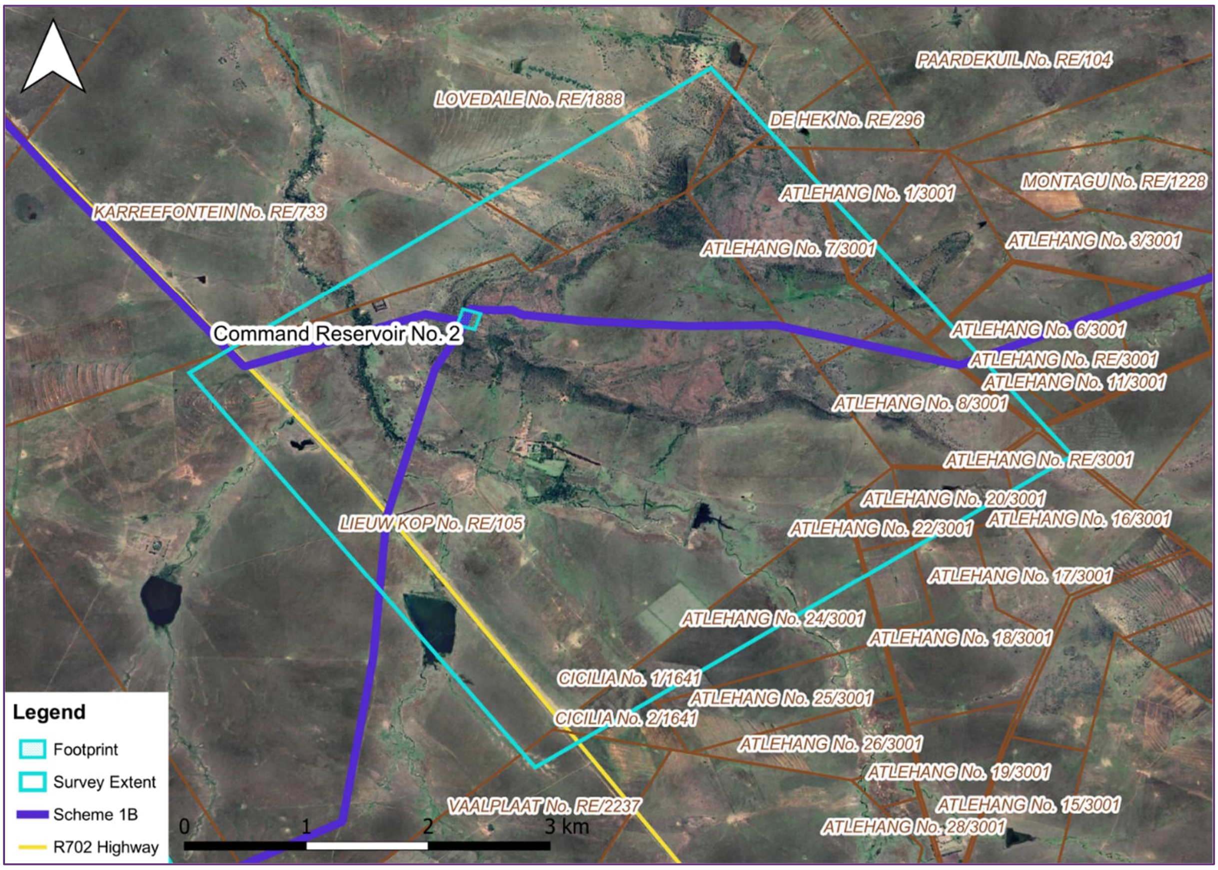


Figure 7-27: Proposed location of second command reservoir

Figure 7-28: Photo of proposed location for second command reservoir

1B2: Suction Reservoir and Booster Pump Station at 1475 masl

- The site is on a slightly raised section in the middle of an open field,
- No power lines could be seen from the site, and
- Road access to the site could be difficult with numerous fences and camps holding cattle and game.

Figure 7-29 shows the proposed location of the Booster Pump Station 1B2. Figure 7-30 shows a photo of the proposed site.

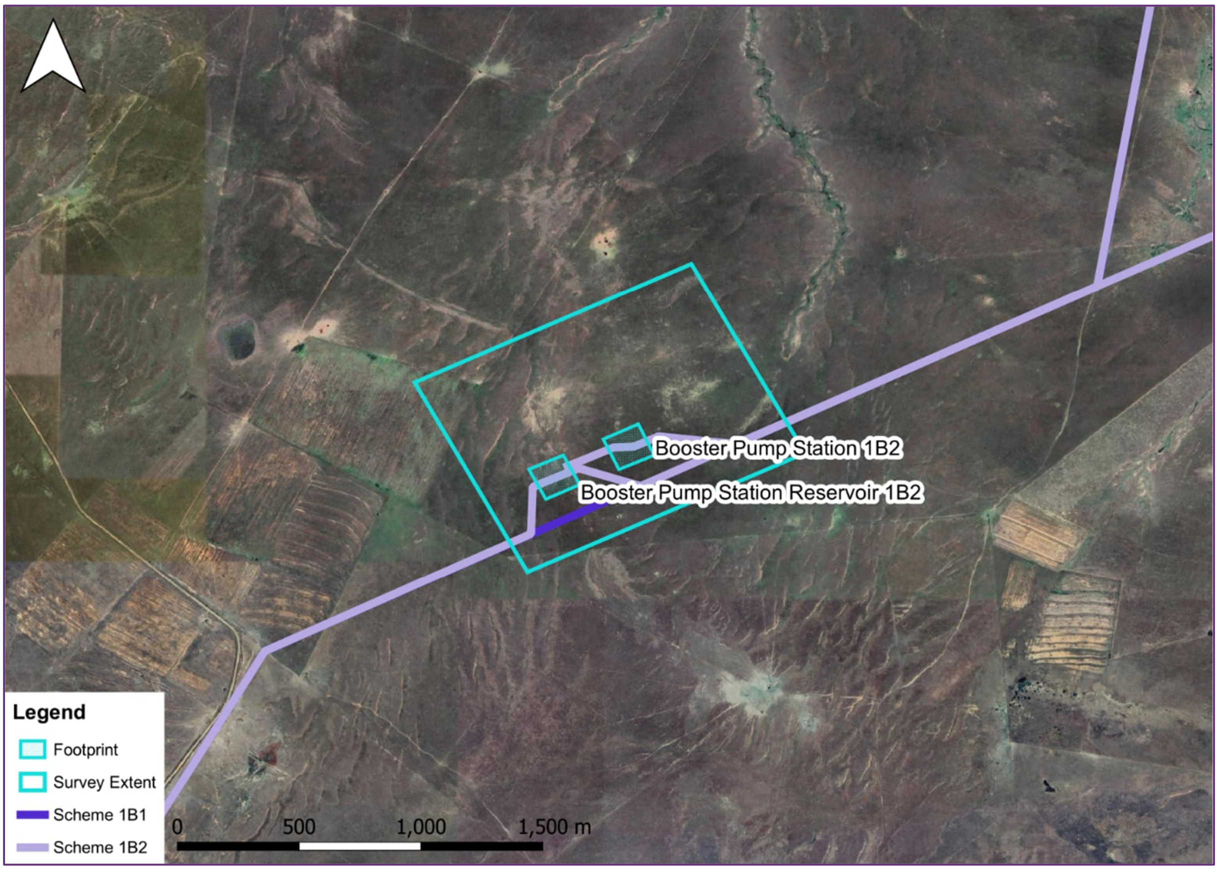


Figure 7-29: Location of Booster Pump Station 1B2

Figure 7-30: Photo of proposed location for suction reservoir and Booster Pump Station 1B2

Both command/suction reservoir sites for configurations 1B1 and 1B2 are located on the same property. The property owner was met with during the site visit, and he indicated that he was opposed to any infrastructure being located on this property.

Alternative locations for the command reservoir (elevation of 1530 masl) were considered with the nearest alternative location approximately 10 km to the south-east of the proposed command reservoir position (refer to Figure 7-31). This alternative location, which is located further away from Bloemfontein and Rustfontein WTW) would add approximately 20 km of additional pipelines and result in increased pumping cost due to these longer pipe lengths. This alternative location was therefore not further evaluated. Further engagements with the landowner will be undertaken as part of the environmental authorisation process.

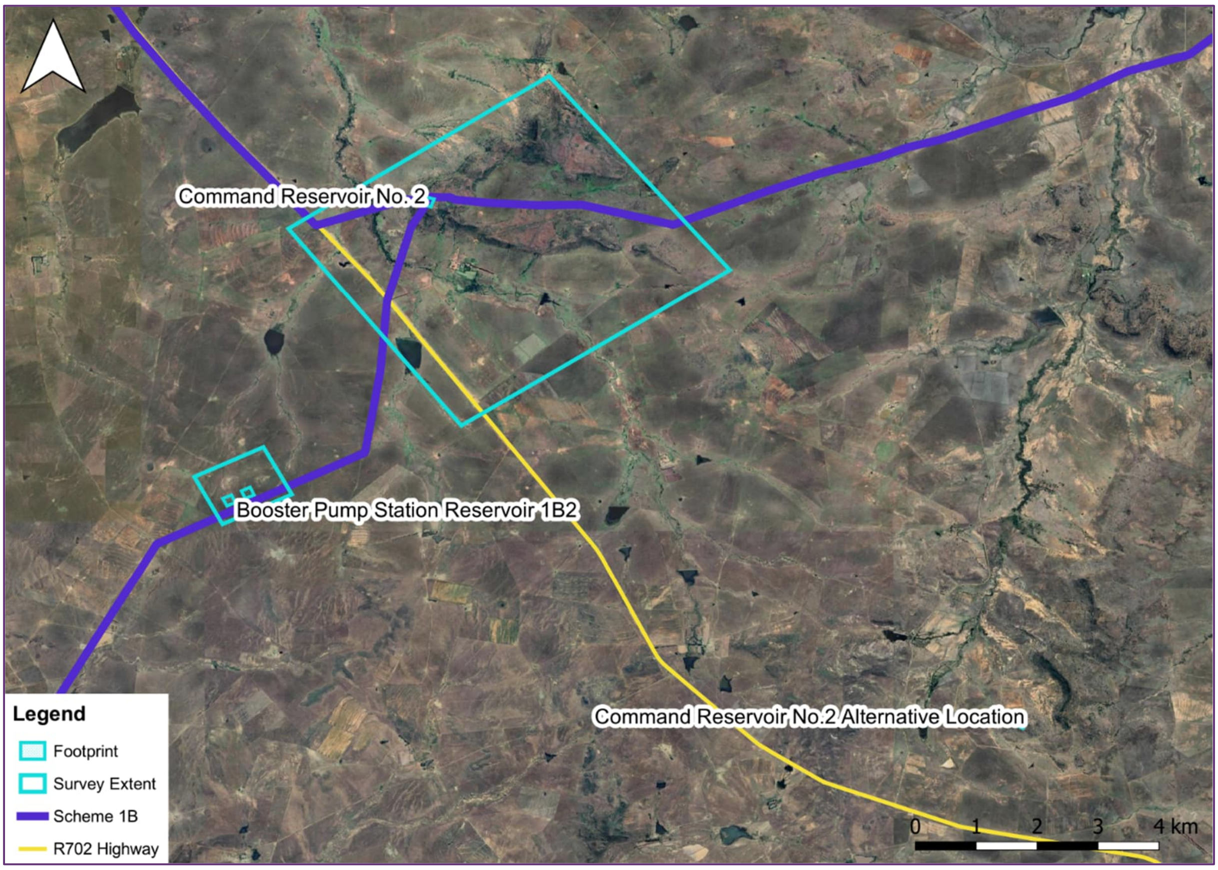


Figure 7-31: Alternative position for second command reservoir

7.6.8 Cost Comparison and Configuration Selection

It was concluded that all three configurations considered as part of Scheme 1B were technically feasible from a hydraulic perspective. The same hydraulic analysis and costing model as used for the original scheme comparison (Section 7.3.3) was used to compare the NPVs and URVs associated with each configuration. As a part of the financial assessment presented in Section 9.4.4, the NPV and URV of configuration 1B2 was shown to be the cheapest and was within 2% of both alternative configurations 1B1(A) and 1B1(B). Configuration 1B1(A) was shown to be marginally cheaper than 1B1(B). Additionally, the cost of all configurations was within 2% of the cost of discarded scheme 1B (with the second command reservoir at 1625 masl). Therefore, the configurations were considered equal in terms of financial feasibility and selection was based on the technical/practical assessment instead.

In light of the information gathered from the site inspection listed in Section 7.6.7, Scheme 1B1 was preferred over Scheme 1B2 based on the following:

- Road access to the pump station and reservoir sites appears more favourable in general,
- Connection to the Eskom power supply grid for the booster pump station adjacent to the N1 is likely to be easier than that of the pump station required in configuration 1B2, which is more remote from any existing Eskom infrastructure,
- Configuration 1B1 will likely be less disruptive to the affected property owners compared to configuration 1B2 that requires a suction reservoir and booster pump station on land where farming activities are undertaken,
- All mechanical and electrical infrastructure for configuration 1B1 is located adjacent to the N1, which provides easier access for operation, maintenance and security, and
- Configuration 1B1 allows flow to Bloemfontein and Rustfontein WTW to gravitate, whereas configuration 1B2 requires the flow to Bloemfontein to be pumped. A gravity supply to two

demand centres simplify the operation and also provides additional flexibility, e.g. more water can be supplied to Rustfontein WTW under gravity if Longridge Reservoir in Bloemfontein is full.

Therefore, configuration 1B1(A) is the considered the most feasible configuration and was selected for the detailed feasibility design. The infrastructure sizing and cost estimates were refined based on the final stochastic yield analysis results and are detailed in Section 7.6.9 and 7.6.10.

7.6.9 Updated Infrastructure Sizing Based on Stochastic Analysis (Pre-feasibility Design)

The flow rates obtained from the stochastic analysis described in Section 3.2 were factored as described in Section 7.3.2. However, for this final iteration of the pre-feasibility design the abstraction pipeline was analysed separately from the main transfer pipeline to provide a more accurate description of the separate infrastructure elements. The flow rates used to size the bulk water pipeline and pump station infrastructure for the pre-feasibility design of Scheme 1B are presented in Table 7-28 and Table 7-29.

Table 7-28: Table of WTW outflows of Scheme 1B (optimised)

Scheme No.		Rustfontein WTW		Maselspoort WTW		Welbedacht WTW		Proposed new Xhariep WTW		Bloem-fontein	Botshabelo & Thaba Nchu
Description (maximum transfer flowrate)	**Historic firm yield (Mm³/a)**	**Avg Flow (Mℓ/d)**	**Max flow (Mℓ/d)**	**Avg Flow (Mℓ/d)**	**Max flow (Mℓ/d)**	**Avg Flow (Mℓ/d)**	**Max flow (Mℓ/d)**	**Avg Flow (Mℓ/d)**	**Max flow (Mℓ/d)**	**2050 Demands met (%)**	**2050 Demands met (%)**
Scheme 1B (hybrid): Potable water to Command res. (120 Mm³/a)	186	50.0	189.2	80.1	110.0	129.0	137.9	240.1	328.3	100.0	100.0
Scheme 1B (hybrid) – Stochastic analysis Potable water to Command res. (101 Mm³/a)	186	52.3	188.8	71.2	110.0	92.0	103.5	273.6	276.5	100.0	100.0

Table 7-29: Design flow rates for pre-feasibility design of scheme 1B

Scheme No.	Maximum transfer flow rate	Average transfer flow rate	LLPS and abstraction	HLPS and bulk transfer pipeline
Calculation	Q_{max}	Q_{avg}	$Q_{abs} = 1.05 \times Q_{des}$	$Q_{des} = 1.13 \times Q_{max}$
Unit	m³/s	m³/s	m³/s	m³/s
1B (hybrid)	3.200	3.167*	3.797	3.616
-1B (Rustfontein)	2.149	1.456	NA	2.428
-1B (Brandkop)	3.200	1.630	NA	3.616

* This includes 7 Mℓ/d (0.081 m³/s) to supply the small towns Bethanie, Gariep Dam, Springfontein and Trompsburg from the Xhariep potable water transfer pipeline.

These design flows were populated in the hydraulic analysis and costing model to determine the bulk water transfer pipe sizes and to develop the hydraulic gradelines needed to size the pumps and calculate their power requirements. A summary of the treatment and bulk transfer infrastructure is provided in Table 7-30 and a summary of the pumping and power requirements in Table 7-31.

Table 7-30: Summary of treatment and bulk transfer infrastructure for pre-feasibility design of scheme 1B

Pipeline description	Pipeline length	Pipeline diameter	Min | Max velocity	Reservoirs	WTW
Units	km	mm	m/s | m/s	No. x size	Mℓ/d
Gariep Dam to LLPS pipeline	1.6	1800	1.34 | 1.53	-	-
Abstraction pipeline to WTW	10.5	1800	1.34 | 1.53	Table 7-25	-
Main transfer pipeline from WTW to command reservoir	176.4	1800	1.27 | 1.49	3 x 80 Mℓ/d	312
Gravity pipeline to Rustfontein from command	24.5	1400	0.98 | 1.64	-	-
Gravity pipeline to Brandkop from command	31.4	2000	0.53 | 1.17	-	-

The reservoirs were sized for 6 hours of storage at the peak week design flow rate (Q_{des}) and the WTW was sized based on the peak week design flow delivered over one day (24hrs x 3600s x $\frac{Qdes}{1000}$).

The hydraulic gradeline for the abstraction pipeline from the LLPS near Gariep Dam to the proposed WTW at maximum abstraction flows of 3.797 m³/s is shown in Figure 7-32. It was shown that one DN1800 pipe could supply the required flow rates with a maximum velocity at peak flows of 1.53 m/s and a minimum velocity at average flows of 1.34 m/s. The calculated maximum pumping head was 92 m from a starting elevation of 1231 masl (just below dead storage of 1233.1 masl at Gariep Dam). Therefore, the pump station would need to fulfil a peak duty point of 3.797 m³/s at 92 m head and an average operating duty point of 3.325 m³/s at 91 m of head when the Gariep Dam is providing the minimum elevation head possible (1233 masl minus ~2 m of friction head over 1.6 km suction line). A full assessment of the water levels and resulting pump operating points will be undertaken during the detailed feasibility design.

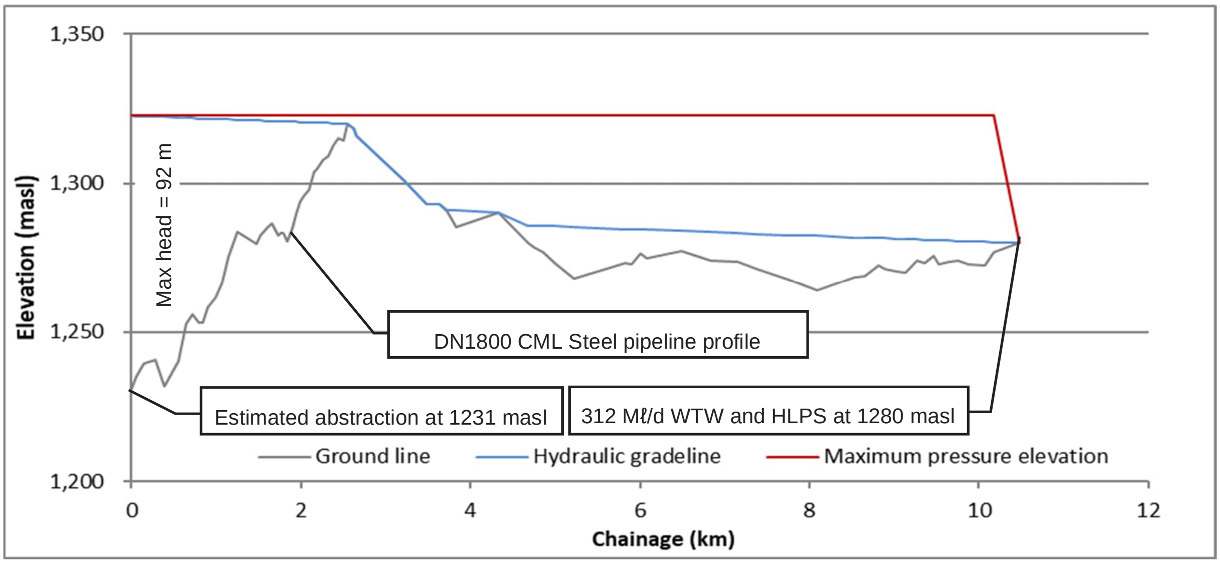


Figure 7-32: Hydraulic gradeline of abstraction pipeline from Gariep Dam to proposed WTW at maximum flow (3.797 m³/s)

The hydraulic gradeline for the bulk water transfer pipeline from the proposed WTW to the second command reservoir at maximum design flow of 3.616 m³/s is shown in Figure 7-33. The pipeline consisted of a rising main to the first command reservoir at CH 42.2 km after which the water gravitated to a suction reservoir and booster pump station at CH 133.6 km from where water is pumped to the second command reservoir at CH 176.4 km. It was shown that one DN1800 pipe could supply the required flow rates with a maximum velocity at peak flows of 1.49 m/s and minimum velocity at average flows of 1.27 m/s.

The high-lift pump station would need to fulfil a peak duty point of 3.616 m³/s at 325 m head and an average operating duty point of 3.167 m³/s at 315 m of head. The booster pump station would require a peak duty point of 3.616 m³/s at 124 m head and an average operating duty point of 3.167 m³/s at 114 m head.

The maximum pressure class of the pipe would be PN40.

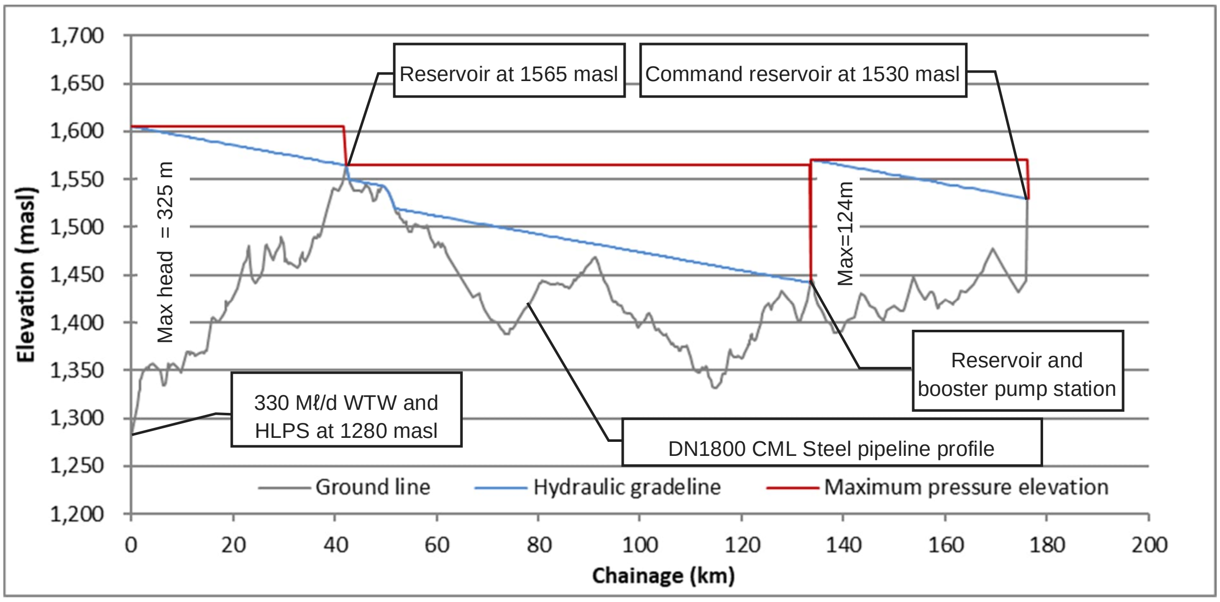


Figure 7-33: Hydraulic gradeline of bulk transfer pipeline from proposed WTW to command reservoir at maximum flow (3.616 m³/s)

The hydraulic gradeline for the bulk water pipeline from the second command reservoir to Rustfontein pump station at maximum flows is shown in Figure 7-34. The water could gravitate via a 24.5 km

DN1400 pipeline with a maximum velocity of 1.64 m/s at the peak flow of 2.428 m³/s and minimum velocity of 0.98 m/s at the average flow of 1.456 m³/s.

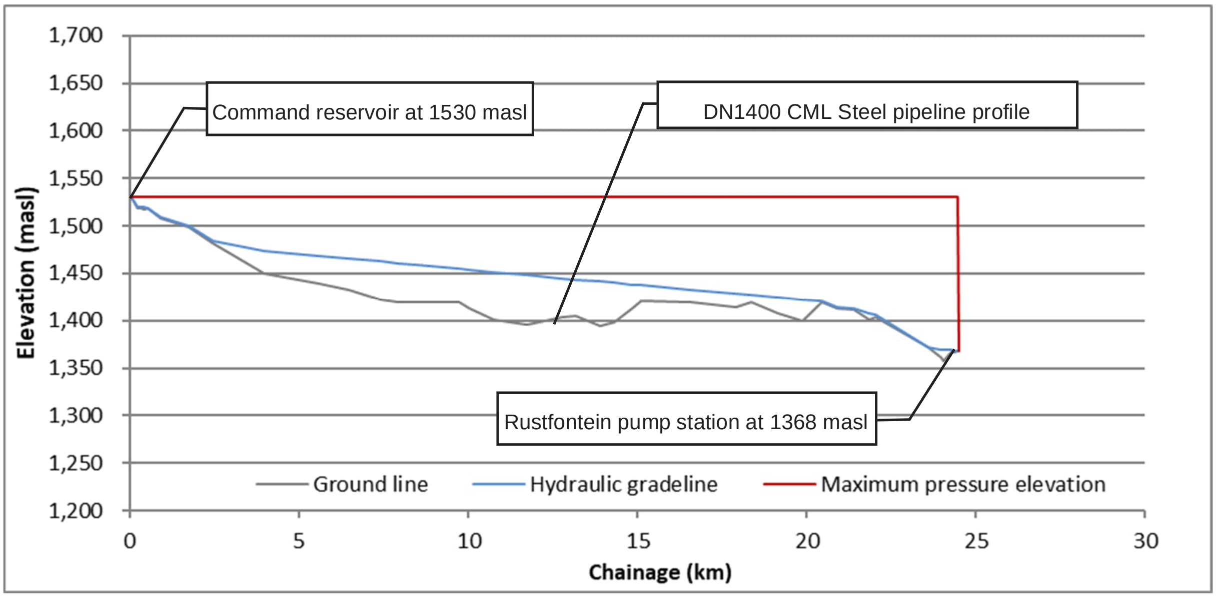


Figure 7-34: Hydraulic gradeline of gravity pipeline from second command reservoir to Rustfontein pump station at maximum flow (2.428 m³/s)

The hydraulic gradeline for the bulk water pipeline from the second command reservoir to Brandkop Reservoir at maximum flow is shown in Figure 7-35. The water could gravitate via a 31.4 km DN2000 pipeline with a maximum velocity of 1.17 m/s at the peak flow of 2.428 m³/s and minimum velocity of 0.53 m/s at the average flow of 1.630 m³/s.

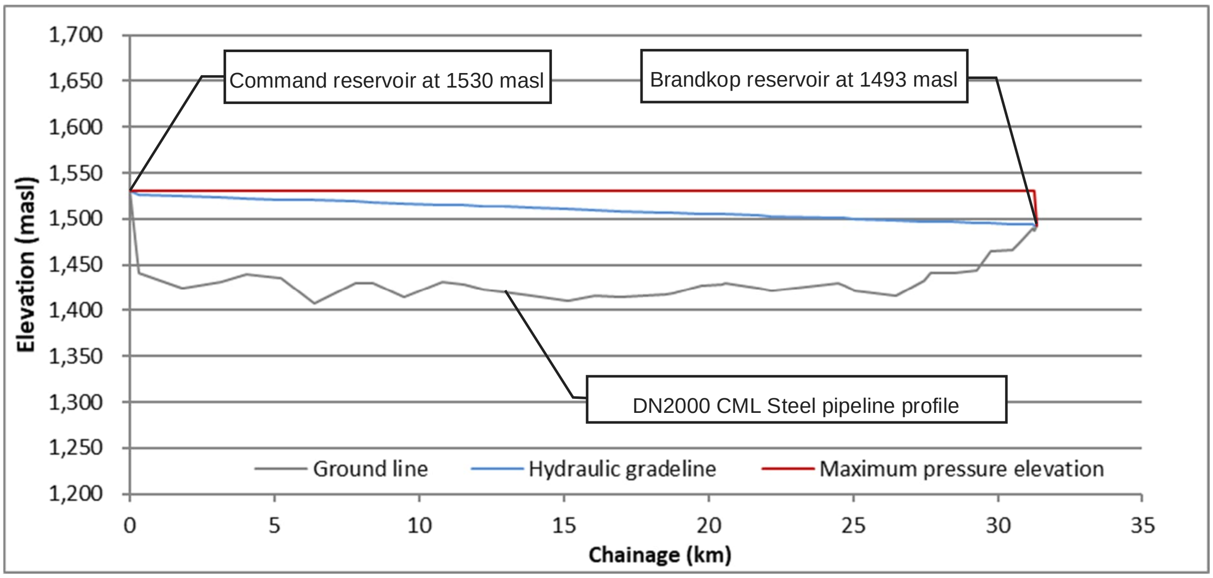


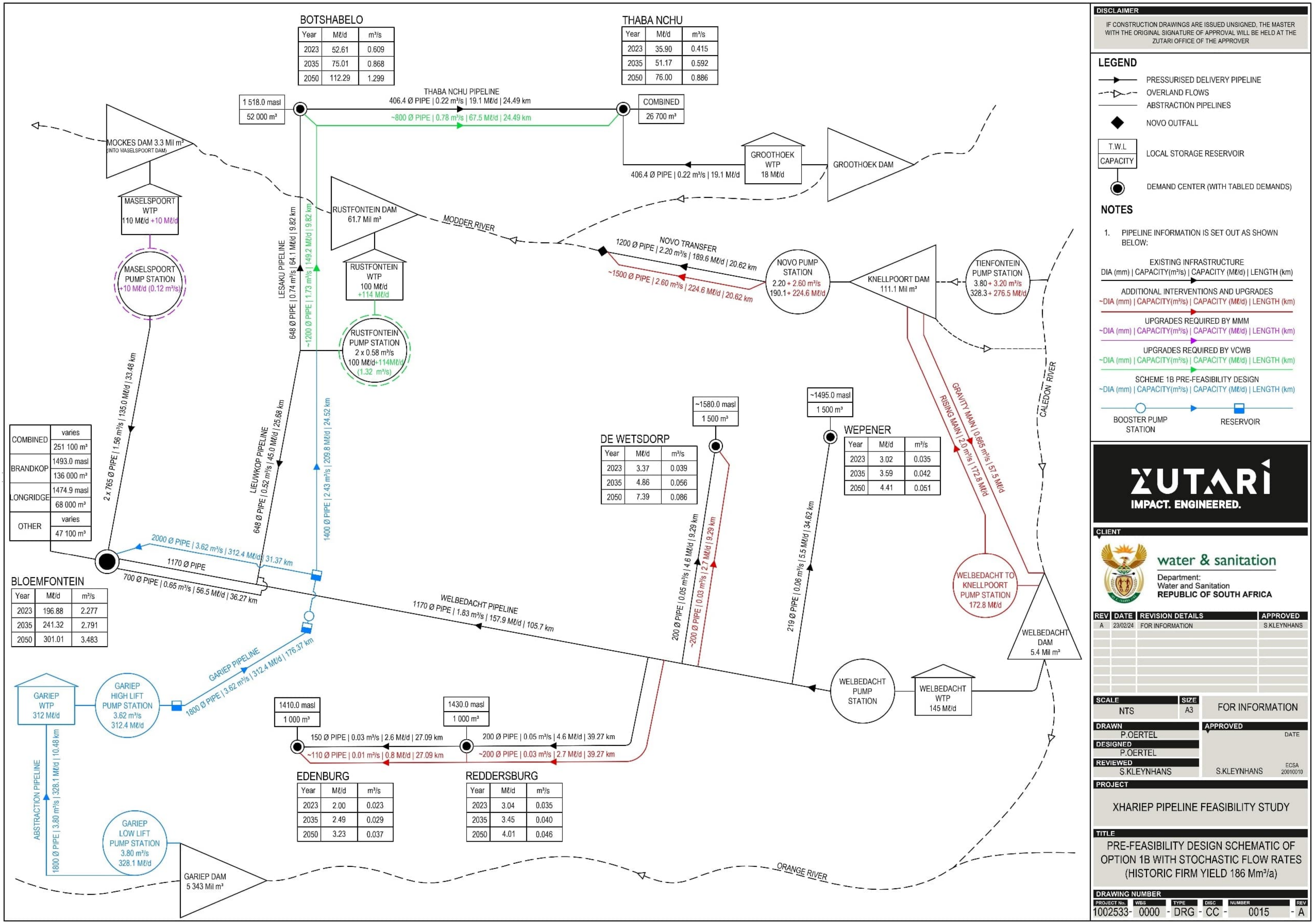
Figure 7-35: Hydraulic gradeline of gravity pipeline from second command reservoir to Brandkop Reservoir at maximum flow (3.616 m³/s)

The gradelines at average flows were calculated (not shown) to determine the expected average operating duty point of the pumps where rising mains are concerned. A summary of the pump station duty points and power requirements is provided in Table 7-31.

Table 7-31: Summary of pumping and power requirements for pre-feasibility design of scheme 1B

Scheme 1B pump station description	Average duty point ℓ/s | m	Maximum duty point ℓ/s | m	Peak power kW	Average Annual transfer Volume m³
LLPS	3325 | 91	3797 | 92	4,882	104,868,238
HLPS at WTW	3167 | 315	3616 | 325	16,467	99,874,512
Booster pump station	3167 | 114	3616 | 124	6,267	99,874,512

It was necessary to indicate to both the MMM and VCWB the infrastructure upgrades required by them to meet the 2050 demands for the entire GBWSS. Therefore, the peak week factor of 1.13 was applied to the 2050 demands of Botshabelo and Thaba Nchu and the Rustfontein WTW and pipeline upgrades were updated for the pre-feasibility design schematic presented in Figure 7-36. These upgrades are shown separately (purple and green) to the 'common' upgrades (red) shown on the previous schematics but are still excluded from the costing of the pre-feasibility design. Only the proposed infrastructure shown for Scheme 1B (blue) was included in the cost estimate in Section 7.6.10.



BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

Figure 7-36: Schematic of pre-feasibility design for scheme 1B

7.6.10 Cost Estimate for Pre-feasibility Design of Scheme 1B

A cost model was developed for the financial comparison of the different schemes investigated in this pre-feasibility study. The same model was used to estimate the capital and operational expenditure for the pre-feasibility design of Scheme 1B. However, a more detailed cost estimate of the water treatment works was undertaken, as described in Section 7.7, which was used in the cost estimate for the project. The main pricing assumptions made for the project cost estimate are summarised as follows:

Pipelines

- Steel pipelines were assumed to be Grade X52 with a 15 mm cement mortar lining. A maximum yield strength of 360 MPa was used to calculate the required wall thickness to resist hoop stresses for each section of the pipeline.
- The cost of the pipelines was based on a steel price of R35/kg and steel density of 7890 kg/m³ with an additional R3/kg included to account for lining and coating costs. These unit costs were developed from prices obtained from steel pipe manufacturers at the time of costing.
- Excavation dimensions for pipe installation were based on SANS 1200DB and backfill requirements were based on SANS 1200LB. Cover to pipe was assumed to be 1.5 m and 20% percent of excavations were assumed to be in hard rock.
- The cost of pipe equipment such as specials, valves, chambers and road crossings were assumed to be 85% of the total steel supply cost.

Pump stations and reservoirs

- A fixed lump sum of R30 million was allowed for the civil works of each pump station which includes the building and associated access road.
- A fixed rate of R1.5 million per Mℓ was allowed for the construction of the earth embankment reservoirs with a concrete lining and concrete roof. Therefore, for the pre-feasibility cost estimate, R120 million was allowed for each 80 Mℓ reservoir.
- The capital cost of the mechanical and electrical installations at the pump stations were based on R30 000/kW installed and priced using the peak power required at each pump station.

Water treatment works (WTW)

- The costing of the WTWs presented in Section 7.7.9 supersedes the rates developed in the hydraulic analysis and costing model.
- The WTW was priced for 312 Mℓ/d potable supply. However, the site footprint and civil building elements were designed for 400 Mℓ/d to accommodate future expansion.

Net Present Value (NPV) and Unit Reference Value (URV)

- The discount rate is defined as:
- $Discount\ rate = \frac{(r-i)}{(1+i)}$
- with r = return on investment (or average bond rate), and i = rate of inflation
-
- A discount rate of 4% was applied to all operational and maintenance costs over a discount period of 30 years.
- The discount rate is based on an average bond rate of 9% and average inflation rate of 5% in South Africa.
- The discount period was reduced to 30 years from the 45 years based on the recommendation from the economic specialist. It was noted that discount periods longer than 30 years can result in an under-estimation of the NPV and URV cost of the project.
- The present value energy cost is based on R1.80 per kWh and was converted to 0.701 cents/m³/m/year by calculating the total annual power required at the average operating duty point. A NPV rate of 16.48 cents/m³/m was calculated for the 30-year time horizon

including 2% annual growth above inflation to account for uncertainties in South Africa's future energy supply costs.

- URVs were calculated by dividing the total NPV project cost by the NPV of the annual scheme water demand using a discount rate of 4% over 30 years. It was assumed that the scheme demand would increase linearly to match the projected 2035 demand and then again up to the projected demand for 2050, after which it remained constant.

Table 7-32: Cost estimate for pre-feasibility design of preferred Scheme 1B

Item Description	Qty	Unit	Cost (R millions)
1 CIVIL WORKS CAPITAL COSTS			
DN1800 Delivery pipeline to LLPS	1.6	km	55
DN1800 Abstraction pipeline to WTW	10.5	km	358
DN1800 Bulk transfer pipeline to command res.	176.4	km	6,954
DN1400 Gravity pipeline (to Rustfontein)	24.5	km	572
DN2000 Gravity pipeline (to Brandkop)	31.4	km	1,275
Pump station buildings	3	No.	90
Concrete reservoirs (80 Mℓ)	3	No.	360
Subtotal:			**9,664**
2 MECHANICAL AND ELECTRICAL CAPITAL COSTS			
Abstraction works pump station (3797 ℓ/s at 102 m)	1	No.	145
High lift pump station at WTW (3616 ℓ/s at 325 m)	1	No.	494
Booster pump station (3616 ℓ/s at 124 m)	1	No.	188
Subtotal:			**827**
3 WATER TREATMENT WORKS CAPITAL COST			
WTW civil and M&E installation costs	312	Mℓ/d	1,512
Subtotal (1)			**12,003**
Preliminary and General	15	%	1,801
Subtotal (2)			**13,804**
Cost Price Adjustment	20	%	2,761
Subtotal (3)			**16,565**
Contingency	15	%	2,485
TOTAL CAPITAL COST	**Sub-total:**		**19,050**
4 ANNUAL OPERATION & MAINTENANCE COSTS			
Water treatment cost (chemicals and sludge disposal)			65
Energy cost (pump stations | WTW)			479
Mechanical replacement cost (pump stations)			55
Civil maintenance cost (pump stations | reservoirs | WTW)			81
M&E maintenance cost (pump stations | WTW)			70
Administration cost			166
Safety and Security			27
ANNUAL OPERATION & MAINTENANCE COST	**Sub-total:**		**944**
TOTAL NPV OPERATIONAL COST (4% DISCOUNT OVER 30 YEARS)			**16,586**
TOTAL NPV PROJECT COST			**35,636**

7.7 Water Treatment Works

The pre-feasibility study focused on understanding all aspects of the proposed schemes necessary for DWS to make an informed decision on which of the schemes to proceed with to the detailed feasibility phase.

The following is addressed as part of this section of the report:

- The design capacity and phasing of the WTW,
- Raw water quality and characterisation,
- Corrosivity of the raw water,
- Preliminary process selection based on treatment objectives,
- Concept WTW layout options, and,
- Development of a high-level cost estimate for the various options including O&M costs.

The Inception Report referred to the evaluation of various treatment options including the impact of the transferred raw water on existing WTWs. The need for this has, however, been negated as the water resource analysis and the infrastructure optimisation, contained in Sections 3.2 and 7.6 indicate that the raw water transfer options are not preferred. The focus of this section will therefore only be on the proposed Scheme 1B treatment site near the Gariep Dam.

7.7.1 WTW Site Location

The proposed WTW will be located along the N1 highway north of the Gariep Dam wall as shown in Figure 7-37. A photo of the site has been included in Figure 7-38.

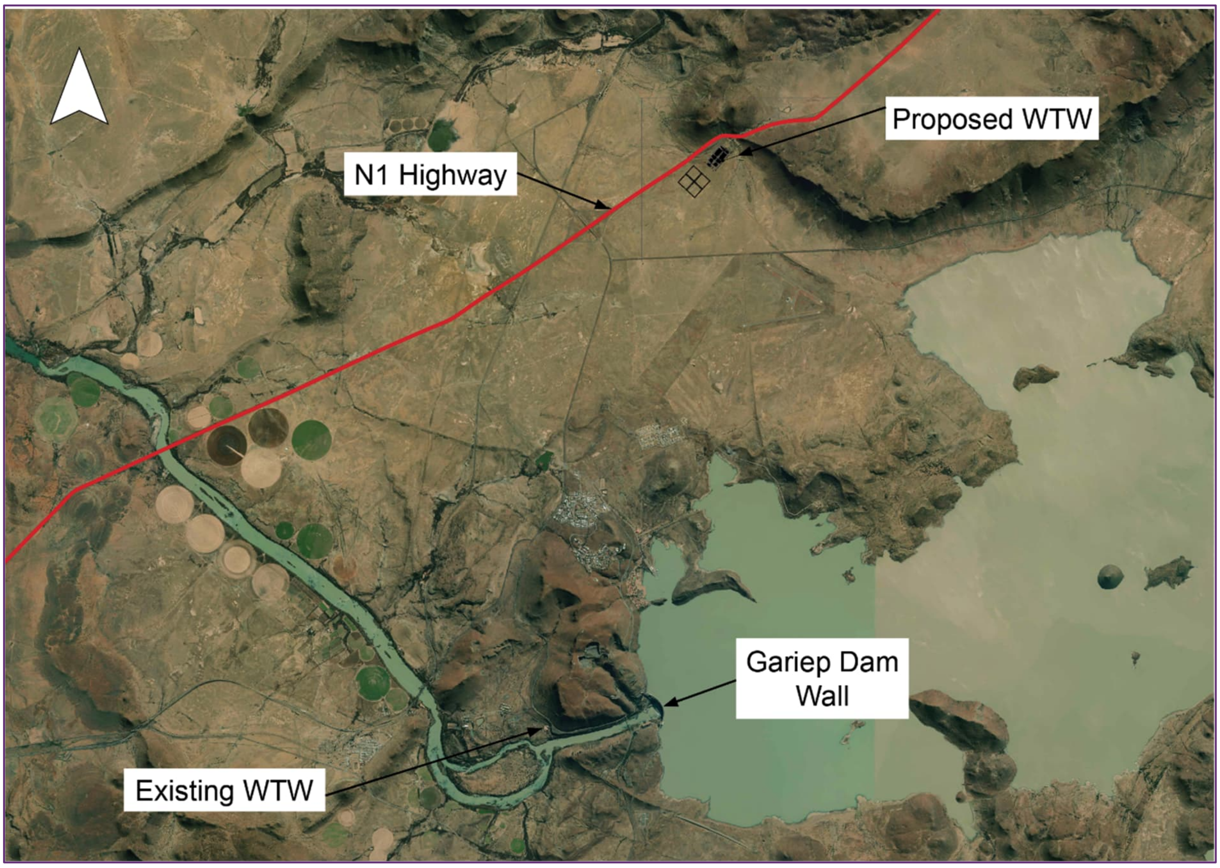


Figure 7-37: Location of the proposed WTW for Scheme 1B

Figure 7-38: View of the proposed WTW from the N1 highway

7.7.2 Design Capacity and proposed Phasing

Based on the yield analysis discussed in Section 3, the WTW will need to be designed for a peak daily demand (AADD) of 312 Mℓ/d (incl. losses) over a 22-hour period. The capacity of the proposed plant provides for a peak supply of 114 million m^3 of treated water per annum with allowances for conveyance losses as well as summer peak factors.

Provision will be made to expand the WTW in 100 Mℓ/d modules for preliminary planning purposes. The final phasing approach will be determined during the feasibility and detailed design stages of the project. For this report, it was assumed that the first phase will be developed immediately and the second phase as the need arises. It is assumed, for planning purposes, that 10 years will elapse between the construction of phases.

The first phase would include two 100 Mℓ/d process modules, providing 200 Mℓ/d during this phase. The second phase would expand the plant with another 100 Mℓ/d phase, and with optimised process operation, each of these phases would accommodate an additional 4 Mℓ/d, bringing the ultimate design capacity of the treatment works to 312 Mℓ/d. During design, provision would be made to further expand the plant with another 100 Mℓ/d process module to accommodate future expansion of the distribution network toward Bloemfontein, additional off-takes to towns located along the rising main, or expansion of bulk distribution toward the south of the WTW. This final expansion will, however, not be included in the costing, but will be included in the inlet structure capacity calculations, and an additional pump bay will be allowed for in the high lift pump station. The cost is also provided for the scenario where the 312 Mℓ/d WTW is constructed in a single phase at the commencement of the project.

7.7.3 Water Quality and Characterisation

7.7.3.1 Raw water quality

Raw water quality and characterisation is discussed in Section 4 of this report. The available information suggested that the water quality is good, and treatment needs to focus mainly on turbidity removal, stability control and disinfection.

Some aerial images indicate that the Gariep Dam does experience occasional algal blooms. These events are not quantified in the available data because the appropriate determinants were not monitored. Data preceding the year 2000 however indicate that high levels of chlorophyll-a may be present in the dam at times.

There was also no indication of the status of Contaminants of Emerging Concern (CECs) in the dam from the available data.

Additional water quality monitoring will therefore be conducted during the detailed feasibility phase of this study.

7.7.3.2 Potable Water Quality Standards, Regulations, and Guidelines

Treatment objectives are based on The South African National Standard (SANS 241:2015): Drinking Water. This reference is supported by international references focusing on water quality objectives that protect consumers from potentially harmful contaminants. One of the agencies that have taken a lead role in this regard is the US Environmental Protection Agency (USEPA). Many of the additional guidelines and water quality objectives referred to in the report are based on USEPA guidelines.

It should be noted that the aim of both SANS 241:2015 and USEPA guidelines is to protect consumer health. The SANS 241 approach is to specify a recommended maximum concentration level for a particular water quality parameter. SANS 241 is limited in some respects as it, for instance, does not restrict the concentration and type of organic pollutants apart from specifying the maximum level of Total Organic Carbon (TOC). The USEPA takes a different approach and does not distinguish between different classes of water but specifies a maximum contaminant level (MCL) and a target contaminant level (TCL). In some cases, no MCL is specified, but replaced with a Best Available Technology (BAT) rule that prescribes the treatment process required in cases where it is not technically or economically feasible to monitor the contaminant. USEPA has published several regulations/guidelines under the US Safe Drinking Water Act (SDWA) in this respect including:

- USEPA (2006): Long-term 2 enhanced surface water treatment rule (LT2ESWTR),
- USEPA (2006): Stage 2 disinfectants and disinfection by-products rule (DBPR),
- USEPA (2007): Simultaneous compliance guidance manual for the long term 2 and stage 2 DBP rules, and,
- USEPA (2012): Guidelines on water reuse.

The last bullet is relevant to a lesser extent as this project is not a reuse project in the conventional sense given the retention time of the Gariep Dam and given the relatively low portion of wastewater return flow included in the catchment. Some of the contaminants are however resistant to naturally occurring breakdown processes and a level of care in this regard remains advised.

Additional parameters noted as an emerging concern, that have not been tested for in the available raw water records, are the presence of Algal blooms, CECs as well as taste and odour causing compounds i.e., Geosmin and MIB (2-Methylisoborneol). These will be tested and evaluated during the detailed feasibility phase against the treatment process stream proposed in this report.

7.7.4 Existing Gariep Dam WTW

Development of a WTW is best supported if a pilot study is undertaken. As an alternative, bench scale tests can be performed in batch fashion in a laboratory. This site is however in the fortunate position that a reference plant, which treats the same water as intended for the Xhariep WTW, is available to see how the raw water will react to treatment processes.

At present, and based on available treated water quality data, the existing Gariep WTW appears to be adequate in terms of its treatment process. The data however excluded analysis of determinants that have been raised as concerns in Section 4 and Section 7.7.3.1 of this report.

The design capacity of the existing Gariep WTW is 2.8 Mℓ/d or 1.023 million m^3/a. The average raw water abstraction from the Gariep Dam is 1.755 Mℓ/d or 0.641 million m^3/a. A Site Inspection Report, prepared as part of this study, can be found in Appendix C.

The existing Gariep WTW consists of the following unit processes:

- Raw water abstraction/supply,
- pH Correction and Stabilisation,
- Coagulation and Rapid Mixing,
- Hydraulic Flocculation,
- Radial Flow Clarifier,
- Intermediate chlorination,
- Rapid Gravity Sand Filters,
- Clearwater contact tank, and
- Washwater and solids residual disposal.

The final treated water characteristics from the existing Gariep WTW are as provided in the Table 7-33. The data was extracted from compliance data for the existing Gariep WTW for the periods 2004 to 2017 and 2018 to 2022. The data was sourced from the DWS and VCWB respectively. Analysis of the data indicated that the existing treatment process performs well. There are occasional water quality failures, but these failures were likely operational in nature and not as a result of the treatment process design. The table does not include results for concerns associated with algal blooms and with contaminants or emerging concerns. This will require further evaluation after a monitoring programme has been undertaken.

Table 7-33: Final water characteristics from the existing Gariep WTW

Parameter	Units	No. of analyses	5th percentile Raw Water Operational Data	50th percentile Raw Water Operational Data	95th percentile Raw Water Operational Data	SANS 241: 2015 and DWS/WHO Standards
Turbidity*	NTU	50	0.18	0.40	***4.25***	≤ 1
Colour	mg/ℓ as Pt	3	0.10	1.00	1.90	≤ 15
TDS	mg/ℓ	221	-1.73	-0.358	131.95	≤ 1200
Conductivity	mS/m	223	13.76	18.70	23.27	≤ 170
pH	[-]	223	6.51	7.60	8.18	≥ 5 to ≤ 9.7
Total Alkalinity	mg $CaCO_3$/ℓ	223	43.72	62.90	84.15	~40-120
Fluoride	mg/ℓ	220	0.05	0.17	0.31	≤ 1.5
Ammonia	mg/ℓ	220	0.02	0.02	0.14	≤ 1.5
Potassium	mg/ℓ	220	0.98	1.26	2.47	≤ 50
Sodium	mg/ℓ	220	4.00	5.68	7.45	≤ 200

Parameter	Units	No. of analyses	5th percentile Raw Water Operational Data	50th percentile Raw Water Operational Data	95th percentile Raw Water Operational Data	SANS 241: 2015 and DWS/WHO Standards
Zinc	mg/ℓ	71	0.00	0.01	0.08	≤ 5
Calcium	mg/ℓ	220	***14.90***	20.00	29.80	≥ 16
Iron	mg/ℓ	117	0.01	0.10	0.10	≤ 0.3
Manganese	mg/ℓ	117	0.00	0.01	0.02	≤ 0.1
Magnesium	mg/ℓ	220	4.70	6.26	7.50	≤ 30
Chloride	mg/ℓ	219	1.50	4.75	8.43	≤ 300
Chlorine, free as Cl_2	mg/ℓ	180	1	1.20	2.60	≥ 0.5; ≤ 5
Nitrate as NO_3 - N	mg/ℓ	223	0.23	0.57	0.98	≤ 11
Nitrite as NO_2 - N	mg/ℓ	16	0.01	0.01	0.01	≤ 0.9
Sulphate as SO_4^{2-}	mg/ℓ	220	4.70	9.97	44.72	≤ 250
Ortho-Phosphate PO_4	mg/ℓ	217	0.00	0.02	0.10	
Calcium Hardness (calculated from above)	mg/ℓ as $CaCO_3$	220	14.90	20.00	29.80	
Magnesium Hardness (calculated from above)	mg/ℓ as $CaCO_3$	170	16.30	21.16	25.61	
Total Hardness (calculated from above)	mg/ℓ as $CaCO_3$	170	58.83	78.71	99.00	≤ 150
Langelier Index	-	169	***-1.88***	***-0.54***	-0.01	~ 0
Ryznar Index	-	169	***8.23***	***8.89***	***10.24***	6.5 – 7.0
Escherichia coli	MPN or CFU per 100 Mℓ	155	0.00	0.00	0.00	0
Heterotrophic plate count (HPC)	CFU	186	0.00	0.00	0.00	≤ 1000
Total coliforms	CFU	186	0.00	0.00	0.00	≤ 10
Calcium Carbonate Precipitation Potential (CCPP) (calculated from above)[2]	mg $CaCO_3$/ℓ		No data	No data	No data	2 to 5
TOC	mg/ℓ	49	1.58	2.83	3.78	<10
DOC	mg/ℓ	1	-	3.33	-	<10

Notes:

1 Figures shown in **red** do not meet the necessary standards

2 Parameters could not be determined, require chemical dosing information from existing Gariep Dam WTW

7.7.5 Preliminary Process Selection

The **first step** in the overall process selection was to determine whether the raw water would be treatable via slow sand filtration (Van Duuren et al, 1997). Due to the turbidity as well as the large filter area required, the use of slow sand filters would not be suitable in this instance and conventional treatment through coagulation, phase separation and disinfection would be required.

The **second step** was to classify the type of water to be treated to determine which coagulants may be used to efficiently agglomerate micro-particles to larger flocs that can be removed from the raw water. Clear waters with low concentration of particles provide limited opportunity for bridging and floc formation due to a lack of particle collisions. A publication from the Water Research Commission (WRC) titled "A Guide for Water Purification and Plant Design", refers to the various combinations of turbidity and alkalinity via the O'Melia raw water classification system. The classification system is described in Table 7-34.

Table 7-34: O'Melia raw water classification system

Limits for the O'Melia raw water classification system (Turbidity measured as nephelometric turbidity units (NTU) and Alkalinity as mg/ℓ $CaCO_3$)			
	Alk > 250	**50 < Alk < 250**	**Alk < 50**
NTU > 100	Type 1	***Gariep Raw Water***	Type 2
10 > NTU > 100		***Gariep Raw Water***	
NTU < 10	Type 3		Type 4

With the water quality available, the raw water falls between Type 1 and Type 3. The difference between Type 1 and Type 2 is mainly the type of flocculant being used and whether stabilisation is required, while the difference between Type 1 and Type 3 is the need for sedimentation. It is anticipated from available raw water quality data that stabilisation and settling would be required which then defines the Gariep Dam water as Type 1. For this report, it was assumed that Poly Aluminium Chloride (PACℓ) will be used as coagulant. This may be reconsidered when laboratory tests are carried out during the detailed feasibility design.

The **third step** was to select the appropriate phase separation process or processes, which would always include rapid sand- or multimedia filtration with some upstream process depending on the characteristics of the water to be treated. For water with high turbidity (NTU > 100) and low algal content (Chlorophyll-a < 10 mg/l), conventional settling is generally used upstream of filtration (Van Duuren, 1997).

As noted in Section 7.7.3.2, additional raw water sampling and testing will be conducted to confirm the algal content to evaluate the preliminary process stream selected as part of this report. Current indications were that algal loads and chlorophyll-a levels may, at times, exceed the threshold levels that mandate specific algal cell removal. This implied the addition of stand-alone Dissolved Air Flotation (DAF), or combined DAF and filtration (DAFF) may be required. This report assumed that DAFF will be applied at the plant. This approach will need to be reassessed in the detailed feasibility phase of the study.

The **fourth step** was to select the type of final water disinfection system and although a large proportion of prevalent micro-organisms are removed by physical processes (i.e., sedimentation and filtration plays an important role in removing viruses, schistosomes and protozoa cysts), full disinfection can only be achieved by adding a chemical agent such as chlorine, sodium hypochlorite, chlorine dioxide or ozone. Chlorine has historically been the disinfectant of choice due to its disinfection efficiency, residual longevity, and ease of handling. Chlorine can also be utilised to oxidise reduced forms of iron and manganese to remove these constituents from the raw water. For this report, it was assumed that chlorine gas will be used as final disinfectant, although the feasibility of using other agents may be evaluated if required.

The stability of the raw water was classified in terms of its corrosivity, aggressiveness and scale forming tendencies based on the calcium carbonate precipitation potential (CCPP), Langelier Saturation Index (LSI) and Ryzner Stability Index (RSI). With the water quality available, the raw water can be classified as moderately aggressive and will require some form of pH correction and stabilisation.

Aggression is the designation given to the phenomenon where water contained in cement or concrete-lined structures attacks the cement matrix and concrete aggregates, while corrosion is defined as electro-chemical reactions between the water and metal components of the system that may give rise to dissolution of the metal and precipitation of metal salts resulting in, for example, pitting, nodule formation, red water and finally destruction of the conduit.

The **fifth and last step** in the overall process design was to assess whether any ‘special problems’ are likely to be encountered at the works. As noted earlier in the report, additional raw water sampling and testing is required for the presence of Algal Blooms, CECs as well as taste and odour causing compounds to evaluate the preliminary process stream selected as part of this report. Based on the available raw water the “special problems” are summarised in Table 7-35 below. Items that will require further evaluation have been highlighted.

Table 7-35: List of 'special problems'

Description	Treatment	Applicable
Water Stability	Stabilisation	Included
Iron / Manganese	Oxidation (Chlorine or Potassium Permanganate)	Not required
Taste & Odour	**Activated Carbon**	**Requires further investigation**
Algae	**Dissolved Air Flotation / Clarification**	**Requires further investigation**
CECs	**Ozone / Activated Carbon**	**Requires further investigation**
Hardness	Ion exchange / high lime softening	Not required
Nitrate	Lime addition/clarification/filtration and chlorination	Not required - implicit in design
Fluoride	Ion exchange	Not required
Organic carbon	Oxidation / Adsorption / Enhanced Coagulation	Not required
Colour	Lime addition/clarification/filtration and chlorination	Included

The highlighted items for advanced processes require further investigation once additional water sampling and testing is carried out as listed in Chapter 4.

7.7.6 Treatment Objectives and Unit Treatment Process Requirements

Prior to the design of any treatment works, it is necessary to understand the raw water quality, the final water quality standards and the specific treatment objectives that should be aimed at removing the constituents of concern. In this case SANS 241:2015 was used primarily as a guide to determine water quality targets. In cases where SANS 241:2015 does not specifically provide a contaminant target value the WHO or USEPA guidelines were considered. Before discussing the different treatment objectives, it is important to briefly discuss water quality standards and treatment effectiveness.

7.7.7 Treatment Objectives and Effectiveness

Most of the measurements of treatment effectiveness developed by USEPA are expressed in terms of log removal and performance efficiency and the tables included in subsequent paragraphs use both approaches.

Table 7-36 below indicates the water quality objective, possible unit treatment process, process function and treatment targets.

Table 7-36: Summary treatment objectives

Water Quality Objective	Possible Unit Treatment Process	Process function	Target Parameter
pH Correction	Lime Dosing	Adjust pH and stabilise water	CCPP 2 to 5 mg/ℓ
Removal of suspended material (Reduce turbidity)	Horizontal Flow Sedimentation Tanks	1a) Removal of settling suspended matter 1b) Removal of pathogens including Giardia and Cryptosporidium 1c) Precipitation of oxidised metals	≤ 1.0 NTU
	Rapid Gravity Sand Filters	2a) Removal of fine suspended matter 2b) Removal of pathogens including Giardia and Cryptosporidium	
Inactivation of pathogens	Chlorination	1) Inactivation of Giardia, bacteria, and viruses	Giardia – 7 to 8 log removal Bacteria – 10 to 11 log removal Viruses – 11 to 12 log removal
Ensure distribution system residual disinfectant	Chlorination	1) Establish a chlorine residual	> 1.0 mg/ℓ
DBP precursor reduction (DOC removal)	1) Clarification 2) Rapid Gravity Sand Filters	1 & 2) Reduce DOC by coagulating dissolved organics - remove with phase separating steps	DOC < 10 THM < 0,005mg/l

It is anticipated that a conventional treatment train will be sufficient to address the treatment requirements while being cost effective in terms of both capital and operating costs.

Conventional treatment normally involves coagulation, flocculation, sedimentation, gravity sand filtration and disinfection. The main treatment processes are briefly discussed below:

- With the turbidity averaging above 10 NTU and exceeding 100 NTU in some instances, it is foreseen that a two-stage phase separation process will be required i.e., sedimentation and rapid gravity sand filtration,
- Seasonal algal blooms will be handled using DAFF,
- Some oxidation of iron may be required while disinfection is mandatory. Chlorine dosing is considered for this purpose, and
- Washwater recovery, and solids residual disposal will be required.

7.7.8 Process Sizing

7.7.8.1 Chemical Dosing Regime

It is anticipated that the following chemical dosing strategies would achieve the desired treatment outcomes:

- pH correction and Stabilisation: Hydrated Lime,
- Coagulant aid: Poly Aluminium Chloride (*PACℓ*), and,
- Disinfectant: chlorine gas.

Alternatives could be considered as part of a detailed feasibility phase and detailed design phase.

In addition to the PACℓ, the use of a flocculation adjuvant may be considered in periods of greater turbidity. This was not included as part of this pre-feasibility phase and would require additional water quality sampling and testing.

7.7.8.2 Raw Water Receiving Bay

The raw water will be conveyed to the New Xhariep WTW through an 1800mm diameter steel pipeline where it will discharge into the raw water receiving bay. The incoming stream will be divided into three equal streams over sharp crested weirs. For Phase 1, two streams will be operational and flowing at 100 Mℓ/d each. For Phase 2, three streams will be operational at a flow of 104 Mℓ/d each. Isolating sluice gates will be provided at this point to shut off one module/unit when required. The future stream, to increase the overall plant capacity to 400 Mℓ/d, will be temporarily closed.

7.7.8.3 pH Correction and Stabilisation

The inlet works will provide sufficient mixing energy for hydrated lime dosing, and this will be followed by a baffled channel providing adequate contact time for the lime to dissolve and stabilise the pH prior to coagulation.

The following dosing rates were assumed in the pre-feasibility study design:

- 20 mg/ℓ maximum, and,
- 10 mg/ℓ average.

The assumed dosing values will be subject to further confirmation during the detailed feasibility phase. The actual dosages are expected to vary with changing raw water characteristics that will occur through the different seasons. Regular (once a week) jar tests will be required to optimise chemical dosages. Additional jar tests will be required when there is a noticeable (visibly or chemically) change in water quality.

At least 120 000 kg (± 200 m^3) of hydrated lime will need to be stored to provide 30 days' storage for first 200 Mℓ/d phase of the WTW at the average coagulant dosage concentration.

The preliminary lime dosing design calculations are indicated in Table 7-37 and the preliminary lime rapid mixing calculations are indicated in Table 7-38.

Table 7-37: Preliminary Lime Dosing Calculations

Parameter	Value	Unit	Remarks/Reference
Phase 1 Flow	200	Mℓ/d	
Phase 2 Flow	112	Mℓ/d	
Maximum dosing rate per Phase	20.0	mg/ℓ	Estimated, specific testing is required
	6,240.00	kg/d	Phase 1: 4,000kg/d Phase 2: 2,240kg/d
	260	kg/hr	Over 22 hour operation
Number of dosing units	2	No	Phase 1: 1 duty + 1 standby unit Phase 2: 1 duty + 1 standby unit
Bulk Density of hydrated lime	650	kg/m^3	
30-day storage capacity	187,200	kg	Phase 1: 120,000 kg (Four 50 m^3 bulk storage silos) Phase 2: 67,200 kg (Two 50 m^3 bulk storage silos)

Table 7-38: Preliminary Rapid Mixing Calculations

Description	Symbol	Design Capacity	Hydraulic Capacity	Units	Formula/ Comments
Flow Rate (total for plant)	Q_{total}	312,000.00	357,382.00	m^3/day	
Number of streams		3.00	3.00		
Flow Rate (per stream)	Q_{stream}	104,000.00	119,127.00	m^3/day	
Water temperature	˚C	20.00	20.00		
Density of the fluid (water)	ρ_{water}	997.46	997.46		
Dynamic viscocity	μ_{water}	0.00112400	0.00112400		
Kinematic viscosity	v_{water}	0.00000100	0.00000100		
Total headloss	h_{total}	500.00	500.00	mm	Hydraulic Mixing (Weir or orifice plate)
Effective Mixing Volume	V	5.88	5.88	m^3	Q*T
Retention time	T	4.89	4.18	sec	
G =	G	1,000.00	1,082	s^{-1}	$((g*h/(v*t))^{0.5}$
G * (T)^0.5 =	GT	17,120	17,120		$G * (T)^{0.5}$
Hydraulic Retention Time per stream	Volume	325.00	372.00	m^3	Lime requires at least 3 minutes retention time

The bulk storage silos will be located next to the raw water receiving bay from where dry lime will be transferred to the day hoppers. The day hoppers, metering feeders and mixing bowls will be installed on the raw water receiving bay with adequate storage capacity to dose hydrated lime upstream of the flocculation tanks.

Three lime dosing points will be provided at the raw water receiving bay to dose lime into each of the three raw water streams at the first sharp crested weir upstream of the flocculation tanks.

Lime will be dosed in slurry form of maximum 5% concentration and at a maximum dosage of 20 mg/ℓ for the new WTW at 312 Mℓ/d.

To ‘stabilise’ the water, hydrated lime will be dosed to produce a slightly over-saturated water with a positive CCPP (typical 2 – 5 mg/ℓ as $CaCO_3$) and increase the pH, which enhances the oxidation of manganese. Under all expected operating conditions, the maximum amount of lime required is estimated at 10 – 20 mg/ℓ.

7.7.8.4 Coagulation, Rapid Mixing and Flow Division

Chemical dosing is purposefully designed to provide flexibility regarding dosing capacity. This is mainly since raw water quality can vary and, given that the chemical dose and raw water quality are directly interlinked, the equipment must provide flexibility in this regard. This flexibility can easily be utilised to make provision for the relatively small increase in plant capacity.

Various chemicals are utilised for the treatment of water for numerous reasons and form an intricate part of the overall treatment process with associated operational costs. Usually, chemicals are added to water for the removal of suspended (e.g., colloids) and dissolved (e.g., chemical softening and removal of colour) particles, disinfection (e.g., chlorine), removal of taste and odour causing constituents (e.g., activated carbon) and stabilisation (lime and/or carbon dioxide).

A new chemical dosing building for the storage of chemicals and to house the new chemical dosing equipment will be required near the rapid mixing and flow division chamber.

Once the water has been stabilised, the ultimate flow of up to 312 Mℓ/d will be split into three modules of 104 Mℓ/d each, in the dividing chamber. Isolating sluice gates will be provided at this point to shut off a module/unit when required. Once flow enters the dividing chamber, the flow splits into equal streams of 51 Mℓ/d (50 Mℓ/d streams for Phase 1 and 51 Mℓ/d streams for Phase 2). The future streams will be temporarily closed.

The following dosing rates have been assumed in the pre-feasibility design:

- 30 mg/ℓ maximum, and,
- 20 mg/ℓ average.

Accurate dosing ranges will require additional test work during implementation. The actual dosages are expected to vary with changing raw water characteristics that will occur through the different seasons. Regular (once a week) jar tests will be required to optimise chemical dosages. Additional jar tests will be required when there is a noticeable (visibly or chemically) change in water quality.

The polymer coagulants, in general, consume considerably less alkalinity than hydrolysing metal salts. They are effective over a broader pH range compared to alum and experience shows that they work satisfactorily over any pH range, therefore no pH adjustment will be required for proper coagulation. However, they do not work well in low turbidity waters.

PACℓ was selected for the pre-feasibility level study as a reference coagulant.

- Bulk Coagulant storage will consist of:
  - Five 40,000 ℓ bulk storage tanks (*three for Phase 1*) will be adequate to provide at least 30 days' storage for 312 Mℓ/d at the average coagulant dosage concentration.
  - The required pipework to the two dosing pumps will have the necessary flexibility to use any pump from the day tank.
- Coagulant Dosing Equipment will include:
  - Ten single phase variable speed drive (VSD) dosing pumps (*five required for Phase 1*) shall be provided (eight duty and two standby) to feed PACℓ from the bulk storage tanks to the dosing points.
  - The dosing pumps are each required to deliver between 100 and 300 ℓ/hour at a pumping head of ±10 m by means of flow paced electronic variable speed controls. Dosing pumps are also stroke adjustable between 10 and 100% of maximum stroke.
  - All pipework is uPVC and delivery lines will be fitted with suitably rated loading valves, pulsation dampeners and flushing lines. A one litre flask will be allowed for on the suction manifold for calibration of the dosing pumps.

The preliminary design calculations for the PACℓ dosing system are indicated in Table 7-39 below.

Table 7-39: PACℓ Dosing System calculations

Parameter	Value	Unit	Remarks/Reference
Flow	312	Mℓ/d	Design capacity of WTW (incl. conveyance loss and summer peak factors)
Maximum dosing rate	30	mg/ℓ	
Density of polymer	1,300	kg/m^3	Concentrated poly-aluminium chloride
Number of dosing units	10	No	Phase 1 4 duty + common stand-by
Maximum delivery of dosing pumps	300.0	ℓ/hr	At 100% Stroke
30-day storage capacity @ avg. dosing rate of 20 mg/ℓ.	9,360	kg	Phase 1: 6,000kg Phase 2: 3,360kg

	200	m^3	5 x 40 m^3 bulk storage tanks to be provide– - Three required for phase 1, two for phase 2.

► Rapid Mixing

Allowance will be made to dose a coagulant into the four individual streams of 50 Mℓ/d in Phase 1 as well as the two 50 Mℓ/d streams of Phase 2. With plant optimisation, all streams will accommodate 51 Mℓ/d at Phase 2. Hydraulic flash-mixing will be affected by means of a weir, orifice plate or in-line static mixer with a head loss of at least 1.0 m immediately after the dosing point.

Sufficient flash-mixing inducing a G-value > 1000 sec^{-1} is of prime importance to optimize the efficiency of coagulants. Table 7-40 shows the preliminary design calculations for the flash-mixing.

Table 7-40: Flash-mixing preliminary design

Description	Design Capacity	Hydraulic Capacity	Units	Formula
Flow Rate (total for plant)	312,000	357,382	m^3/day	Design Capacity of WTW (incl. conveyance loss and summer peak factors)
Number of streams	6			
Flow Rate (per stream)	52,000	59,564	m^3/day	
Water temperature	20		°C	
Density of the fluid (water)	997.46		kg/m^3	
Dynamic viscosity (µ)	0.001124		kg/m.s	
Kinematic viscosity (v)	0.000001004		m^2/s	
Total head loss	1000		mm	Weir, Static Mixer, or Orifice Plate
Effective Mixing Volume	5.65	5.65	m^3	Q*T
Retention time	9.77	8.53	sec	V/Q
G [1] =	1,000.00	1,070.00	s^{-1}	((g*h/(v*t)^0.5
GT =	24,213.00	24,225.00		G * (T)^0.5

7.7.8.5 Flocculation and Clarification

It should be noted that various types of flocculation and clarification alternatives are available and will need to be evaluated as part of the detailed feasibility study. For the pre-feasibility study, the calculations were based on the sludge blanket (Pulsator Clarifier) with internal flocculation.

The design features 12 sludge blanket clarifiers (four per 100 Mℓ/d phase) each with a central flocculation column. From the flocculation column the flow diverted into four conduits from which the clarifier will be fed through several pipes with orifices. The excess sludge in the sludge blanket will flow laterally into concentrated pockets and will be removed by hydrostatic pressure through discharge pipes to a central sludge gallery.

Twelve separate sedimentation tanks will be selected to maintain the maximum loading rate of 2.8 m/h when one tank is out of service. Under normal operating conditions this will result in a hydraulic loading rate of 2.52 m/h. By using a length to width ratio of 2:1, each tank surface area was calculated as 420 m^2. The tank depth was selected as 4.5 m. Launders with submerged holes with a hydraulic load rate of 8 m^3/h/m with one tank out of operation result in a total double weir length of 36.95 m or four double weirs of 9.25 m each.

The design calculations for the sludge blanket clarifier are indicated in Table 7-41 below.

Table 7-41: Sludge Blanket Clarifier (Pulsator) Preliminary Design

Parameter		Value	Unit	Remarks/Reference
INLET CONDITIONS	Total raw water inflow	312,000	m^3/day	Design Capacity of WTW (incl. conveyance loss and summer peak factors)
	Water temperature	25	˚ C	
	Density of the fluid (water)	996.48	kg/m^3	
	Fluid viscosity	0.00114	cP/100	
	Density of the dried solids	1,005	kg/m^3	
	Solids concentration	300	mg/ℓ	Assumed 2 x Turbidity
	Typical settling velocity for polymer flocs	2.52	m/h	
	Δρ	0.00254	kg/m^3	
	Number of process units	12		
DIMENSIONAL PARAMETERS	Flow per unit process	0.30093	m^3/s	
	Surface loading	2.5	m/h	
	Tank Width	14.5	m	
	Tank Length	29.0	m	
	Length: width ratio	2		
	Surface area	420.5	m^2	
	Surface loading (check)	2.5	m/h	
	Tank depth	4.5	m	
	Width:depth ratio	3.22		
	Wetted perimeter	23.5	m	
	Cross sectional area	65.3	m^2	
	Average cross flow velocity	0.00461	m/s	
WEIR SIZING	Total weir length required @ 6m^3/m/h	45.15	m	
	Total weir length required @ 8m^3/m/h (one tank o/s)	36.95	m	
	Flow per meter	7.3	m^3/m/h	
	Number of weirs across width	4		
	Pipe size of weir @ 0.5m/s	0.35		
	Theoretical retention time	1.75	hr	
KPI’ s	Reynolds number	12,754		
	Froude number	7.81E-07		
	Densimetric Froude number	0.43436		
	Fd^2	0.18867		

The flow characteristics of the sedimentation tank are estimated using the Reynolds number, Froude numbers and Densimetric Froude number. The use of the Reynolds number is limited to establish if turbulent flow can be expected or not while the Froude number relates the average tank velocity to the gravitational acceleration. The Densimetric Froude number gives an indication of the tank’s stability. If the ratio is less than unity, the gravity effects will dominate the momentum forces. Conversely the momentum forces will dominate gravity forces. In this case, the Densimetric Froude number is close to unity.

7.7.8.6 Dissolved Air Flotation

The available raw water quality data did not provide clarity on the need for DAF. The pre-feasibility study assumed that seasonal algal blooms do occur but that this can adequately be handled using DAFF. This allows for the DAF infrastructure to be accommodated within the filter bays. This does place some restrictions on the hydraulic regime of the filter but not to the extent that it will materially impact on the required outcome of the pre-feasibility study.

The DAF loading rates were therefore assumed to be equal to the filtration loading rates.

7.7.8.7 Rapid Gravity Sand Filtration

Many different filtration technologies exist and some of the most suitable options were evaluated such as continuous backwash filters, autonomous filters, rapid gravity sand filters (RGSF), pressure filters and cloth filters. It was decided to keep the same filtration technology as the existing Gariep WTW for ease of operation and maintenance.

A total of 30 rapid gravity sand filters will be provided i.e., 20 for Phase 1 and 10 for Phase 2 of the works with a combined filter area of 2,388 m² for a full stream of 312 Mℓ/d. This arrangement will produce a filtering rate of 6.24 m/hr with 30 filters in operation, and 6.68 m/hr when two filters are being backwashed.

The filter bays will also act as DAF units.

The incoming flow will be equally divided on the inlet side to the filters by inlet pipes while the outlet valves will allow filter water to waste on a time cycle and will allow a slow start operation.

Each filter will be backwashed independently, first with air at a rate of 52 m/hour, then with water also at a rate of 37 m/hour.

The minimum water level will be maintained by a fixed outlet weir. A 200 mm deep false floor will be provided for a nozzle system.

While operating in DAFF mode the filters will have to operate in a constant level and constant rate mode. This will require careful flow and level management using outlet control valves.

Provision was made in the design for 1000 mm deep filter media. The filter media will be made up of sand. The sand should have a uniformity coefficient (UC) < 1.4 and must have an effective grain size of 0.9 mm. Limits will also be placed on the over and under size fractions.

Each filter will be fully automated and equipped with pressure sensors (one mounted above the media to measure loss of head and one mounted above the filter outlet to measure flow rate) and on-line turbidity meters.

The filter washing will be automatic following initiation when the pressure level in a filter reaches a pre-set level or when a filter has completed a pre-set run length or will be manually initiated. Table 7-42 and Table 7-43 shows the filtration bed and filter design calculations.

Table 7-42: Filtration Bed Fluidization and Expansion

Parameter	Value	Unit	Remarks/Reference
Bed Expansion	10	%	
Bed Depth	1.0	m	
Porosity	0.42	-	
Density	2,650	kg/m³	
D_{10}	0.90	mm	
T	15	˚C	
Rho (water)	999.13	kg/m³	

Parameter	Value	Unit	Remarks/Reference
Viscosity (water)	1.447	10^{-6} m^2/s	
Le	1.00	m	
Ee	0.42	m	
B	669.72		
Kv	112.00		
Ki	2.25		
Re	8.06		
Backwash Velocity	36.96	m/h	
	10.27	mm/s	
Effective velocity at b/wash weir	51.80	m/h	
	14.39	mm/s	
L/d ratio	1111	%	

Table 7-43: Preliminary Filter Design

Parameter	Value	Unit	Remarks/Reference
Flow rate	357,382	m^3/day	Operational 22 of 24 hours and 5% treatment loss
	4.14	m^3/s	
	14,890.90	m^3/h	
Number of filter units	30.00		
Capacity per filter unit	11.91	Mℓ/d	
	496.36	m^3/h	
Filtration rate	6.24	$m^3/m^2/h$	
Required filter area	79.6	m^2	Required filter area
Filter bed Length	16.9	m	Filter bed Length
Filter bed Width	4.7	m	Filter bed Width
Provided Filter Area	79.6	m^2	Provided Filter Area
Total filter area	2,388	m^2	
Filtration rate	6.24	$m^3/m^2/h$	
Filtration capacity (net)	357.382	Mℓ/d	
Filtration rate with 2 filters O/S	6.68	$m^3/m^2/h$	
Backwash rate	37	$m^3/m^2/h$	
	2,943	m^3/h	
Max velocity in washwater header	2.5	m/s	
Backwash pipe size (diameter)	650	mm	

7.7.8.8 Washwater Disposal

For purposes of this pre-feasibility report, it was assumed that the backwash water from the filters shall be wasted to the sludge lagoons that will be constructed next to the WTW. This assumption was made to allow for worst case land use planning and evaluation of environmental requirements. The solids in the wasted backwash water would settle out while the excess water will be allowed to drain into the stormwater system through a proposed outlet pipe, recycled back to the inlet of the works or allowed to simply evaporate. This would require the removal of sediment from the lagoons from time to time.

The detailed feasibility study will have to consider whether alternative disposal options inclusive of thickening and dewatering are viable solutions for this site.

7.7.8.9 Disinfection

Chlorine gas was proposed for disinfection with an estimated maximum dosage of 10 mg/ℓ chlorine required to maintain a 1.5 mg/ℓ residual at the outlet of the treated water tank at full production capacity. A full stand-by unit will be provided for post-chlorination and will be automated proportional to the mass flow rate.

At a maximum dosage of 7.5 mg/ℓ and ultimate flow rate of 14,182 m^3/hr, the mass dosing rate of chlorine required will be 97.5 kg/hr. The high dosing rate was proposed for the long pipeline to the final discharge point and required residual chlorine concentration. This will require a total of 70 one tonne chlorine cylinders per month, and around 12 cylinders per manifold at the indicated dosing rate.

The final determination of the dosing rate will have to consider the chlorine demand of the treated water as well as the effect of the transfer and storage infrastructure on the chlorine residual. It is recommended that laboratory-based studies be undertaken during the detailed feasibility study to shed light on the chlorine demand and chlorine decay from which the optimal chlorine dosing strategy can be determined.

From Table 7-44 and Table 7-45, a Contact Time (CT) value of 104 mg.min/ℓ will be required for a 3-log inactivation of Giardia Cysts. Assuming a residual chlorine value of 1.5 mg/ℓ, the estimated contact time required is 69 minutes.

Table 7-44: CT Values for Inactivation of Giardia Cysts

Disinfectant	Inactivation (mg · min/L)					
	0.5-log	1-log	1.5-log	2-log	2.5-log	3-log
Chlorine [1]	17	35	52	69	87	104
Chloramine [2]	310	615	930	1 230	1 540	1 850
Chlorine Dioxide [3]	4	7.7	12	15	19	23
Ozone [3]	0.23	0.48	0.72	0.95	1.2	1.43

CT values were obtained from AWWA, 1991

1. Values are based on a free chlorine residual less than or equal to 0.4mg/L, temperature of 10°C, and a pH of 7.
2. Values are based on a temperature of 10°C and a pH in the rant of 6 to 9.
3. Values are based on a temperature of 10°C and a pH of 6 to 9.

(Alternative Disinfectants and Oxidants Guidance Manual, EPA, April 1999)

Table 7-45: CT Values for Inactivation of Bacteria and Viruses

Disinfectant	Inactivation (mg · min/L)			
	Bacteria		Viruses	
	2-log	4-log	2-log	4-log
Chlorine	0.1 – 0.2	10 - 12	2.5 – 3.5	6 - 7
Chlorine Dioxide	8 - 10	50 - 70	2 - 4	12 - 20

Disinfectant	Inactivation (mg · min/L)			
	Bacteria		Viruses	
	2-log	4-log	2-log	4-log
Ozone	3 - 4	N/A	0.3 – 0.5	0.6 – 1.0

It was assumed that the minimum storage time will be catered for onsite as this will allow the disinfection process to be finalised on site and confirmed that the water will be safe immediately after it is released from the treatment site.

Consideration will have to be given to sustaining disinfection residuals in the long transfer line to the GBWSS. The current disinfection strategy used by MMM and VCWB must be taken into consideration during this effort as not all strategies are complimentary and cannot be mixed.

7.7.8.10 Clearwater Storage

It was proposed to construct two clearwater tanks with a full supply capacity of 250,000 m³ each to provide the following:

- Balancing tank for clearwater pumps.
- Baffled chlorine contact tank with 35 minutes contact time (at 50% water depth).
- Internal baffles will ensure that a baffling ratio of 0.7 is achieved and that no stagnant zones or short-circuiting will occur.
- Fixed level pump sump in first section of the tank to provide a constant head to the filter washwater pumps. This compartment will have a fixed overflow with a bottom sluice which can be kept open during commissioning stages to utilise the entire tank volume. During normal operating mode the sluice will be closed.

7.7.8.11 Solids Handling and Disposal

Refer to Section 7.7.8.8 on washwater disposal.

Sludge generated from water treatment processes includes suspended solids removal from the raw water and chemical precipitates produced by the treatment processes. In conventional treatment systems, sludge is normally generated by solids in the filter washwater stream, aluminium, or iron coagulant sludges as well as iron and manganese precipitates.

Five methods of disposal are typically considered for process waste (AWWA, 2000):

- Discharge to natural waterway,
- Discharge to sanitary sewer system,
- Discharge to permanent lagoons,
- Burial in a landfill after dewatering/drying, and
- Re-use of all or a portion of the wastes.

Disposal of process sludges to permanent lagoons has generally been the option of choice in South Africa and where adequate land is available, this strategy is very cost-effective, although it must be kept in mind that the lagoons will eventually fill up and require cleaning.

The sizing of the sludge lagoon(s) is dependent on the raw water quality and the type and amount of chemicals used in the treatment process.

In terms of raw water suspended solids, it was conservatively assumed in the absence of adequate historical data that the maximum suspended solids concentration will occur during a 3-month period (typically late summer), while suspended solids should generally be significantly lower during the balance of the year. An average suspended solids concentration for the wet season was assumed at 50 mg/ℓ based on 1 NTU generating 1–2 mg/ℓ of TSS as per the USEPA, while the average suspended solids concentration during the dry season was also assumed to be 20 mg/ℓ. For an average daily flow of 312,000 m^3/day and based on the above-mentioned assumptions, approximately 4,875 tons of solids will be removed by the treatment plant over a 1-year period.

Although coagulant sludges normally concentrate to 10% (AWWA, 2000) in sludge lagoons, the concentration of suspended solids will generally be higher and an overall concentration of 20% and density of 1,320 kg/m^3 has been assumed. Allowing at least 6-months storage for the lagoon implies that the lagoon must have a total volume for sludge storage of at least 245,000 m^3.

At least four lagoons, each with a six-month storage capacity would be required, one for drying sludge and three more for receiving sludge. Sludge from the clarifiers and filter wash water will discharge into one of the four sludge lagoons, thus allowing cleaning of the one, and redundancy from two more lagoons. Table 7-46 shows the sludge production and storage calculations.

Supernatant from the lagoons will be collected in a central chamber and discharged into the stream downstream of the lagoons or recycled back to the receiving bay.

Table 7-46: Sludge Production and Storage Requirements

Parameter	Based on raw water quality (1% Solids)	Based on raw water quality (2% Solids)	Based on raw water quality (3% Solids)	Units
Average suspended solids (Summer)	50.00	50.00	50.00	mg/ℓ
Average suspended solids (Winter)	20.00	20.00	20.00	mg/ℓ
PACℓ added	20.00	20.00	20.00	mg/ℓ
Lime added	10.00	10.00	10.00	mg/ℓ
Average production rate	312	312	312	Mℓ/d
Average mass dried sludge produced	13,352.75	13,352.75	13,352.75	kg/d
Volume of sludge produced per day	1,272	635	424	m^3/d
Dried sludge produced per year	4,873,752	4,873,752	4,873,752	kg
Dried sludge produced over 2 years	9,747,504	9,747,504	9,747,504	kg
Dried solids density	1,320.00	1,320.00	1,320.00	kg/m^3
Volume of un-thickened sludge produced over 1 year	464,167	232,084	154,722	m^3
Final solids content				
Sludge thickened after evaporation (Solids Content)	20%	20%	20%	
Bulk density of 20% solids Sludge	1,320.00	1,320.00	1,320.00	kg/m^3
Mass of thickened sludge	66,764.00	66,764.00	66,764.00	kg/d
	2,030,730.00	2,030,730.00	2,030,730.00	kg/month
	24,368,760.00	24,368,760.00	24,368,760.00	kg/year
Volume of sludge in lagoon (after drying)	50.60	50.60	50.60	m^3/day
	1,540	1,540	1,540	m^3/month
	18,461	18,461	18,461	m^3/year
Filling period (months)	6	6	6	No
Volume of sludge in lagoon (before drying)	232,100	116,50	77,400	m^3
Dry solids content	2,436,876	2,436,876	2,436,876	kg
Area required	116,042	58,021	38,681	m^2
Length / Width	4	4	4	

Parameter	Based on raw water quality (1% Solids)	Based on raw water quality (2% Solids)	Based on raw water quality (3% Solids)	Units
Width of lagoon	170	120	98	
Length of Lagoon	681	482	393	
Volume of water to remove	222,852.84	222,852.84	222,852.84	m^3
Evaporation based on area	960.23	1,920.45	2,880.68	mm

7.7.9 Financial Considerations

7.7.9.1 Introduction

The following items are priced separately:

- Civil Works,
- Pump stations,
- Pipelines – Gravity and Rising Mains, fittings and valves required,
- Mechanical Equipment,
- Electrical Equipment,
- Electronic Control and Instrumentation,
- Bulk Electrical Supply/connections, and,
- Supervisory Control and Data Acquisition (SCADA) system.

The proposed Gariep WTW was assessed for the following specific financial criteria:

- Capital Expenditure, and,
- Operation and maintenance costs.

7.7.9.2 Preliminary Capital Expenditure Estimate

The high-level first order capital expenditure (CAPEX) estimate was calculated based on historical data collected from similar sized treatment plants and infrastructure as shown in Table 7-47.

The assumptions used for the capital costing were as follows:

- Unit costs (civil, mechanical, electrical, and electronic) for process units were adopted from projects of a similar nature in terms of process technology, process size and hydraulic capacity.
- Preliminary and General Costs of 25% were assumed.
- Foreign exchange adjustment was calculated for all options for specific mechanical and electrical components that may be imported. Exchange rates at the time of tender and at the time of order were considered and the resultant foreign exchange adjustment was determined.
- Contract Price adjustment of 20% were assumed for the construction period.
- Contingencies of 15% were assumed to account for unforeseen items.
- Professional Fees were calculated based on ECSA 2021 Fee Scales gazetted on 26 March 2021 for each scenario based on the CAPEX and engineering discipline.
- Site Supervision was not included as part of the CAPEX.

Table 7-47: Estimated Capital Expenditure for the New Gariep WTW

Description	Amount		
	312 Mℓ/d [1]	Phase 1 200 Mℓ/d [2]	Phase 2 112 Mℓ/d [3]
Preliminary and general	R 377 981 523	R 254 780 713	R 319 551 174
Provisional sums and dayworks	R 11 832 727	R 7 585 082	R 11 017 299

Description	Amount		
Site clearance and bulk earthworks	R 68 776 228	R 44 087 325	R 64 036 654
Gatehouse	R 4 815 845	R 4 815 845	-
Administration building	R 40 691 977	R 40 691 977	-
Chemical dosing building	R 43 058 136	R 43 058 136	-
Chlorination building	R 10 389 779	R 10 389 779	-
Inlet works	R 14 516 849	R 14 516 849	
Pulsators	R 243 905 013	R 156 349 367	R 227 096 796
Filter block	R 115 923 952	R 74 310 226	R 107 935 289
Machine room	R 42 543 784	R 27 271 656	R 39 611 966
Contact tank	R 134 025 892	R 85 914 033	R 124 789 771
Solids handling and disposal	R 103 478 318	R 66 332 255	R 96 347 320
Ducting	R 24 572 146	R 15 751 376	R 22 878 807
Inter-connecting pipework	R 118 823 942	R 76 169 194	R 110 635 432
Continuously welded steel pipelines	R 91 459 977	R 58 628 190	R 85 157 199
Service water reticulation and sewerage	R 29 942 787	R 19 194 094	R 27 879 341
Stormwater	R 34 486 604	R 22 106 797	R 32 110 030
Access roads	R 196 139 210	R 125 730 263	R 182 622 675
Internal road works and paving	R 88 313 217	R 56 611 036	R 82 227 291
Landscaping and irrigation	R 35 782 267	R 22 937 350	R 33 316 405
Fencing	R 25 644 476	R 25 644 476	-
Miscellaneous	R 4 278 511	R 2 742 635	R 3 983 666
General electrical work	R 28 524 460	R 18 284 910	R 26 558 755
Construction Cost - Sub-Total A	**R 1 889 907 616**	**R 1 273 903 563**	**R 1 597 755 868**
Allowance for Contingencies (15%)	R 283 486 142	R 191 085 535	R 239 663 380
Allowance for Contract Price Adjustment (20%)	R 377 981 523	R 254 780 713	R 319 551 174
Allowance for Foreign Exchange (5%)	R 94 495 381	R 63 695 178	R 79 887 793
Estimated Professional Fees	R 170 091 685	R 114 651 321	R 143 798 028
Capital Cost Excl. VAT R 2 815 962 348	**R 1 898 116 309**		**R 2 380 656 243**

Notes:
1 Cost Estimate Base Date – October 2023.
2 Cost Estimate Base Date – October 2023
3 Cost estimate Base Date – October 2033 (assumed implementation 10 years after Phase 1).

7.7.10 Operation and Maintenance Cost

A design for a WTW is highly dependent on the operation and maintenance requirements, which in turn are based on the operational complexity of the infrastructure installed and the process technology implemented. The requirements of the Client, technical skills of staff and ability of the Client to maintain the infrastructure are also key aspects to be considered.

In addition to comparing the capital costs, it is also worthwhile to compare the costs over the project lifecycle for the different options including the capital costs as well as the operation and maintenance costs (particularly electricity) of the WTW. The following assumptions were made for estimation of the operation and maintenance costs:

- Chemical costs were obtained from suppliers and dosing rates were calculated based on accepted principles.
- Maintenance costs were based on the type of structure, mechanical or electrical component. This was done as a % of the CAPEX and/or capital replacement cost (CRC).
- Process controllers will be required for the New Gariep WTW.

7.7.10.1 Estimated Maintenance Cost

The annual maintenance costs are estimated as a percentage of the CRC of the specific components as shown in Table 7-48.

Table 7-48: Estimated Annual Maintenance Cost as a function of CRC for the New Gariep WTW

Description	Civil	Mechanical	Electrical	C&I	
Maintenance	0.75% of CRC	2.25% of CRC	2.25% of CRC	4% of CRC	
Value (% of total)	45%	35%	15%	5%	
					Total
Complete Phase [1]	R9 503 873	R22 175 703	R9 503 873	R5 631 925	**R46 815 374**
Phase 1 [1]	R6 406 143	R14 947 666	R6 406 143	R3 796 233	**R31 556 184**
Phase 2 [2]	R8 034 715	R18 747 668	R8 034 715	R4 761 312	**R39 578 410**

Notes:

1 Estimated maintenance cost required for first year of plant operation are based on 2023 Costs.

2 Estimated maintenance cost required for first year of plant operation are based on 2033 Costs

7.7.10.2 Estimated Operational Cost

7.7.10.2.1 Human Resources Cost

The cost of staff employed at the proposed WTW was calculated based on the level of Plant Manager, Supervisor, Process Controller, and maintenance required for each option. The salary estimates were escalated accordingly and based on industry accepted monthly/annual salaries for suitably qualified personnel. Note that these are first order estimates only and have been derived from previous projects of a similar nature and size.

The costs documented in Table 7-49 represents a summary of the approximate annual salaries for a WTW employing the same conventional treatment processes and were scaled to suit the capacity of the New Xhariep WTW.

Table 7-49: Approximate annual staffing costs for the New Gariep WTW

Title	R/Month	Number	Annual Cost		
			312 Mℓ/d [1]	Phase 1 200 Mℓ/d [1]	312 Mℓ/d [2]
Operators (8-hour shifts)	R 30,000	8	R2,880,000	R2,880,000	R7,469,978
Shift supervisors	R 45,000	4	R2,160,000	R2,160,000	R5,602,484
Superintendent	R 75,000	1	R900,000	R900,000	R2,334,368
General labourers	R 15,000	10	R1,800,000	R1,800,000	R4,668,736
Mechanical foreman	R 35,000	4	R1,680,000	R1,680,000	R4,357,487
Electrical foreman	R 35,000	4	R1,680,000	R1,680,000	R4,357,487
Mechanical technician	R 20,000	8	R1,920,000	R1,920,000	R4,979,986

Title	R/Month	Number	Annual Cost 312 Mℓ/d ¹	Annual Cost Phase 1 200 Mℓ/d ¹	Annual Cost 312 Mℓ/d ²
Mechanical technician	R 20,000	8	R1,920,000	R1,920,000	R4,979,986
Receptionist	R 15,000	1	R180,000	R180,000	R466,874
Cleaners	R 7,500	6	R540,000	R540,000	R1,400,621
Total		**42**	**R15,660,000**	**R15,660,000**	**R40,618,007**

Notes:
1 Estimated Operational cost required for first year of plant operation are based on 2023 Costs.
2 Estimated Operational cost required for first year of plant operation are based on 2033 Costs.

7.7.10.2.2 Other Operational Costs

The other operational costs include energy cost, chemical and sludge disposal costs. The energy cost was estimated based on the estimated mechanical load of the New Xhariep WTW with a blended energy tariff of R 1.80 kWh while the chemical and sludge disposal costs were estimated based on approximate current chemical costs and estimated usage rates. It should be noted that these costs will increase gradually as the flow to the plant increases until the design horizon.

The other operational costs are shown in for the first year of operation of the works.

Table 7-50: Estimated other Operational Costs for the New Gariep WTW (per annum)

Description	Energy Cost	Chemical Cost	Sludge Disposal Cost	Plant capacity	Total
Complete Phase¹	R6 149 520	R60 356 400	R4 232 000	312 Mℓ/d	**R70 737 920**
Phase 1 ¹	R3 942 000	R38 690 000	R2 116 000	200 Mℓ/d	**R44 748 000**
Phase 2 ²	R5 726 000	R57 640 800	R6 572 000	112 Mℓ/d	**R69 938 800**

Notes:
1 Estimated other operational cost required for first year of plant operation based on 2023 Costs
2 Estimated other operational cost required for first year of plant operation based on 2033 Costs

7.7.10.2.3 Summary

The estimated operation and maintenance budget required for the first year of operation is summarised in Table 7-51, showing an estimated minimum O&M budget requirement.

Table 7-51: Estimated Annual Operation and Maintenance Budget for the New Gariep WTW

Description	Maintenance Budget	Operational Budget	Total Budget
Complete Phase ¹	R47 871 360	R87 744 000	**R135 615 360**
Phase 1 ¹	R32 267 977	R67 071 400	**R99 339 377**
Phase 2 ²	R40 471 156	R55 837 200	**R96 308 356**

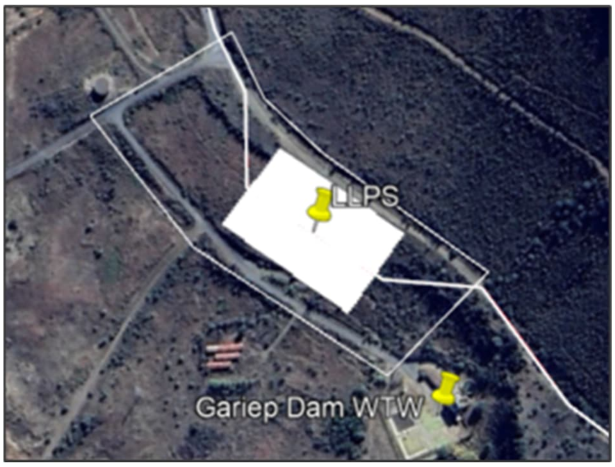
Notes:
1 Estimated operation and maintenance budget required for first year of plant operation based on 2023 Costs
2 Estimated operation and maintenance budget required for first year of plant operation based on 2033 Costs

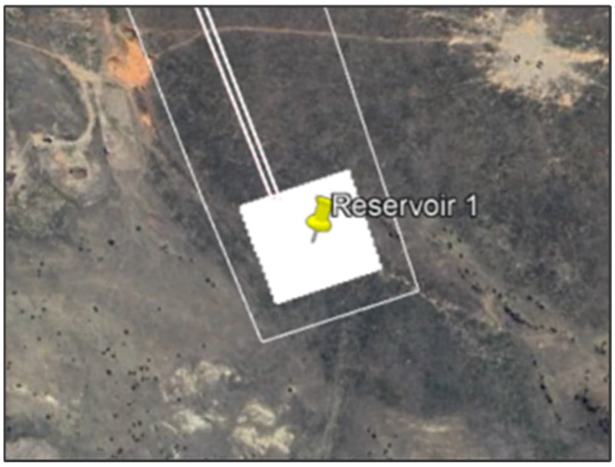
7.8 Ancillary Infrastructure

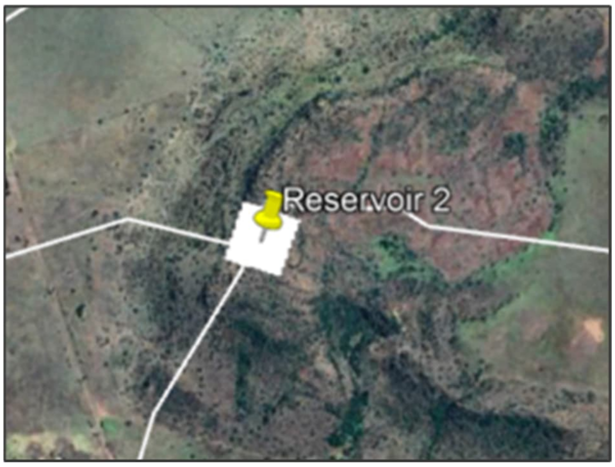
7.8.1 Power Supply

Scheme 1B is the preferred scheme for implementation. The Eskom Free State and Eskom Northern Cape offices were contacted to assess the availability of power for each of the infrastructure sites, and to determine the upgrades/infrastructure required to provide power at each site. A high-level cost estimate was undertaken for the electrical infrastructure required. Table 7-52 provides a summary of the medium and high-voltage infrastructure required for the project.

Table 7-52: Summary of medium and high-voltage upgrades

Site	Image and coordinates	Medium and high-voltage scope of works
Low-lift Pump Station (LLPS)	 • Latitude = 30°37'29.76"S • Longitude = 25°29'6.97"E	• Closest Substation from Eskom is Oranjekrag where Eskom can add two new 22kV feeder bays. • DWS to install two x 800 meter overhead powerlines or buried cables to the new LLPS. • At LLPS, install two incomer feeder bays, busbar section and two 22/11kV 5MVA transformers bays with a 11kV Indoor Circuit Breaker Building. • Approximate capital cost = R 20 million.
Water Treatment Works and High-lift Pump Station (HLPS)	 • Latitude = 30°32'30.95"S • Longitude = 25°30'58.52"E	• New 132kV Loop-In Loop-Out from the Ruigtevallei-Valley Dora 132kV Line with Eskom Switching Station. • Two 132kV incomer feeders and two 132kV out-going feeder bays including busbar section. • Firm supply 30MVA= 132/11kV • Substation Complete with metering protection, earth grid and 11kV Circuit Breaker Room for supply to the WTW and HLPS. • Approximate Cost = R 45 million.

Site	Image and coordinates	Medium and high-voltage scope of works
Reservoir 1	 • Latitude = 30°15'2.05"S • Longitude = 25°44'17.71"E	• Build 800m 11kV overhead Line from Pole SPEL 5-24-3 on Elstow 11kV Line to reservoir position. • Install 25kVA 11/0.4kV Pole mounted Transformer with metering kiosk. • Approximate Cost = R 0.5 million.
Booster Pump Station (Proposed - Site 1)	 • Latitude = 29°30'53.25"S • Longitude = 26° 5'14.73"E	• Two 66kV feeder bays and possible busbar reconfiguration at Reddersburg Substation. • Build new 66kV Double Circuit Overhead Line, 18km long. • Add new 66/11kV 10 MVA substation complete with MV and LV switchgear. • Approximate Cost = R 36 million.
Booster Pump Station (Proposed - Site 2)	 • Latitude = 29°22'13.78"S • Longitude = 26°21'7.32"E	• Add 10MVA Transformer Bay at Paradys Substation and add two 22kV Feeders Bays. • Build double overhead line 22kV 10MVA to Booster Pumpstation Option 2, 23km long. • Build new firm 10MVA 22/22kV substation complete with MV and LV switchgear. • Approximate Cost = R 31 million.

Site	Image and coordinates	Medium and high-voltage scope of works
Reservoir 2	 • Latitude = 29°19'17.44"S • Longitude = 26°23'4.76"E	• Build 500m 22kV Paradys overhead line (OHL) from Lovedale 22kV OHL to reservoir position. • Install 25kVA 11/0.4kV pole mounted transformer with metering kiosk. • Approximate cost = R 0.45 million.

The applications for the power supply to the various infrastructure sites must be submitted to Eskom during the detailed design phase of the project, whereafter Eskom must undertake the design and construction of the electrical infrastructure.

7.8.2 Alternative power sources

Table 7-31 shows the power requirements for the three main pump stations. The LLPS, the HLPS and the booster pump station require 5 MW, 16 MW and 6 MW, respectively.

The feasibility of alternative power supply sources such as hydropower or solar power was briefly considered.

The hydropower potential at a site with residual head can be calculated as:

$$Power = QgH\eta$$

With

Q = flow (m^3/s)
g = gravitational constant (9.81 m/s^2)
H = net head (m)
η = overall efficiency (0.75)

The hydraulic gradelines for the respective pipelines are shown in Figure 7-32 to Figure 7-35 with the average annual flows shown in Table 7-29. Based on average annual flows, it is only at the proposed Gariep WTW and at Rustfontein WTW where sufficient residual head is available to consider hydropower installations. The hydropower potential for these two sites is shown in Table 7-53.

Table 7-53: Hydropower potential at various sites

Site	Average flow (m^3/s)	Net head (m)	Hydropower (kW)
Gariep WTW	3.167	30	699
Rustfontein WTW	1.456	40	429

The estimated cost per kW for hydropower installations smaller than 1 MW is approximately R 40,000/kW. The estimated capital costs for hydropower installations at the Gariep WTW and Rustfontein WTW is therefore R 35 million and R 21 million, respectively when allowing for 25% standby capacity. Given that approximately 16 MW is required at the Gariep WTW, this is not considered feasible.

VCWB can evaluate the feasibility of a hydropower installation as part of the planned Rustfontein WTW upgrade as they will first need to determine the power requirements for the upgraded WTW.

Solar plants need to allow for peaking, e.g. a 2.5 MWp plant is required to feed a 1 MW demand. As such, the LLPS, HLPS and booster pump station sites will require solar plants with peaking capacities of 12.5 MWp, 40 MWp and 15 MWp, respectively. The current cost for solar plant installations in South Africa is approximately R 15 million per MW, excluding the network integration costs, i.e. the solar plants required for this project will cost roughly between R 200 million to R 600 million per plant. It might, however, still be beneficial to consider solar plants subject to the energy tariff at which the end-user can purchase the power from the solar plants. It is estimated that the HLPS and WTW will use on average about 11.5 MWh or 101 GWh per annum. At an Eskom tariff of R 1.80/kWh, this equates to an annual electricity cost of about R 181 million, which amounts to about R 2 billion when discounted at 4% over a 15-year period.

It is therefore recommended that a detailed investigation be undertaken into the viability of solar plants as alternative energy sources for the Xhariep Pipeline project.

8 System Supply Risk Analysis

8.1 Risk Analysis Approach

The system supply risk analysis involved assessing the resilience of schemes by considering potential failures at supply sources or at a WTW. This evaluation entailed the examination of the system's ability to meet the 2050 demands of the major demand centres, Bloemfontein, Botshabelo and Thaba Nchu, in the event of a failure. For instance, if Welbedacht WTW were to become unavailable, the analysis considered what percentage of demands could still be supplied from alternative sources like Gariep, Rustfontein and Maselspoort WTWs if they operate at full capacity.

Four potential failures were evaluated for each scheme, including a breakdown at the WTW or supply pipeline from:

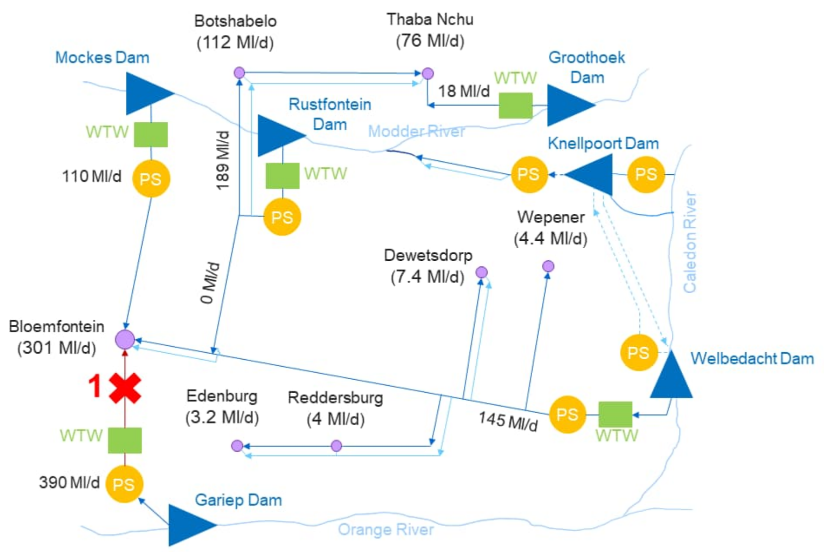
- Proposed Gariep WTW,
- Maselspoort WTW,
- Welbedacht WTW, and,
- Rustfontein WTW.

The system risk analysis was undertaken for Schemes 1A, 1B and 4B when transferring 120 million m^3/a (Schemes 1A and 1B) and 142 million m^3/a (Scheme 4B) from Gariep Dam.

8.2 Risk Analysis Scheme 1A

The risk analysis for Scheme 1A, the potable option directly supplying Bloemfontein, is summarised in Table 8-1.

Table 8-1: Risk analysis Scheme 1A summary

<table>
<tr><th>Failure Location on Scheme 1A</th><th>Description</th></tr>
<tr><td>


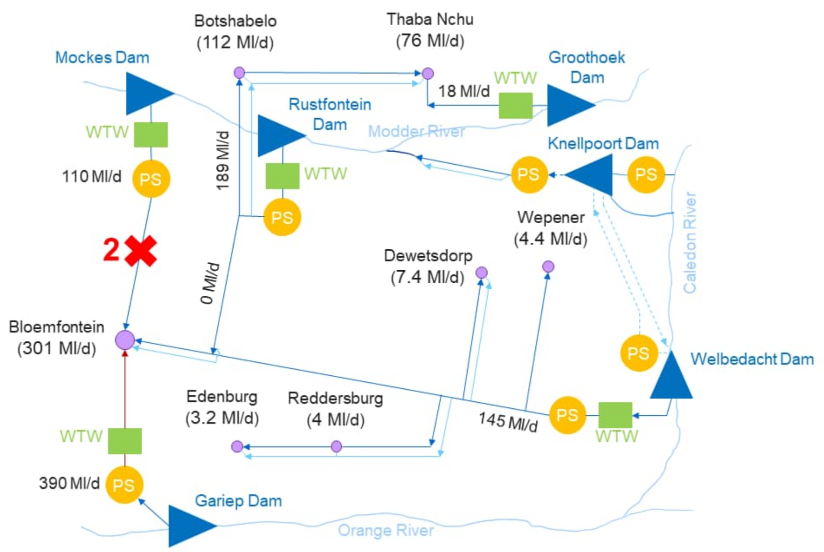
</td><td>

Failure from proposed Xhariep WTW:

Should the proposed Xhariep WTW or supply pipeline fail in Scheme 1A, the system configuration will allow 78% of Bloemfontein's 2050 demands to be met, while meeting 100% of Botshabelo's/Thaba Nchu's demands.

The supply percentages were calculated as follows:

- Bloemfontein will receive 110 Mℓ/d (Maselspoort) + 145 Mℓ/d (Welbedacht) – 19 Mℓ/d (small towns). The available 236 Mℓ/d equates to 78% of the 301 Mℓ/d demand.
- Botshabelo and Thaba Nchu will receive 189 Mℓ/d (Rustfontein) + 18 Mℓ/d (Groothoek), which is more than the 2050 demand of 188 Mℓ/d, hence meeting 100% of the demand.

</td></tr>
<tr><td>


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Failure from Maselspoort WTW:

If the supply from Maselspoort fails, the Scheme 1A configuration will still meet 100% of demands from both Bloemfontein, Botshabelo and Thaba Nchu.

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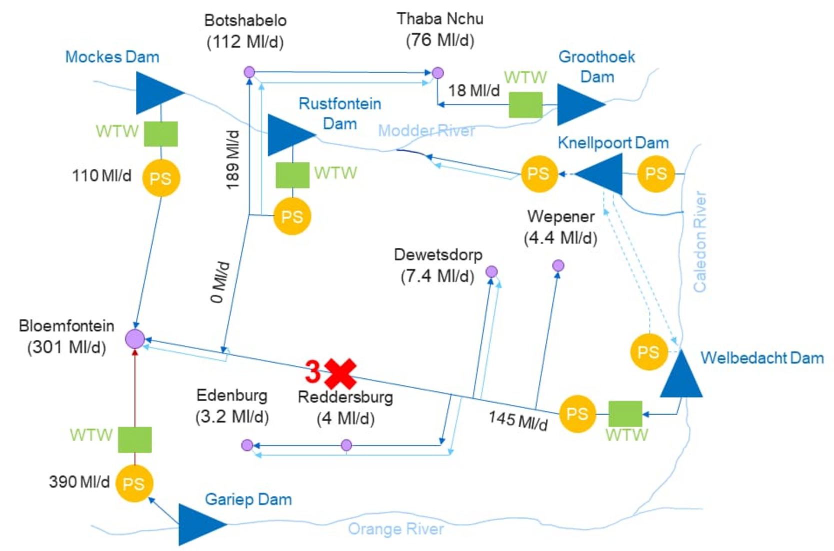
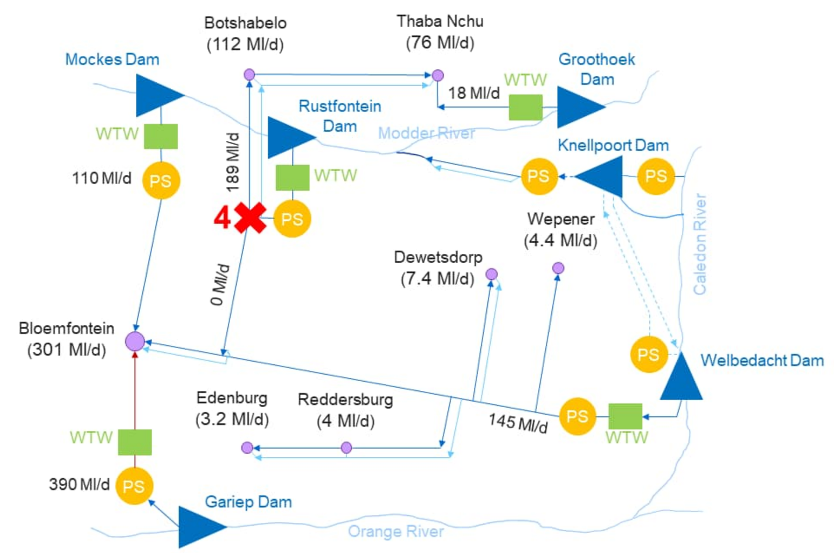
</table>

<table>
<tr><th>Failure Location on Scheme 1A</th><th>Description</th></tr>
<tr><td>
</td><td>Failure from Welbedacht WTW:
Should the Welbedacht WTW or pipeline fail, 100% of demands from both Bloemfontein, Botshabelo and Thaba Nchu will still be met.</td></tr>
<tr><td>
</td><td>Failure from Rustfontein WTW:
If the Rustfontein WTW or pipeline fail, the Scheme 1A configuration will still allow 100% of the demand from Bloemfontein to be met, but only 10% of the demands from Botshabelo and Thaba Nchu will be met from Groothoek WTW.</td></tr>
</table>

8.3 Risk Analysis Scheme 1B

The risk analysis for Scheme 1B, the potable option that ties into the bulk supply network at Bloemfontein and Rustfontein WTW, is summarised in Table 8-2.

Table 8-2: Risk analysis Scheme 1B summary

<table>
<tr><th>Failure Location on Scheme 1B</th><th>Description</th></tr>
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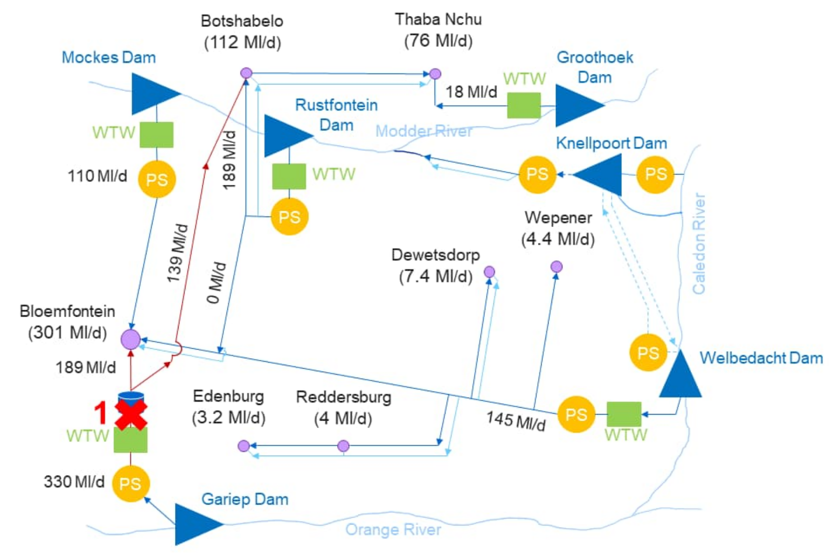

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Failure from proposed Xhariep WTW:

If the proposed Xhariep WTW or pipeline fails, the Scheme 1B configuration will allow 85% of the demands from Bloemfontein to be met and will still be able to supply 100% of Botshabelo and Thaba Nchu's demands.

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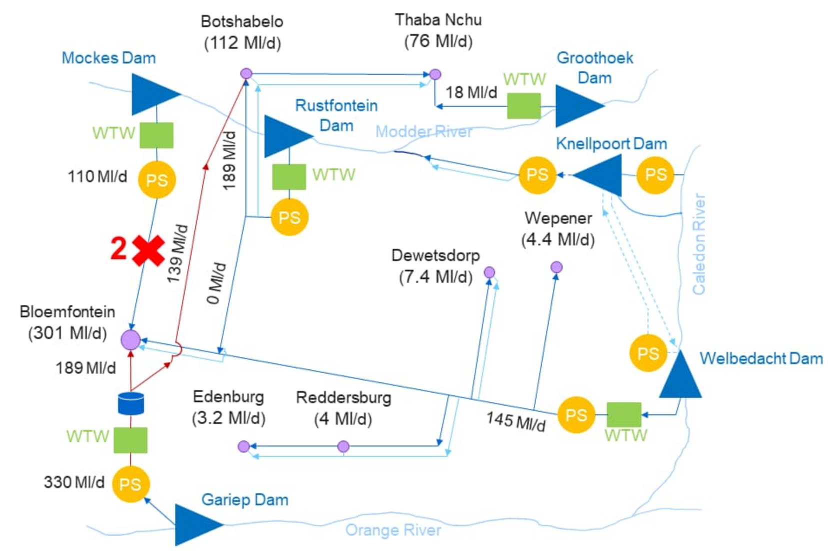

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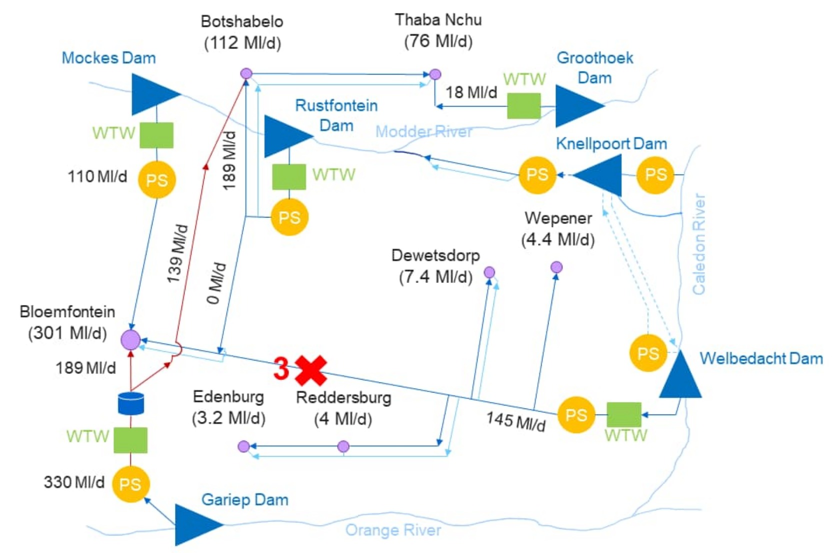
Failure from Maselspoort WTW:

Should supply from Maselspoort be interrupted, Scheme 1B will still be able to supply 100% of the demands from Bloemfontein, Botshabelo and Thaba Nchu.

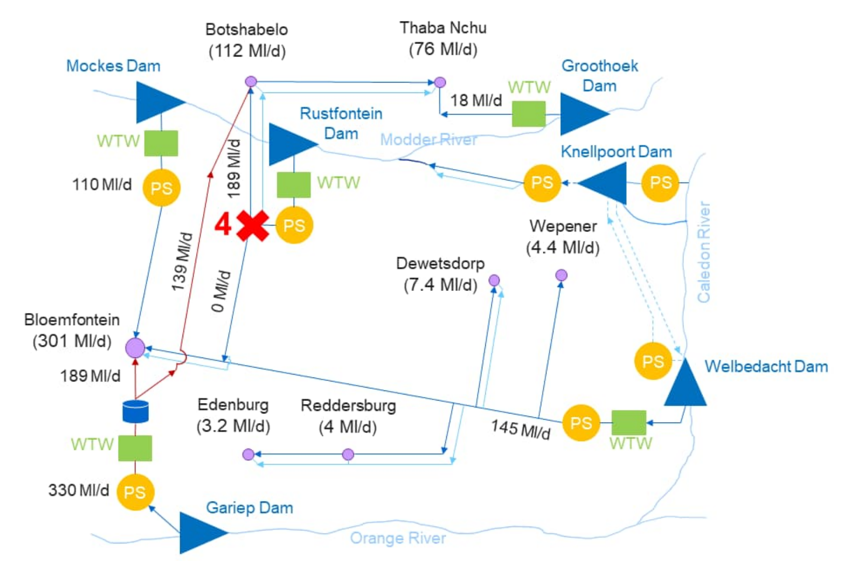
</td></tr>
<tr><td>


</td><td>

Failure from Welbedacht WTW:

If the supply from Welbedacht WTW fails, Scheme 1B will supply 99% of Bloemfontein's demands while maintaining 100% of supply to Botshabelo and Thaba Nchu.

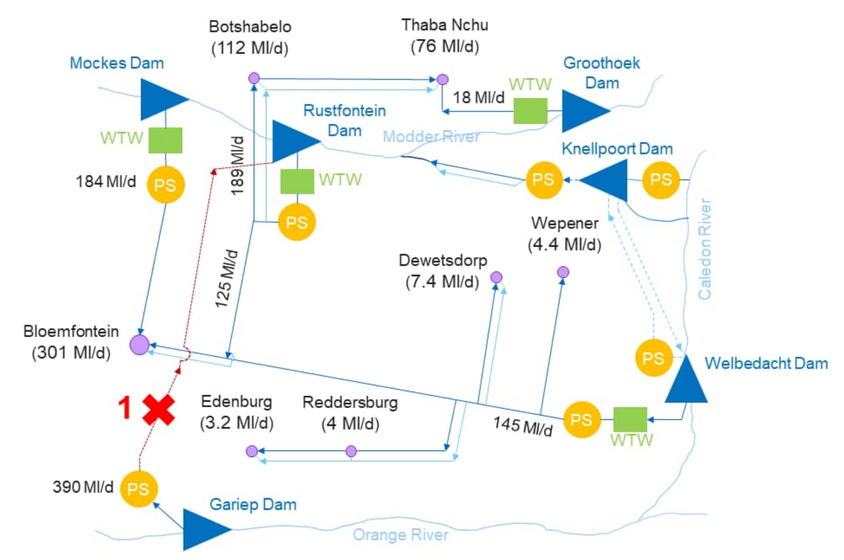
</td></tr>
</table>

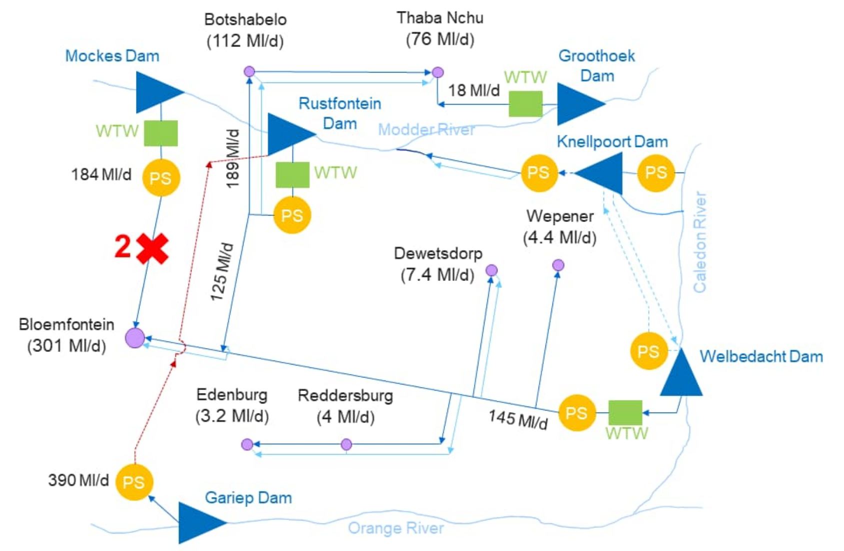
<table>
<tr><th>Failure Location on Scheme 1B</th><th>Description</th></tr>
<tr><td>
</td>
<td>Failure from Rustfontein WTW:
Should the supply from Rustfontein WTW fail, Scheme 1B will still meet 100% of Bloemfontein’s demands, and 83% of demands from Botshabelo and Thaba Nchu.</td></tr>
</table>

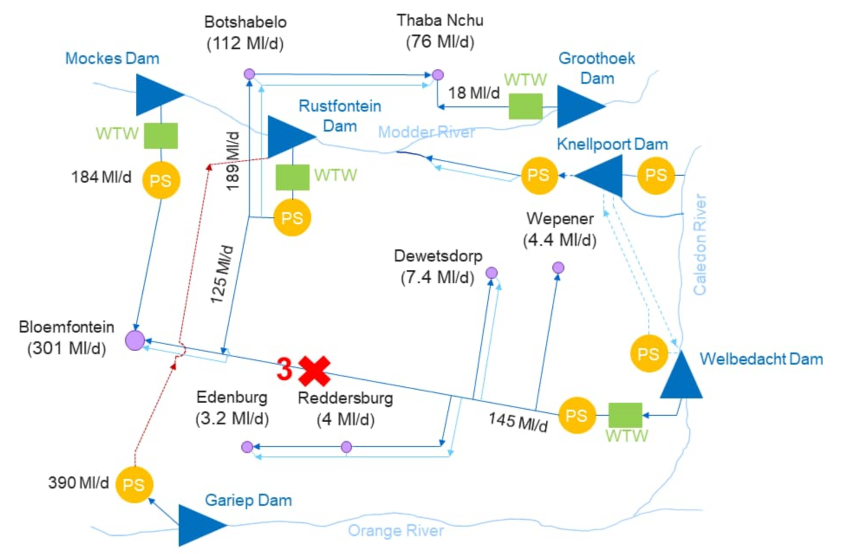
8.4 Risk Analysis Scheme 4B

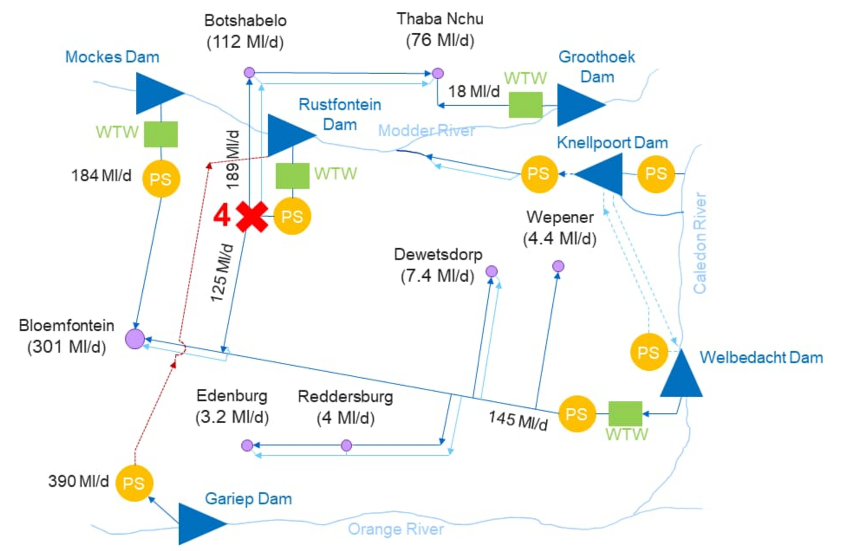
The risk analysis for Scheme 4B, the raw water option that discharges into Rustfontein Dam, is summarised in Table 8-3.

Table 8-3: Risk analysis Scheme 4B summary

<table>
<tr><th>Failure Location on Scheme 4B</th><th>Description</th></tr>
<tr><td>
</td>
<td>Failure from proposed Xhariep WTW:
The Scheme 4B configuration enables the system to maintain supply of 100% of the demands for Bloemfontein, Botshabelo and Thaba Nchu if there is a failure of supply from the proposed Xhariep WTW.</td></tr>
</table>

<table>
<tr><th>Failure Location on Scheme 4B</th><th>Description</th></tr>
<tr><td>


</td><td>
Failure from Maselspoort WTW:
If there is a failure of supply from Maselspoort WTW, the Scheme 4B can supply 90% of the demands from Bloemfontein and 100% of the demands from Botshabelo and Thaba Nchu.
</td></tr>
<tr><td>


</td><td>
Failure from Welbedacht WTW:
Should the supply from Welbedacht WTW be interrupted, the Scheme 4B configuration will still allow of the supply of 100% of demands to Bloemfontein, Botshabelo and Thaba Nchu.
</td></tr>
<tr><td>


</td><td>
Failure from Rustfontein WTW:
If supply from Rustfontein WTW fails, Scheme 4B will be able to supply 100% of Bloemfontein's demands but only 10% of the demands of Botshabelo and Thaba Nchu.
</td></tr>
</table>

8.5 Risk Analysis Comparison

Each scheme configuration has specific vulnerabilities based on its ability to convey water to the demand centres. Table 8-4 compares the risk of failure of the three schemes, indicating the percentage of the demand that can be met.

Table 8-4: System supply risk analysis comparison summary

Scheme	% of Supply to:	Failure of supply from:			
		Xhariep WTW	Maselspoort WTW	Welbedacht WTW	Rustfontein WTW
1A (Potable to MMM)	Bloemfontein	85	100	100	100
	Botshabelo & Thaba Nchu	100	100	100	10
1B (Potable hybrid)	Bloemfontein	85	100	99	100
	Botshabelo & Thaba Nchu	100	100	100	83
4B (Raw water to Rustfontein)	Bloemfontein	100	90	100	100
	Botshabelo & Thaba Nchu	100	100	100	10

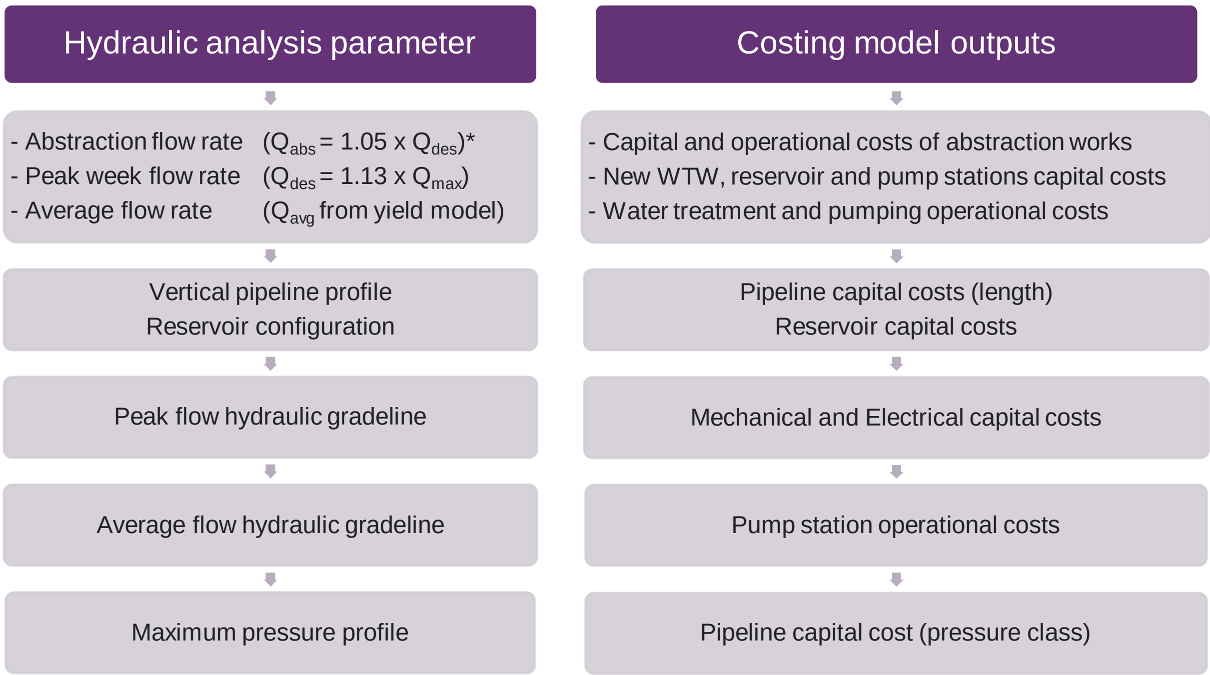
If the proposed Xhariep WTW fails, the worst performing schemes are Schemes 1A and 1B as they are only able to supply 85% of the 2050 Bloemfontein demands. However, the critical point of failure was found to be the supply from Rustfontein WTW. If it fails, Schemes 1A and 4B experiences the supply to Botshabelo and Thaba Nchu being reduced to 10% of the 2050 demands. Scheme 1B provides the most resilience and operational flexibility of the three schemes, as in the event of failure from any one of the four WTW over 80% of the 2050 demands can still be supplied at worst.

9 Financial Assessment of Schemes (Including NPVs and URVs)

9.1 Description of Hydraulic Analysis and Costing Model

A hydraulic analysis and costing model was developed for this pre-feasibility study. The purpose of the model was to analyse and compare both the technical and economic feasibility of each proposed scheme (and its sub-options) respectively. The hydraulic analysis consisted of determining the hydraulic gradelines and pressure profiles based on the flows obtained from the water resources yield modelling and took into account the pipe diameter, vertical alignment and locations of the pump stations and reservoirs along the pipeline. These gradelines were developed for both peak and average flow rates. The hydraulic gradelines at peak flow were used to determine the maximum pumping and power requirements for infrastructure sizing and capital costing. The hydraulic gradelines at average flow were used to determine the operating duty point of the pump stations and the average annual power required for operational costing. The scheme development, hydraulic analysis and infrastructure sizing processes are described in Section 7.1 to 7.5.

A flow chart showing the overarching hydraulic analysis parameters and related costing model outputs to which they are applicable is presented in Figure 9-1.



*For the financial comparison of schemes (and their sub-options), the abstraction pipeline was included as a part of the main transfer pipeline with flows at Q_{des}. The abstraction ipeline was modelled separately using Q_{abs} for the final pre-feasibility design and cost estimate presented in Section 7.6.9 and 7.6.10 respectively.

Figure 9-1: Flow chart linking hydraulic analysis parameters to financial model outputs

A more detailed description of the CAPEX determination for individual infrastructure elements in the cost model is provided in Section 9.2. The operational costs of each scheme is based on the total volume of water treated and pumped in the system at average flows obtained from the yield modelling. This accounts for the varying utilisation of existing infrastructure between schemes to provide a more holistic cost comparison. All OPEX for energy, water treatment, maintenance, security, and administration were discounted to a net present value in the cost model as described in Section 9.3.

9.2 Capital Expenditure (CAPEX)

9.2.1 Bulk Water Pipelines

The costs of the bulk water pipelines required for each scheme were calculated for both the new transfer pipeline from Gariep Dam as well as for any additional pipelines not included in the 'common' upgrades (refer to Section 7.2). All bulk transfer pipelines were assumed to be cement mortar lined steel pipes manufactured from Grade X52 steel for diameters greater than 600 mm.

The cost of the steel pipelines is a function of the diameter and pressure class. The pipe diameters required for each scheme were calculated based on the design flow rates at allowable design velocities, with the maximum working pressures obtained from the hydraulic analysis (refer to Chapter 7).

Once the diameters and maximum working pressures were determined, a hoop stress calculation was performed to calculate the minimum wall thickness required based on a maximum yield strength of 360 MPa for Grade X52 steel. This minimum wall thickness was used to select the standard steel plate thickness for that diameter and pressure class. The weight of the steel required for each section of pipe could then be calculated and used in conjunction with the rates described below to determine the pipe material costs.

The capital cost of the bulk water pipelines included both the cost to supply the pipe material as well as the estimated installation costs (trenching, backfill etc). A summary of the rates and assumptions used to calculate these costs in the model is provided in Table 9-1.

Table 9-1: Summary of rates and pricing assumptions used for costing of steel pipelines

Item	Rate	Unit	Assumption
Pipe supply costs			
Steel (Grade X52)	35	R/kg	Standard steel thickness for pressure class
Cement mortar lining	3	R/kg	15 mm thickness
Pipe installation costs			
Excavation (in all materials)	170	R/m³	Average cover to pipe 1.5 m
Excavation (in hard rock)	1100	R/m³	20% total excavation
Imported bedding and blanket	400	R/m³	80% of all bedding and blanket material
Bedding and blanket from trench	80	R/m³	20% of all bedding and blanket material
Additional costs			
Pipe specials	20	%	Of total pipe supply cost
Valves	25	%	Of total pipe supply cost
Chambers	20	%	Of total pipe supply cost
Other (road or stream crossings)	20	%	Of total pipe supply cost

The trench excavation dimensions were based on SANS 1200 DB where the trench bottom width includes 300 mm on either side of pipe. The bedding and blanket quantities are based on the requirements of SANS 1200 LB, namely 100 mm of selected bedding under pipe and 300 mm of selected blanket above pipe.

9.2.2 Reservoirs

The civil works capital cost of the reservoirs was calculated based on a fixed rate of R1.5 million/Mℓ as derived from costs for completed reservoir projects of a similar nature. The total cost of the reservoir was a function of the reservoir sizes which were calculated to accommodate 6 hours of storage at peak week flows for each scheme (refer to Section 7.3.5).

9.2.3 Pump Stations

The civil works capital cost of the pump stations was priced as a fixed lumpsum of R30 million per pump station building based on the estimated footprint. This lump sum was included in the costing of high-lift and booster pump stations required by each scheme. The low-lift pump station was priced with a fixed lump sum of R20 million for potable water Schemes 1, 1A and 1B as it was located next to existing access roads. However, this was increased to R30 million for the final pre-feasibility costing of Scheme 1B given that the proposed site could have restricted working space due to the location of existing structures and overhead powerlines, which will increase the construction costs.

The mechanical and electrical (M&E) capital costs for each scheme were calculated as function of the peak power required by each pump station in that scheme. The power required was calculated at the maximum pump duty point (peak flows) as detailed in Section 7.3.4 and multiplied by a fixed rate of R30 000/kW to obtain the capital cost.

9.2.4 Water Treatment Works

A capital cost function was developed for the pricing of the additional water treatment works capacity required by each scheme. This cost function was used to price both the new proposed WTWs for the potable schemes as well as upgrades to existing WTWs required by the raw water schemes. The total new and/or additional water treatment capacity required for each scheme was calculated based on the water treatment outflows obtained from the water resources yield modelling.

The capital cost function was developed based on recent detailed cost estimates for WTWs of a similar nature (conventional or direct filtration). The function was calculated based on a cost of R1000 million for a 50 Mℓ/d plant and R 3,500 million for a 200 Mℓ/d plant. A linear curve was used to calculate the capital cost rate in Rands/Mℓ. A summary of the required capacity upgrades and the capital costs for each scheme are summarised in Table 9-2.

Table 9-2: Capital cost rates calculated for the WTW capacity increases required by each scheme

Scheme No.	Description of scheme specific WTW upgrades required*	Total capacity increase	Total annual volume treated at all WTWs	Capital cost rate
	Units	**Mℓ/d**	**Mm³/a**	**Rands / Mℓ**
1	Proposed new water treatment works	165	133.2	18,083,333
2	Upgrade of 64 Mℓ/d at Maselspoort WTW	64	115.3	19,766,667
3	Upgrade of 64 Mℓ/d at Maselspoort WTW	64	116.5	19,766,667
4	Upgrade of 64 Mℓ/d at Maselspoort WTW	64	130.3	19,766,667
1A	Proposed new water treatment works	390	185.9	14,333,333
4B	Upgrade of 125 Mℓ/d at Rustfontein WTW and 64 Mℓ/d at Maselspoort WTW	189	182.2	17,683,333
1B*	Proposed new water treatment works	330*	182.2	15,333,333

*The final capital cost estimate for the proposed new WTW for Scheme 1B was based on the updated stochastic flows detailed in Section 7.6.9 (312 Mℓ/d) and a more detailed capital cost estimation provided in Section 7.7.9.

It is noted that the cost function shown in Table 9-2 was used for financial comparison of the various schemes and sub-options. Compared to the more detailed cost estimate presented for the Option 1B WTW in Section 7.7.9, it is evident that the cost functions in Table 9-2 were more conservative, which might have introduced a marginal bias towards the raw water schemes which required smaller WTW capacity upgrades in relation to the potable water schemes. The final scheme selected for the pre-feasibility design is the potable Scheme 1B which was shown to be comparable in price to its equivalent raw water Scheme 4B regardless of this potential bias.

9.3 Operational Expenditure (OPEX)

9.3.1 Discount Rates and Planning Horizon for Net Present Value (NPV)

To fairly compare the cost of the schemes, it was necessary to reduce the total operation and maintenance (O&M) costs to net present values (NPVs) over the intended planning horizon. The NPV was calculated for the following operation and maintenance components required by each scheme:

- Water treatment required at all treatment works,
- Energy required at all new and existing pump stations,
- Mechanical replacement at new pump stations,
- Maintenance of civil and mechanical & electrical (M&E) infrastructure, and,
- Administration and security.

A discount rate of 6% over a discount period (planning horizon) of 45 years was originally used to calculate the O&M NPV for each scheme. This approach was used for the comparison of the original four schemes modelled with a maximum transfer volume of 60 million m^3/s from Gariep Dam, as well as the additional three schemes modelled with maximum transfer volumes of 120 million m^3/a (Schemes 1A), 139 million m^3/a (Scheme 1B) and 142 million m^3/a (Scheme 4B), respectively.

In the final pre-feasibility cost estimate of Scheme 1B presented in Section 7.6.10, an economist was consulted regarding the discount rate and discount period to be adopted to ensure alignment with the financial analysis that will be required during the detailed feasibility phase of the project. Given that South African bond rates which lie between 8% and 11% (note that the current 10-year bond rate is close to 10%), and inflation rates between 4.5% and 6%, it was recommended to adopt a discount rate of 4% with a discount period of 30 years. A sensitivity analysis will, however, be undertaken during the detailed feasibility phase to test discount rates of 3% and 6% (or even 8%) as potential lower and upper limits. It was noted that discount periods longer than 30 years could result in an under estimation of the operational and maintenance costs of the project, e.g. using a 45-year discount period will increase the NPV of the O&M costs by only 12% compared to when a 30-year discount period is used. Therefore, using a long discount period can result in schemes with very low CAPEX but very high OPEX being favoured.

9.3.2 Unit Reference Values (URV)

The original four schemes investigated (i.e. Schemes 1, 2, 3 and 4) had different historic firm yields (HFY's) so comparing the costs of the schemes had to be undertaken based on a URV approach. The projected water demands of each scheme was discounted at 6% over 45 years, similar to what was done for the O&M costs, to calculate the NPV of the discounted water demands.

The original four schemes were assumed to reach the maximum water demand by 2035 after which they remained constant. The three additional higher transfer schemes (i.e. Schemes 1A, 1B and 4B) assumed that the water demand will match the total system demands by 2035 and thereafter increase linearly to reach the maximum projected water demand by 2050 after which they remained constant. A summary of the discounted water demands, and reference intervals used for the URV calculations is provided in Table 9-3.

Table 9-3: Discounted water demands used for unit reference value calculations

Water Demand	Scheme No.				
	1	2	3	4	1A, 4B & 1B (hybrid)
Historic firm yield (Mm^3/a)	137	119	120	134	186
Discounted HFY (6% | 45 years)	1986	1797	1808	1955	2208
Water demand 2023 (Mm^3/a)	109	109	109	109	109
Water demand 2035 (Mm^3/a)	137	119	120	134	139
Water demand 2050 (Mm^3/a)	137	119	120	134	186

The URV was calculated by dividing the total NPV cost (capital and O&M) by the discounted water demand which provides a comparative estimate of each scheme's cost per cubic meter of water. The three higher transfer schemes had the same water demands and could also be directly compared using the total NPV costs. It is noted that the URV should not be confused with a water tariff and was simply used as a comparative unit water cost value.

9.3.3 Water Treatment Cost

The cost to treat one cubic meter of water was developed based on the detailed cost estimate of a conventional water treatment works with a reasonably good water supply quality that does not require advanced residual treatment (sludge management). The estimate assumes an 80% assurance of supply (i.e. that the plant will run at 80% of its design capacity to allow for peak periods and maintenance) and no allowance was made for capital recovery (amortisation costs were excluded).

The OPEX of the WTWs includes both fixed and variable costs as summarised in Table 9-4.

Table 9-4: Summary of water treatment costs used for OPEX cost model

Description	Rand/a	R/m^3	Percentage of total
Fixed costs			
Civil maintenance	9,217,233	0.66	19.68%
Mechanical maintenance	14,374,584	1.03	30.69%
Salaries and wages	4,837,400	0.35	10.33%
Security	600,000	0.03	1.28%
Variable costs			
Chemicals	17,502,480	1.00	37.37%
Electricity	304,978	0.02	0.65%
TOTAL	**46,836,675**	**3.08**	**100%**

The treatment cost of R3.08 per cubic meter was assumed to remain constant in real terms over the discount period of 45 years and the NPV treatment cost was calculated to be R50.68 per cubic meter at a discount rate of 6%. This was multiplied by the total annual volume of water treated in each scheme (shown in Table 9-2) to get the total NPV cost of water treatment.

9.3.4 Energy Cost

The pumping energy required by each scheme was based on the average annual flows obtained from the water resources yield modelling. The average duty points and power requirements were determined at existing pump stations as well as the proposed new pump stations in the scheme.

The cost of energy is highly variable and depends on the supply authority and the pump operating times. Therefore, a more conservative rate of R1.80 per kWh was used in the OPEX cost function. The energy cost was assumed to grow annually at 2% above inflation for the same duration as the discount period to allow for uncertainties in South Africa's energy market.

The total energy cost per pump station was calculated as a function of the total volume transferred per annum. For example, the high-lift pump station in Scheme 1B would be required to lift 87,560,917 m³ of water by 362 m (head at average flow of 2.776 m³/s) every year. This equates to an average power requirement of 14,083 kWh required over 8760 hours (per annum) resulting in an annual energy cost of R 222 million. This was divided by the total annual volume and total head to develop a rate of 0.701 cents/m³/m which was escalated at 2% and then discounted at 6% over 45 years to give an NPV rate of 15.40 cents/m³/m. This rate could be multiplied by the volume of water pumped at the average operating head of each pump station in the scheme to yield the total NPV energy cost for that scheme.

9.3.5 Maintenance and Administration Cost

The maintenance and administration costs were estimated for the proposed new civil works and M&E infrastructure required by each scheme. These annual costs were calculated as a fraction of the corresponding capital expenditure as follows:

- Civil works maintenance cost = 0.5% of all civil works capital cost (including contingencies).
- M&E maintenance cost = 4% of M&E capital costs.
- Administration cost = 1% of total capital costs (including contingencies).

These costs were assumed to remain fixed over the planning horizon and were discounted at 6% to provide NPV maintenance and administration rates as follows:

- Civil works maintenance cost = R82 per R1000 capital cost of civil works.
- M&E maintenance cost = R658 per R1000 capital cost of M&E.
- Administration cost = R165 per R1000 total capital cost.

These NPV rates were multiplied by the applicable capital costs in each scheme to determine the total maintenance and administration costs.

Additionally, the OPEX cost model included a replacement of mechanical and electrical equipment every 15 years. Calculated at the same discount rate and period this equated to an NPV rate of R1591 per R1000 capital cost of M&E.

Safety and security costs were included for the proposed pump station and reservoirs in each scheme. These costs were based on 24-hour manned security at R300,000 salary cost per annum. The NPV cost at 6% over 45 years was calculated to be R4,937,000 per geographic location.

9.4 Results of Financial Comparisons

This section presents the total CAPEX, OPEX, NPV and URV calculated for each scheme using the cost model described in Sections 9.1 to 9.3. It is noted that the costing only applies to the scheme specific infrastructure described in Sections 7.3 to 7.6 and excludes all 'common' infrastructure upgrades required by all schemes as identified in Section 7.2.

The NPV costs presented in this section, which are for comparative purposes between the different schemes, are all based on a 6% discount rate over a period (planning horizon) of 45 years as described in Section 9.3.1.

9.4.1 Original Schemes 1 to 4 at Maximum Transfer of 60 million m³/a

The infrastructure requirements and cost estimates of Schemes 1 to 4 are summarised in Table 9-6. These schemes could not be compared on their NPVs given the varying HFYs and were thus compared using their URVs.

Scheme 4, conveying raw water from Gariep Dam to Rustfontein Dam, was shown to be the most economical scheme with a URV of R13.05/m³. Scheme 1, conveying potable water from Gariep Dam to Bloemfontein, was shown to be the second most economical scheme with a URV of R14.03/m³ (7.5% more expensive than Scheme 4). Scheme 2, conveying raw water to Knellpoort Dam, and Scheme 3, conveying raw water to the Novo Outfall Structure, were shown to be the two least economical options with URVs of R15.07/m³ and R14.85/m³, respectively (15.4% and 13.7% more expensive than Scheme 4).

None of the original four schemes could meet the 2050 demands at a maximum transfer volume of 60 million m³/annum from Gariep Dam. Additional yield modelling was undertaken for Schemes 1 and 4 to determine the required transfer flows from Gariep Dam that would satisfy the 2050 water demands. Since Scheme 4 was the most economical raw water scheme, Schemes 2 and 3 were not investigated further.

9.4.2 Schemes 1A, 4B and 1B at Maximum Transfer of 120 and 142 million m³/a

It was shown from the further water resources yield modelling that volumes of 120 million m^3/a (Schemes 1A), 139 million m^3/a (Scheme 1B) and 142 million m^3/a (Scheme 4B) had to be transferred from Gariep Dam to satisfy the 2050 water demands, i.e. to match the historic firm yield with the total 2050 water demand. However, when considering the distribution of water within the GBWSS, Scheme 1A still had a 23% shortfall in supplying the 2050 demands of Botshabelo and Thaba Nchu, whereas Scheme 4B had a marginal 2.5% shortfall in supplying these same demands. Scheme 1B was able to supply 100% of the 2050 demands to all demand centres.

The infrastructure requirements and cost estimates of Schemes 1A, 1B and 4B are summarised in Table 9-7. Since the HFY for all these schemes is 186 million m^3/a, the schemes can be compared on their total NPV or URV.

The raw water Scheme 4B was shown to be more economical than the potable water Scheme 1A with a URV of R22.12/m³ compared to R22.80/m³. Scheme 1B has a URV of R22.75/m³. All three schemes were very similar in cost with a 3.1% difference in their total NPVs and URVs, which was lower than the expected level of accuracy of the cost estimates for feasibility studies. Therefore, the schemes were considered equal in terms of economic feasibility and selection was based on other factors that include overall flexibility and resilience, environmental impacts, etc. Chapter 8 presents the outcome of the system risk analysis, whereas high-level comment on the socio-economic and institutional considerations are provided in Chapter 10. A high-level comparative assessment of potential environmental impacts between Schemes 1/1A/1B, 2, 3 and 4/4B is provided in Chapter 11.

9.4.3 Sensitivity Analysis of NPV and URV

The discount rate and discount period (planning horizon) selected for the NPV and URV calculations had a direct impact on the OPEX component of each scheme’s total cost. Therefore, a sensitivity analysis was undertaken to ensure a fair financial comparison between the higher transfer Schemes 1A, 1B and 4B. This analysis consisted of costing the schemes using three different discount rates of 4%, 6% and 8% over two discount periods of 30 years and 45 years. The results of this analysis are presented in Table 9-5.

Table 9-5: Results of sensitivity analysis of URVs at different discount rates and planning horizons

.	NPV Discount rate | Project time horizon						
Value	4% | 30 yr	6% | 30 yr	8% | 30 yr	4% | 45 yr	6% | 45 yr	8% | 45 yr	Avg
NPV of HFY	2480	1911	1519	3093	2208	1666	2480
Unit	Percentage difference in O&M NPV and URV from lowest cost scheme (Scheme 4B)						
1A	+2.8	+3.6	+4.3	+2.0	+3.1	+4.0	+3.3
4B	0	0	0	0	0	0	0
1B	+2.9	+2.8	+2.7	+3.0	+2.8	+2.7	+2.8

It was shown that Scheme 4B remained the most economical option in terms of the NPV and URV. This was used as the reference to compare the other two schemes. Scheme 1B remained the second most economical alternative except at discount rates of 4% where Scheme 1A was shown to be cheaper. This is expected as Scheme 1B has a higher OPEX cost relative to its CAPEX cost when compared to Scheme 1A. Therefore, at lower discount rates the OPEX cost of scheme 1B increased to a point where it outweighed the overall benefit of its lower capital cost compared to Scheme 1A.

On average Scheme 1B remained the second most economical alternative and was within 3% of the most economical Scheme 4B across all discount rates and discount periods. It was concluded that the impact of different discount rates and periods would not significantly affect the financial comparison between these three schemes or alter the decision making with regards to selection. Therefore, Scheme 1B was still the preferred option for implementation.

9.4.4 Scheme 1B optimisation

Following the selection of Scheme 1B for implementation, three configurations of the scheme were developed to optimise the infrastructure layout and costs as described in Section 7.6.4. A summary of the infrastructure requirements and cost estimates for these three configurations is provided in Table 9-8.

The NPV and URV of Configuration 1B2 was shown to be the lowest and was within 2% of both alternative configurations 1B1(A) and 1B1(B). Configuration 1B1(A) was shown to be marginally cheaper than Configuration 1B1(B). Therefore, the configurations were considered equal in terms of economic feasibility and selection was based on the technical/practical assessment instead. This assessment is discussed in Section 7.6.8 and it was concluded that Configuration 1B1(A) would be preferred for implementation.

Table 9-6: Results of financial comparison for original schemes 1 to 4 with maximum annual transfer of 60 Mm³/a

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme comparison of original transfer at 60 Mm³ / annum	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**Mℓ**	**Mm³/annum**	**Mm³/annum**	**Mm³**	**Million Rands**	**(Million Rands)**	**Million Rands**	**R/m³**	**R/m³**	**%**
Scheme 1 [Sc5b] Potable water from Gariep Dam to Bloemfontein	181.2	DN1600	1411 | 348	2148 | 367	-	-	165	133.2	44.5	1986	11895	12543	24438	6.32	12.31	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1434 | 199	-	-	-	-	-	45.2	1986	0	1384	1384	0.70	0.70	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1293 | 247	-	-	-	-	-	40.8	1986	0	1549	1549	0.78	0.78	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	734 | 136	-	-	-	-	-	23.2	1986	0	485	485	0.24	0.24	-
										Total	**11895**	**15962**	**27857**	**8.04**	**14.03**	**+7.5**
Scheme 2 [Sc4b(ii)] Raw water from Gariep Dam to Knellpoort Dam	190.4	DN1300	1035 | 298	1901 | 418	1035 | 132	1901 | 207	64	115.3	32.6	1797	9948	11648	21597	6.48	12.02	-
+ Maselspoort pipeline and PS upgrades	33.5	DN800	257 | 135	570 | 210	-	-	Incl above	Incl above	8.1	1797	498	423	921	0.24	0.51	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1297 | 194	-	-	-	-	-	40.9	1797	0	1224	1224	0.68	0.68	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	963 | 244	-	-	-	-	-	30.4	1797	0	1142	1142	0.64	0.64	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	2438 | 185	-	-	-	-	-	76.9	1797	0	2194	2194	1.22	1.22	-
										Total	**10446**	**16632**	**27078**	**9**	**15.07**	**+15.4**
Scheme 3 [Sc4c(i)] Raw water from Gariep Dam to Novo outfall	197.8	DN1300 and DN1400	1645 | 361	1901 | 377	1645 | 231	1901 | 248	64	116.5	51.9	1808	8918	13811	22729	7.64	12.57	-
+ Maselspoort upgrades	33.5	DN800	235 | 133	570 | 210	-	-	Incl above	Incl above	7.4	1808	498	405	903	0.22	0.50	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1373 | 197	-	-	-	-	-	43.3	1808	0	1312	1312	0.73	0.73	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	985 | 244	-	-	-	-	-	31.1	1808	0	1169	1169	0.65	0.65	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	1054 | 141	-	-	-	-	-	33.2	1808	0	723	723	0.40	0.40	-
										Total	**9416**	**17421**	**26836**	**10**	**14.85**	**+13.7**
Scheme 4 [Sc5f] Raw water from Gariep Dam to Rustfontein Dam	203.9	DN1300	1431 | 378	1901 | 412	1431 | 78	1901 | 104	64	130.3	45.1	1955	8473	12718	21191	6.51	10.84	-
+ Maselspoort upgrades	33.5	DN800	299 | 140	570 | 210	-	-	Incl above	Incl above	9.4	1955	498	459	957	0.24	0.49	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	1390 | 197	-	-	-	-	-	43.8	1955	0	1332	1332	0.68	0.68	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1219 | 246	-	-	-	-	-	38.4	1955	0	1457	1457	0.75	0.75	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	863 | 138	-	-	-	-	-	27.2	1955	0	578	578	0.30	0.30	-
										Total	**8971**	**16544**	**25515**	**8**	**13.05**	**100**

Table 9-7: Results of financial comparison for additional schemes 1A, 4B and 1B (Hybrid) with maximum annual transfer of 142 Mm³/a

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Present Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme comparison of increased transfer at 142 Mm³ / annum	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**Mℓ**	**Mm³/annum**	**Mm³/annum**	**Mm³**	**Million Rands**	**(Million Rands)**	**Million Rands**	**R/m³**	**R/m³**	**%**
Scheme 1A [Sc5b] Potable water from Gariep Dam to Bloemfontein	181.2	2 x DN1500	2925 | 356	5085 | 399	2925 | 89	5085 | 119	390	185.9	92.2	2208	25120	21983	47103	9.96	21.33	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	941 | 185	-	-	-	-	-	29.7	2208	0	844	844	0.38	0.38	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	726 | 243	-	-	-	-	-	22.9	2208	0	856	856	0.39	0.39	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	1926 | 165	-	-	-	-	-	60.7	2208	0	1544	1544	0.70	0.70	-
										Total	**25120**	**25226**	**50347**	**11.42**	**22.80**	**+3.1**
Scheme 4B [Sc5f] Raw water from Gariep Dam to Rustfontein Dam	203.9	2 x DN1400 & DN1600	3016 | 367	4500 | 408	-	-	189	182.2	95.1	2208	21317	20941	42258	9.48	19.14	-
+ Maselspoort upgrades	33.5	DN800	706 | 217	570 | 210	-	-	Incl above	Incl above	22.3	2208	505	1034	1539	0.47	0.70	-
+ New pipeline from Rustfontein to Bloemfontein	50.2	DN1000	63 | 98	920 | 242	-	-	-	-	-	2208	914	199	1112	0.09	0.50	-
+ Rustfontein pump upgrades + operating cost (to Bloemfontein)	Varies	Equivalent DN1400	63 | 98	1440 | 156	-	-	-	-	2.0	2208	119	265	384	0.12	0.17	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1529 | 249	-	-	-	-	-	48.2	2208	0	1849	1849	0.84	0.84	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	475 | 133	-	-	-	-	-	15.0	2208	0	307	307	0.14	0.14	-
										Total	**22855**	**25992**	**48847**	**12**	**22.12**	**100**
Scheme 1B [Sc5b] Potable water from Gariep Dam to Rustfontein	186.1	2 x DN1400	2776 | 362	4294 | 401	2776 | 210	4294 | 233	330	182.2	87.5	2208	21846	24675	46521	11.17	21.07	-
+ Gravity pipeline to Longridge reservoir from command reservoir	26.0	DN1200	-	-	-	-	-	-	69.2	2208	663	144	808	0.07	0.37	-
+ Gravity pipeline to Rustfontein from command reservoir	25.7	DN1100	-	-	-	-	-	-	50.7	2208	590	128	718	0.06	0.33	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	-
										Total	**23099**	**27125**	**50224**	**12.28**	**22.75**	**+2.8**

Table 9-8: Results of financial comparison for optimization of scheme 1B (Hybrid)

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Present Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme 1B optimization	km	mm	ℓ/s | m	ℓ/s | m	ℓ/s | m	ℓ/s | m	Mℓ	Mm³/annum	Mm³/annum	Mm³	Million Rands	(Million Rands)	(Million Rands)	R/m³	R/m³	%
Scheme 1B1A [Sc5b] Potable water from Gariep Dam to Rustfontein	186.9	2 x DN1400	2776 | 363	4294 | 403	2776 | 106	4294 | 138	330	182.2	87.5	2208	21493	22965	44458	10.40	20.13	-
+ Gravity pipeline to Brandkop reservoir from command reservoir	31.4	DN1600	-	-	-	-	-	-	69.2	2208	1100	239	1339	0.11	0.61	-
+ Gravity pipeline to Rustfontein from command reservoir	24.5	DN1400	-	-	-	-	-	-	68.9	2208	648	141	789	0.06	0.36	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	-
										Total	**23241**	**26920**	**50161**	**12.19**	**22.72**	**+1.2**
Scheme 1B1B [Sc5b] Potable water from Gariep Dam to Botshabelo	186.9	2 x DN1400	2776 | 363	4294 | 403	2776 | 106	4294 | 138	330	182.2	87.5	2208	21493	22965	44458	10.40	20.13	-
+ Gravity pipeline to Brandkop reservoir from command reservoir	31.4	DN1600	-	-	-	-	-	-	69.2	2208	1100	239	1339	0.11	0.61	-
+ Gravity pipeline to Botshabelo from command reservoir	30.3	DN2000	-	-	-	-	-	-	68.9	2208	1571	342	1913	0.15	0.87	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	579 | 180						18.3	2208	0	506	506	0.23	0.23	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	
										Total	**24164**	**26230**	**50394**	**11.88**	**22.82**	**+1.7**
Scheme 1B2 [Sc5b] Potable water from Gariep Dam to Rustfontein	180.2	2 x DN1400 & DN1600	2776 | 363	4294 | 403	-	-	330	182.2	87.5	2208	22081	20695	42776	9.37	19.37	-
+ Pumped pipeline to Brandkop reservoir from command reservoir	36.4	DN1500	1489 | 49	2193 | 85	-	-	-	-	69.2	2208	1171	777	1948	0.35	0.88	-
+ Gravity pipeline to Rustfontein from command reservoir	29.3	DN1600	-	-	-	-	-	-	68.9	2208	1025	223	1248	0.10	0.57	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	
										Total	**24277**	**25271**	**49547**	**11**	**22.44**	**100**

10 Economic, Financing and Institutional Considerations

This section of the report assesses socio-economic, financing, and institutional considerations to determine whether the drivers will influence the selection of any scheme. A detailed socio-economic, financing, and institutional evaluation of the preferred scheme will be undertaken in the feasibility phase of this study.

10.1 Socio-Economic Considerations

Schemes 1A, 1B and 4B will serve the same regions and areas, ensuring uniform coverage except for Scheme 4B, which cannot supply potable water to towns along the pipeline route. Scheme 1A also cannot supply 100% of the demands to Botshabelo and Thaba Nchu, but in general consumer service remains consistent across all schemes, ensuring equal access to water and service levels. The selection of the most socially and economically preferable scheme must consider the following factors:

- Cashflow, capital and operational costs for each option,
- Equitable distribution of water volume among user groups,
- Tariff structure and affordability for consumers,
- Mitigation strategies for any projected income shortfalls, and,
- Assessment of the economic and social value of each option.

Initial assessment against the abovementioned factors indicated that socio-economic considerations will not dictate the selection of a scheme, although schemes providing potable water have the added benefit of reaching additional end-users.

10.2 Financing Considerations

The choice of financing arrangements is contingent upon the financial health of each scheme. Two broad financing categories are delineated based on their commercial or social orientation:

- **Commercially viable schemes** can support cost recovery through tariffs which are affordable to consumers. Such projects are financed through commercial sources like debt or loans without government guarantees.
- **Support schemes** are unable to recover costs through tariffs, but must cater to basic needs, necessitate public funding and financing, including operational subsidies and grants.

The schemes under consideration serve the same market, offer comparable service levels (e.g. area served, reliability) and at similar costs. The three schemes will also have similar social and commercial characteristics, i.e. one scheme will not be considered ‘social’ and another scheme ‘commercial’. The same financing plan will therefore be applicable to all three schemes.

It is worth noting that different potential implementing institutions, e.g. VCWB, MMM, Trans-Caledon Tunnel Authority (TCTA), etc., may have varying creditworthiness and could influence who the implementing agent/party will be.

Schemes 1A, 1B, and 4B will have the same financing plans and financing considerations did not therefore identify a preferred scheme.

10.3 Institutional Considerations

Institutional arrangements are determined by the implementing institution rather than by the schemes themselves. The principle of "structure follows functions" emphasises the importance of first defining functions (or schemes) and subsequently structuring institutions accordingly.

The layout of each scheme may influence the choice of the implementing institution, e.g.:

- Direct water supply to the MMM with take-offs to other users may favour MMM as the implementing institution, and,
- Schemes with branching-off points to MMM and other users might require regional or national implementing institutions, e.g. TCTA or VCWB.

For the same institution, the institutional arrangements would be the same for all the schemes.

Based on the above, the institutional considerations did not identify a preferred scheme.

11 Environmental Considerations

The Zutari environmental team conducted an environmental screening exercise during the initial infrastructure optimisation to assess potential sensitivities and identify any fatal flaws from an environmental authorisation perspective for the Xhariep Pipeline project. The schemes evaluated were Schemes 1 to 4, as shown in Figure 1-2. The following outcomes were derived from this assessment:

- The Department of Forestry, Fisheries, and Environment (DFFE) screening tool was applied to evaluate sensitivities per scheme, categorising them into sensitivity ratings of very high, high, medium, or low. Specialist studies required in accordance with the National Environmental Management Act (NEMA) protocols were identified based on these ratings.
- Themes rated as very high, high, or medium sensitivity necessitate specialist impact assessments as per NEMA protocols, while those with low sensitivity require statements by specialists.
- The screening tool results facilitated a high-level comparison of schemes to assess environmental implications.
- Listed activities associated with the project, as described in Government Notices 983 and 985 of 4 December 2014 (as amended), indicated the requirement for a Basic Assessment process for environmental authorisation for all four schemes.

Specific findings from the screening exercise include:

- Schemes 2 and 3 exhibit six themes with a rating of high and very high sensitivity,
- Schemes 1 and 4 display seven themes with a rating of high and very high sensitivity,
- Scheme 1, which was previously authorised, had no fatal flaws identified in its environmental assessment, and,
- Schemes 4B and 1B largely follow the same alignment as Scheme 1, suggesting similar environmental considerations.

In summary, all schemes are expected to demonstrate similar environmental sensitivities and requirements based on the screening exercise's outcomes, i.e. the schemes are considered comparable based on environmental considerations. Further details on the environmental screening are available in the Environmental Scoping Report (Report No. P WMA 06/D00/00/3423/13).

12 Conclusions and Recommendations

The existing GBWSS has experienced water restrictions since 2014 due to the inability of the existing infrastructure to supply the growth in water demand. Various studies have been undertaken by VCWB, MMM and DWS to identify options to augment the supply to the GBWSS.

The 2012 Reconciliation Strategy identified the following major interventions:

- Implementation of water conservation and water demand management,
- Increase capacity of Tienfontein pump station,
- Implementation of the Welbedacht / Knellpoort bi-directional pipeline, and,
- Implementation of re-use of treated effluent.

Other recommendations from the 2012 Reconciliation Strategy include:

- Addressing the siltation problems at Welbedacht WTW to increase the operating capacity of the plant,
- Improving the integrity of the Welbedacht pipeline, and
- Increasing the capacity of the Maselspoort WTW and raise Mockes Dam.

The above interventions and recommendations were considered the most economical options that can be implemented in the shortest possible timeframes. The 2012 Reconciliation Strategy also identified the transfer to water from Gariep Dam as the next augmentation scheme to be considered after implementation of the above interventions.

MMM and VCWB both investigated the transfer of water from Gariep Dam and came to different conclusions on the preferred solution, i.e.:

- MMM concluded that a direct pipeline from Gariep Dam to Bloemfontein, conveying potable water, will be the optimal solution, and
- VCWB concluded that a pipeline from Gariep Dam to Knellpoort Dam, conveying raw water, will be the optimal solution.

As a result, DWS decided to initiate this study, referred to as the "Greater Mangaung Water Augmentation Project – Xhariep Pipeline Feasibility Study". The purpose of this study is to appraise, at a pre-feasibility level of detail, the most viable previously identified development options (routes) for the Xhariep Pipeline Project and to recommend the optimal system size (including phasing) and the best water conveyance route from a regional and national perspective that should be taken forward to the feasibility stages of study.

The three most feasible pipeline route options identified from previous studies were:

- Scheme 1: Direct potable pipeline from Gariep Dam to Bloemfontein,
- Scheme 2: Raw water pipeline from Gariep Dam to Knellpoort Dam, and
- Scheme 3: Raw water pipeline from Gariep Dam to the Novo Outfall Structures.

A fourth scheme, referred to as Scheme 4, was identified at the commencement of this study. Scheme 4 is a raw water pipeline from Gariep Dam to Rustfontein Dam, which aims to reduce the losses associated with Scheme 3 where water will be conveyed along the upper reaches of the Modder River before being discharged into Rustfontein Dam. The pipeline routes for the four schemes are shown in Figure 1-2.

Feasibility studies are iterative in nature as interventions/schemes are required such that the water resources yield matches the forecasted water demands, followed by infrastructure option identification, refinement of the yield modelling, refinement of the infrastructure sizing, etc. This iterative process, and the steps followed for this study, is shown in Figure 1-3 (repeated below as Figure 12-1 for ease of reference).

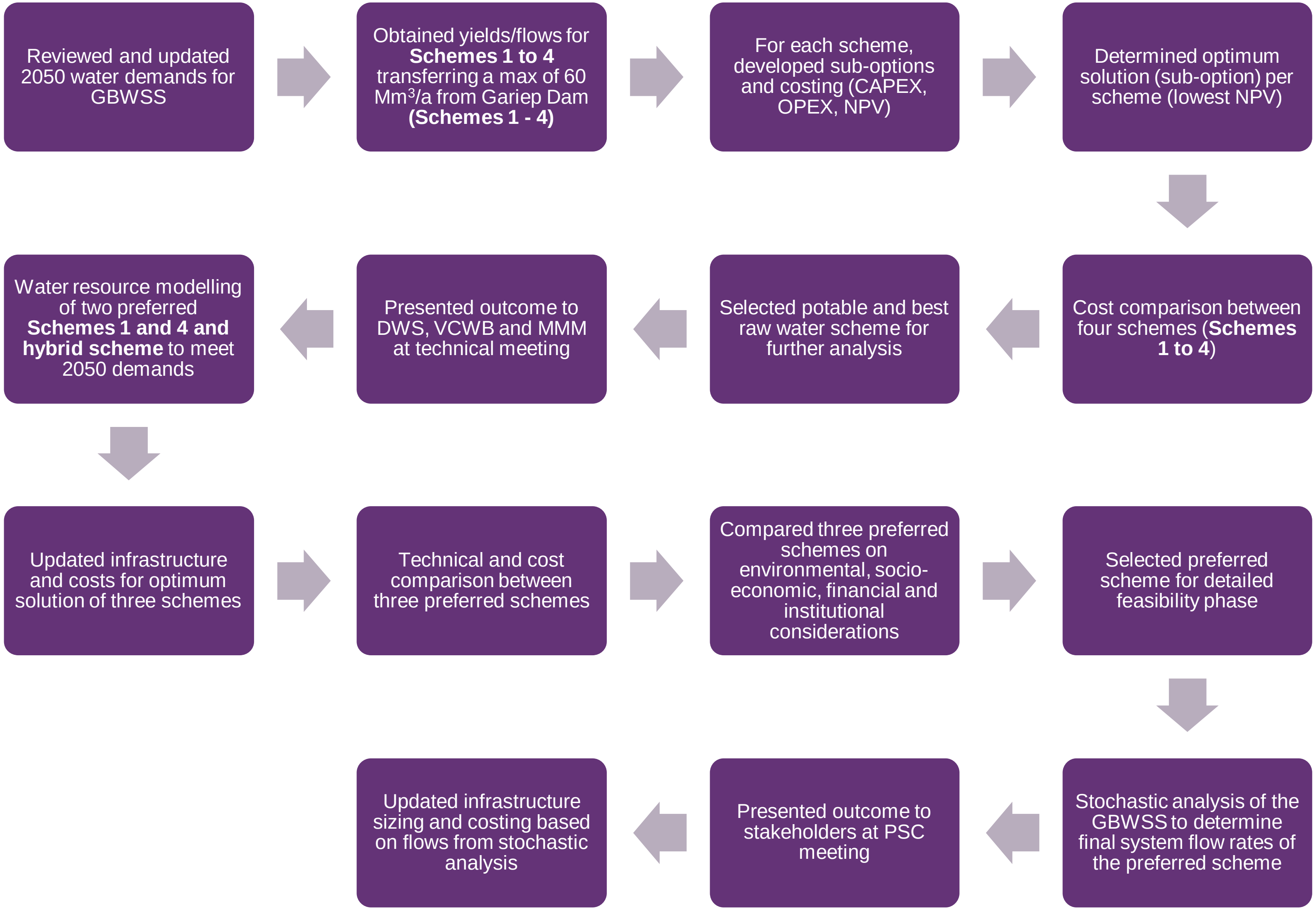


Figure 12-1: Flow chart of scheme development process

Previous studies included water demand projections to 2035 or 2040, whereas the planning horizon adopted for this project was 2050. The previous studies recommended annual growth rates in water demands that varied from 1% to 3%. This study evaluated the water demands from two different approaches, i.e. (1) based on historic water demands and assuming a similar growth rate over time, and (2) based on published population data by Stats SA and accounting for an improvement in level of service so that all households will have a house connection by 2050. The actual water demands in 2014 were used as the starting point for the demand projections. Table 12-1 provides a comparison of the water demands calculated in the previous studies against the demands calculated in this study (i.e. 'Observed Projected' is based on historical growth in demand, and 'Scenario 2' is based on population data and improvement in level of service).

Table 12-1: Total comparison results

Source	2023		2035		2050		Average % Increase
	mil m^3/a	Mℓ/d	mil m^3/a	Mℓ/d	mil m^3/a	Mℓ/d	
Observed Projected (this study)	118.21	323.86	145.55	398.78	179.74	492.42	1.56%
Scenario 2 (this study)	110.96	303.99	140.17	384.02	186.40	510.70	1.94%
2012 Reconciliation Strategy	133.92	366.91	186.78	511.74	286.49	784.89	2.86%
2015 Technical Feasibility Study	115.79	317.23	146.85	402.33	197.64	541.48	2.00%
2018 Mangaung Study	114.71	314.26	139.35	381.78	179.86	492.76	1.68%
2022/23 AOA	108.48	297.20	139.25	381.51	185.16	507.29	2.00%

It is evident from Table 12-1 that the 2050 demand projections calculated as part of this project came to 180 million m^3/a and 186 million m^3/a, respectively. This compared favourably with the demand projections of 180 million m^3/a and 185 million m^3/a determined as part of the 2018 Mangaung Study and the 2022/23 Annual Operating Analysis. The demand determined as part of Scenario 2, i.e. 186 million m^3/a, was adopted as the 2050 water demand that had to be satisfied for the GBWSS. Scenario 2 also included the demands of the towns and villages located within 10 km from the proposed pipeline routes, should these towns wish to connect to the proposed Xhariep Pipeline.

The ToR for this study recommended that the first phase of the water resources yield modelling be based on transferring a maximum volume of 60 million m^3/a from Gariep Dam. Table 12-2 summarises the HFY determined for each of the four schemes as well as the percentage of the 2050 demands that can be met by each scheme for the major demand centres.

Table 12-2: System Historic Firm Yield based on 60 million m^3/a transfer from Gariep Dam

Scheme	Historic Firm Yield (million m^3/a)	Percentage of 2050 demands met	
		Bloemfontein (%)	Botshabelo & Thaba Nchu (%)
1 (potable water to Bloemfontein)	131	59.1	84.3
2 (raw water to Knellpoort Dam)	119	44.3	92.6
3 (raw water to Novo Outfall Structure)	120	43.1	96.2
4 (raw water to Rustfontein Dam)	134	55.2	97.1

It is evident from Table 12-2 that (a) the HFY differs from scheme to scheme, (b) the HFY was considerably lower than the 2050 demand of 186 million m^3/a, and (c) a higher volume would need to be transferred from Gariep Dam to satisfy the 2050 demand.

The infrastructure required for each of the four schemes, based on a maximum transfer volume of 60 million m^3/a from Gariep Dam, was determined and costed to undertake a comparison of the schemes. Multiple sub-options were developed for each scheme where different pipeline diameters and pump stations positions were evaluated. The purpose of these sub-options was to optimise the infrastructure for each scheme.

The infrastructure for each scheme and sub-option was sized based on the peak flows derived from the water resources yield modelling, whereas the operating and maintenance costs were calculated from the average annual flows determined by the yield modelling. The operating and maintenance costs were converted to a NPV using a discount rate of 6% and a discount period of 45 years. Given the different HFY of the four schemes, the yields were also converted to NPVs, which allowed URVs to be calculated. The URV for each sub-option was the total NPV of the costs divided by the NPV of the water demands. Table 9-6 (repeated below as Table 12-3 for ease of reference) shows the NPV and URV calculated for the preferred sub-options of Schemes 1, 2, 3 and 4. It is evident from Table 12-3 that Scheme 4 was the most economical raw water scheme (compared to Schemes 2 and 3) and that Scheme 1 was 7.5% more expensive than Scheme 4.

Based on the NPVs and URVs shown in Table 12-3, it was decided to undertake the additional water resources yield modelling for Schemes 1 (potable option) and 4 (most economical raw water option). A stakeholder engagement with DWS, MMM and VCWB took place on 2 November 2023 where feedback was provided on progress to date and where operational matters could be discussed. The following specific matters were raised at the meeting:

- Botshabelo and Thaba Nchu were experiencing higher levels of restriction compared to other towns within the GBWSS, mainly as these two towns only have Rustfontein WTW as supply whereas Bloemfontein can receive water from Welbedacht, Rustfontein and Maselspoort WTWs,
- VCWB preferred Scheme 2 (raw water supply to Knellpoort Dam) due to greater operational flexibility, e.g. raw water can be supplied from Knellpoort Dam to Welbedacht Dam as well as to Rustfontein and Maselspoort WTWs,
- The supply of potable water to towns located along the proposed pipeline route remains a priority from a regional water supply perspective,
- Scheme 1 is the only potable scheme under consideration but can only supply Bloemfontein and the towns along the pipeline route, i.e. it would not resolve the challenges experienced at Botshabelo and Thaba Nchu, and
- All parties agreed that Scheme 1 and Scheme 4 had limitations in terms of overall flexibility and improving the resilience of the GBWSS.

This led to the development of Scheme 1B (also referred to as the “hybrid” scheme since the pipeline route is a combination of the routes for Schemes 1 and 4) as shown in Figure 7-6 and Figure 7-7 (repeated below as Figure 12-2 for ease of reference) where potable water would be supplied from Gariep Dam to a command reservoir located between Bloemfontein and Rustfontein WTW. Water from the command reservoir could then gravitate to Bloemfontein and Rustfontein WTW.

The water resources yield modelling was updated for Schemes 1 and 4 (now referred to Schemes 1A and 4B to distinguish them from Schemes 1 and 4), as well as Scheme 1B, to determine the transfer volume required from Gariep Dam that would satisfy the 2050 demands. Table 12-4 summarises the maximum annual transfer volumes required from Gariep Dam and the percentage of demand that could be supplied for each of the large demand centres.

Table 12-3: Results of financial comparison for original schemes 1 to 4 with maximum annual transfer of 60 Mm³/a

<table>
<tr><th>Scheme Number</th><th>Pipeline length</th><th>Pipe diameter</th><th>HLPS Average duty</th><th>HLPS Maximum duty</th><th>Booster PS Avg duty</th><th>Booster PS Max duty</th><th>WTW upgrades</th><th>Total volume treated (2050 demands)</th><th>Volume pumped (2050 demands)</th><th>HFY for URV (6% over 45 yrs)</th><th>Total Capital Cost</th><th>Net Present Value of O&M</th><th>Total Net Present Value</th><th>O&M URV</th><th>Total URV</th><th>Comparison to lowest option cost</th></tr>
<tr><th>Scheme comparison of original transfer at 60 Mm³ / annum</th><th>km</th><th>mm</th><th>ℓ/s | m</th><th>ℓ/s | m</th><th>ℓ/s | m</th><th>ℓ/s | m</th><th>Mℓ</th><th>Mm³/annum</th><th>Mm³/annum</th><th>Mm³</th><th>Million Rands</th><th>(Million Rands)</th><th>(Million Rands)</th><th>R/m³</th><th>R/m³</th><th>%</th></tr>
<tr><td>Scheme 1 [Sc5b] Potable water from Gariep Dam to Bloemfontein</td><td>181.2</td><td>DN1600</td><td>1411 | 348</td><td>2148 | 367</td><td>-</td><td>-</td><td>165</td><td>133.2</td><td>44.5</td><td>1986</td><td>11895</td><td>12543</td><td>24438</td><td>6.32</td><td>12.31</td><td>-</td></tr>
<tr><td>+ Rustfontein pumping operation cost</td><td>~29 km</td><td>Assumed DN1100</td><td>1434 | 199</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>45.2</td><td>1986</td><td>0</td><td>1384</td><td>1384</td><td>0.70</td><td>0.70</td><td>-</td></tr>
<tr><td>+ Welbedacht pumping operation cost</td><td>~5.2 km to reservoir</td><td>DN1170</td><td>1293 | 247</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>40.8</td><td>1986</td><td>0</td><td>1549</td><td>1549</td><td>0.78</td><td>0.78</td><td>-</td></tr>
<tr><td>+ Novo transfer pumping operation cost</td><td>~14.4 km to reservoir</td><td>DN1200</td><td>734 | 136</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>23.2</td><td>1986</td><td>0</td><td>485</td><td>485</td><td>0.24</td><td>0.24</td><td>-</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Total</td><td>11895</td><td>15962</td><td>27857</td><td>8.04</td><td>14.03</td><td>+7.5</td></tr>
<tr><td>Scheme 2 [Sc4b(ii)] Raw water from Gariep Dam to Knellpoort Dam</td><td>190.4</td><td>DN1300</td><td>1035 | 298</td><td>1901 | 418</td><td>1035 | 132</td><td>1901 | 207</td><td>64</td><td>115.3</td><td>32.6</td><td>1797</td><td>9948</td><td>11648</td><td>21597</td><td>6.48</td><td>12.02</td><td>-</td></tr>
<tr><td>+ Maselspoort pipeline and PS upgrades</td><td>33.5</td><td>DN800</td><td>257 | 135</td><td>570 | 210</td><td>-</td><td>-</td><td>Incl above</td><td>Incl above</td><td>8.1</td><td>1797</td><td>498</td><td>423</td><td>921</td><td>0.24</td><td>0.51</td><td>-</td></tr>
<tr><td>+ Rustfontein pumping operation cost</td><td>~29 km</td><td>Assumed DN1100</td><td>1297 | 194</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>40.9</td><td>1797</td><td>0</td><td>1224</td><td>1224</td><td>0.68</td><td>0.68</td><td>-</td></tr>
<tr><td>+ Welbedacht pumping operation cost</td><td>~5.2 km to reservoir</td><td>DN1170</td><td>963 | 244</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>30.4</td><td>1797</td><td>0</td><td>1142</td><td>1142</td><td>0.64</td><td>0.64</td><td>-</td></tr>
<tr><td>+ Novo transfer pumping operation cost</td><td>~14.4 km to reservoir</td><td>DN1200</td><td>2438 | 185</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>76.9</td><td>1797</td><td>0</td><td>2194</td><td>2194</td><td>1.22</td><td>1.22</td><td>-</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Total</td><td>10446</td><td>16632</td><td>27078</td><td>9</td><td>15.07</td><td>+15.4</td></tr>
<tr><td>Scheme 3 [Sc4c(i)] Raw water from Gariep Dam to Novo outfall</td><td>197.8</td><td>DN1300 and DN1400</td><td>1645 | 361</td><td>1901 | 377</td><td>1645 | 231</td><td>1901 | 248</td><td>64</td><td>116.5</td><td>51.9</td><td>1808</td><td>8918</td><td>13811</td><td>22729</td><td>7.64</td><td>12.57</td><td>-</td></tr>
<tr><td>+ Maselspoort upgrades</td><td>33.5</td><td>DN800</td><td>235 | 133</td><td>570 | 210</td><td>-</td><td>-</td><td>Incl above</td><td>Incl above</td><td>7.4</td><td>1808</td><td>498</td><td>405</td><td>903</td><td>0.22</td><td>0.50</td><td>-</td></tr>
<tr><td>+ Rustfontein pumping operation cost</td><td>~29 km</td><td>Assumed DN1100</td><td>1373 | 197</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>43.3</td><td>1808</td><td>0</td><td>1312</td><td>1312</td><td>0.73</td><td>0.73</td><td>-</td></tr>
<tr><td>+ Welbedacht pumping operation cost</td><td>~5.2 km to reservoir</td><td>DN1170</td><td>985 | 244</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>31.1</td><td>1808</td><td>0</td><td>1169</td><td>1169</td><td>0.65</td><td>0.65</td><td>-</td></tr>
<tr><td>+ Novo transfer pumping operation cost</td><td>~14.4 km to reservoir</td><td>DN1200</td><td>1054 | 141</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>33.2</td><td>1808</td><td>0</td><td>723</td><td>723</td><td>0.40</td><td>0.40</td><td>-</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Total</td><td>9416</td><td>17421</td><td>26836</td><td>10</td><td>14.85</td><td>+13.7</td></tr>
<tr><td>Scheme 4 [Sc5f] Raw water from Gariep Dam to Rustfontein Dam</td><td>203.9</td><td>DN1300</td><td>1431 | 378</td><td>1901 | 412</td><td>1431 | 78</td><td>1901 | 104</td><td>64</td><td>130.3</td><td>45.1</td><td>1955</td><td>8473</td><td>12718</td><td>21191</td><td>6.51</td><td>10.84</td><td>-</td></tr>
<tr><td>+ Maselspoort upgrades</td><td>33.5</td><td>DN800</td><td>299 | 140</td><td>570 | 210</td><td>-</td><td>-</td><td>Incl above</td><td>Incl above</td><td>9.4</td><td>1955</td><td>498</td><td>459</td><td>957</td><td>0.24</td><td>0.49</td><td>-</td></tr>
<tr><td>+ Rustfontein pumping operation cost</td><td>~29 km</td><td>Assumed DN1100</td><td>1390 | 197</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>43.8</td><td>1955</td><td>0</td><td>1332</td><td>1332</td><td>0.68</td><td>0.68</td><td>-</td></tr>
<tr><td>+ Welbedacht pumping operation cost</td><td>~5.2 km to reservoir</td><td>DN1170</td><td>1219 | 246</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>38.4</td><td>1955</td><td>0</td><td>1457</td><td>1457</td><td>0.75</td><td>0.75</td><td>-</td></tr>
<tr><td>+ Novo transfer pumping operation cost</td><td>~14.4 km to reservoir</td><td>DN1200</td><td>863 | 138</td><td>-</td><td>-</td><td>-</td><td>-</td><td>-</td><td>27.2</td><td>1955</td><td>0</td><td>578</td><td>578</td><td>0.30</td><td>0.30</td><td>-</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Total</td><td>8971</td><td>16544</td><td>25515</td><td>8</td><td>13.05</td><td>100</td></tr>
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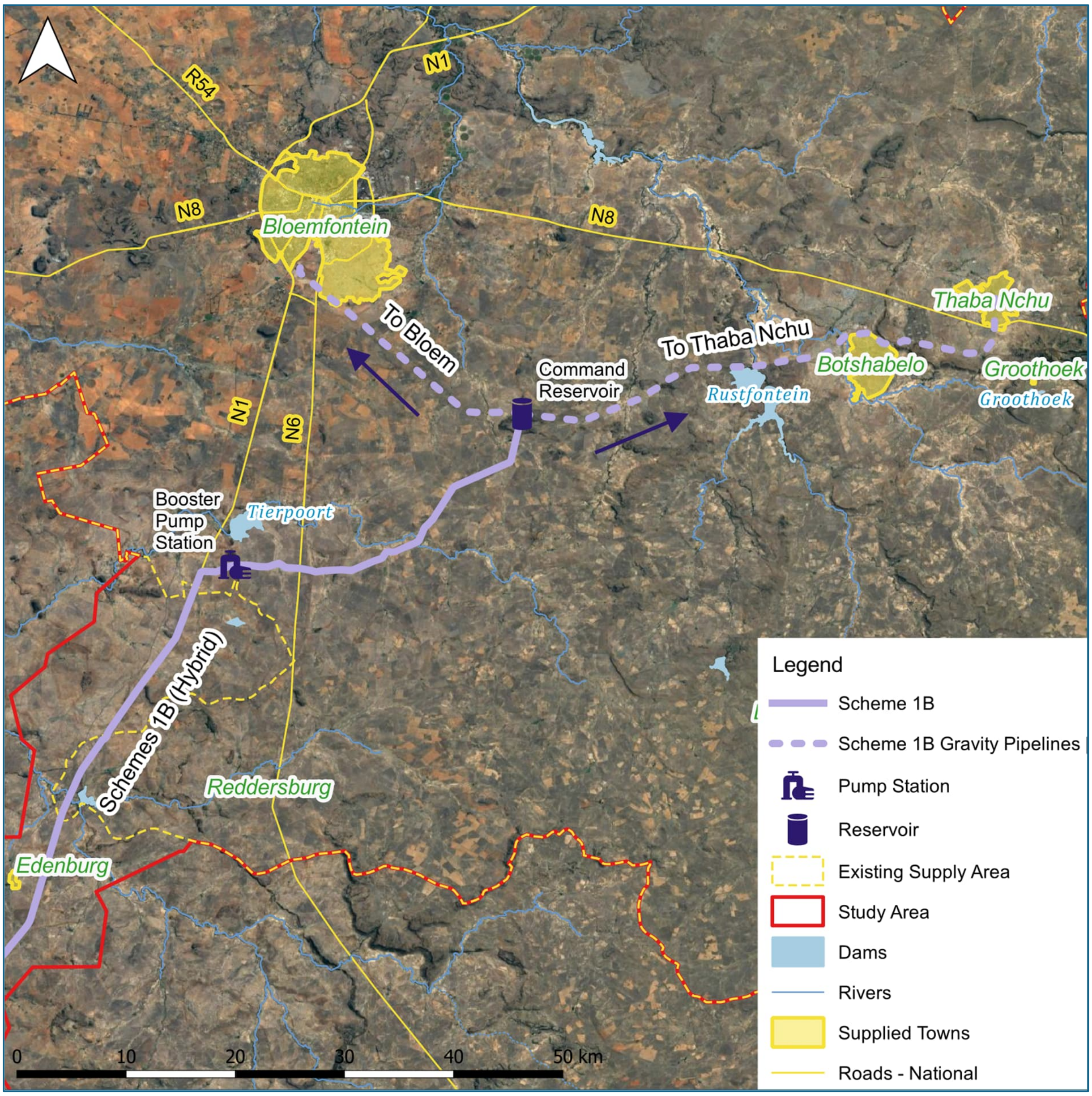

Figure 12-2: Scheme 1B supply to Bloemfontein and Rustfontein WTW

Table 12-4: Historic Firm Yield required from Gariep Dam to satisfy 2050 demand

Scheme	Historic Firm Yield (million m^3/a)	Maximum transfer volume (million m^3/a)	Percentage of 2050 demands met	
			Bloemfontein (%)	Botshabelo & Thaba Nchu (%)
1A (potable to Bloemfontein)	186	120	100.0	84.4
1B (potable to regional command reservoir)	186	120	100.0	99.6
4B (raw water to Rustfontein Dam)	186	142	100.0	100.0

It is evident from Table 12-4 that Schemes 1A and 4B would not be able to supply 100% of the demands for Botshabelo and Thaba Nchu, mainly due to bottlenecks in the existing GBWSS infrastructure. The infrastructure sizing and cost estimates were updated for Schemes 1A, 1B and 4B with the calculated NPV and URV information shown in Table 12-5.

Table 12-5: Results of financial comparison for additional schemes 1A, 4B and 1B (Hybrid) with maximum annual transfer of 142 Mm³/a

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Present Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme comparison of increased transfer at 142 Mm³ / annum	**km**	**mm**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**ℓ/s | m**	**Mℓ**	**Mm³/annum**	**Mm³/annum**	**Mm³**	**(Million Rands)**	**(Million Rands)**	**(Million Rands)**	**R/m³**	**R/m³**	**%**
Scheme 1A [Sc5b] Potable water from Gariep Dam to Bloemfontein	181.2	2 x DN1500	2925 | 356	5085 | 399	2925 | 89	5085 | 119	390	185.9	92.2	2208	25120	21983	47103	9.96	21.33	-
+ Rustfontein pumping operation cost	~29 km	Assumed DN1100	941 | 185	-	-	-	-	-	29.7	2208	0	844	844	0.38	0.38	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	726 | 243	-	-	-	-	-	22.9	2208	0	856	856	0.39	0.39	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	1926 | 165	-	-	-	-	-	60.7	2208	0	1544	1544	0.70	0.70	-
										Total	**25120**	**25226**	**50347**	**11.42**	**22.80**	**+3.1**
Scheme 4B [Sc5f] Raw water from Gariep Dam to Rustfontein Dam	203.9	2 x DN1400 & DN1600	3016 | 367	4500 | 408	-	-	189	182.2	95.1	2208	21317	20941	42258	9.48	19.14	-
+ Maselspoort upgrades	33.5	DN800	706 | 217	570 | 210	-	-	Incl above	Incl above	22.3	2208	505	1034	1539	0.47	0.70	-
+ New pipeline from Rustfontein to Bloemfontein	50.2	DN1000	63 | 98	920 | 242	-	-	-	-	-	2208	914	199	1112	0.09	0.50	-
+ Rustfontein pump upgrades + operating cost (to Bloemfontein)	Varies	Equivalent DN1400	63 | 98	1440 | 156	-	-	-	-	2.0	2208	119	265	384	0.12	0.17	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1529 | 249	-	-	-	-	-	48.2	2208	0	1849	1849	0.84	0.84	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	475 | 133	-	-	-	-	-	15.0	2208	0	307	307	0.14	0.14	-
										Total	**22855**	**25992**	**48847**	**12**	**22.12**	**100**
Scheme 1B [Sc5b] Potable water from Gariep Dam to Rustfontein	186.1	2 x DN1400	2776 | 362	4294 | 401	2776 | 210	4294 | 233	330	182.2	87.5	2208	21846	24675	46521	11.17	21.07	-
+ Gravity pipeline to Longridge reservoir from command reservoir	26.0	DN1200	-	-	-	-	-	-	69.2	2208	663	144	808	0.07	0.37	-
+ Gravity pipeline to Rustfontein from command reservoir	25.7	DN1100	-	-	-	-	-	-	50.7	2208	590	128	718	0.06	0.33	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	-
										Total	**23099**	**27125**	**50224**	**12.28**	**22.75**	**+2.8**

It is evident from Table 12-5 that the URVs of Schemes 1A, 1B and 4B were within 3% of each other and therefore considered comparable from a financial perspective. Given that Scheme 1A can only supply 84.4% of the demands to Botshabelo and Thaba Nchu, and that Scheme 4B is a raw water scheme that cannot supply the towns along the pipeline route, it was proposed that Scheme 1B be considered for implementation.

The water resources yield modelling was further optimised for Scheme 1B by testing different operating rules and maximising the utilisation of existing and proposed infrastructure. This optimisation process resulted in reducing the maximum transfer volume from 120 million m^3/a to 101 million m^3/a.

A stochastic analysis was subsequently undertaken for Scheme 1B to confirm that the 2050 demand can be delivered at a minimum of 98% assurance of supply (i.e. a 1:50 year recurrence interval). Figure 12-4 shows the outcome of the stochastic analysis, which indicates that yields of approximately 220 million m^3/a and 213 million m^3/a can be delivered at 98% and 99% assurance of supply, respectively. It is recommended that the maximum transfer volume from Gariep Dam remains at 101 million m^3/a as the higher assurance of supply provides flexibility should additional towns be included in future as part of the GBWSS or to cater for any unforeseen delays experienced with the implementation of any of the 2012 Reconciliation Strategy interventions.

Upon completion of the stochastic analysis and based on a maximum transfer volume of 101 million m^3/a, three alternative configurations for Scheme 1B were considered as part of the design optimisation process (refer to Figure 7-10). These alternative configurations mainly evaluated different locations for the booster pump station, different elevations for the second command reservoir, and connecting pipeline sizes between the second command reservoir and Bloemfontein as well as between the command reservoir and Rustfontein WTW. The NPVs and URVs for the three configurations were determined and are shown in Table 12-6. The URVs of the three configurations differ by less than 2%, meaning that the configurations are comparable from a financial perspective.

A site visit was undertaken in January 2024 to evaluate the various infrastructure sites in terms of topography, impact on farming activities, location relative to existing access roads and powerlines, as well as any other observations that could impact the feasibility of the sites. Operational aspects were also considered, e.g. preference will be given to configurations where demands can be met under gravity flow, rather than flow being pumped. Based on the findings of the site visit and accounting for operational considerations, it is recommended that the detailed feasibility design proceed based on Configuration 1B1(A) as shown in Figure 12-3.

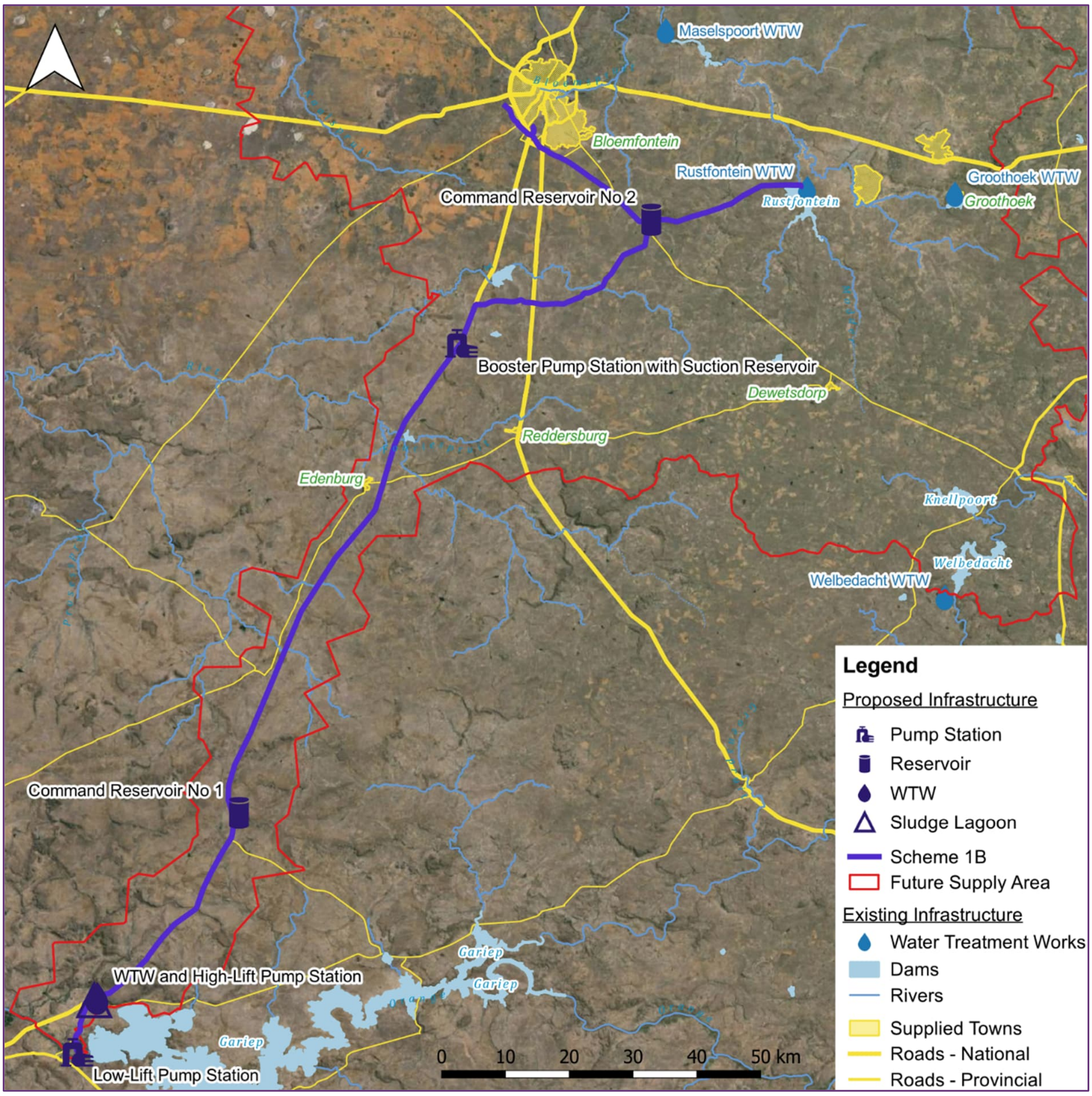

Figure 12-3: Configuration of Scheme 1B1(A)

The main infrastructure components for Configuration 1B1(A) are shown in Table 12-7.

A system risk failure analysis was undertaken for Schemes 1A, 1B and 4B that involved assessing the resilience of schemes by considering potential failures at supply sources or at a WTW. This evaluation entailed examining the system's ability to meet the 2050 demands of the major demand centres, Bloemfontein, Botshabelo and Thaba Nchu, in the event of a failure. The critical point of failure was found to be the supply from Rustfontein WTW, which if it fails, Schemes 1A and 4B result in the supply to Botshabelo and Thaba Nchu being reduced to 10% of the 2050 demands. Scheme 1B provides the most resilience and operational flexibility of the three schemes, as in the event of failure from any one of the four WTWs over 80% of the 2050 demands can still be supplied.

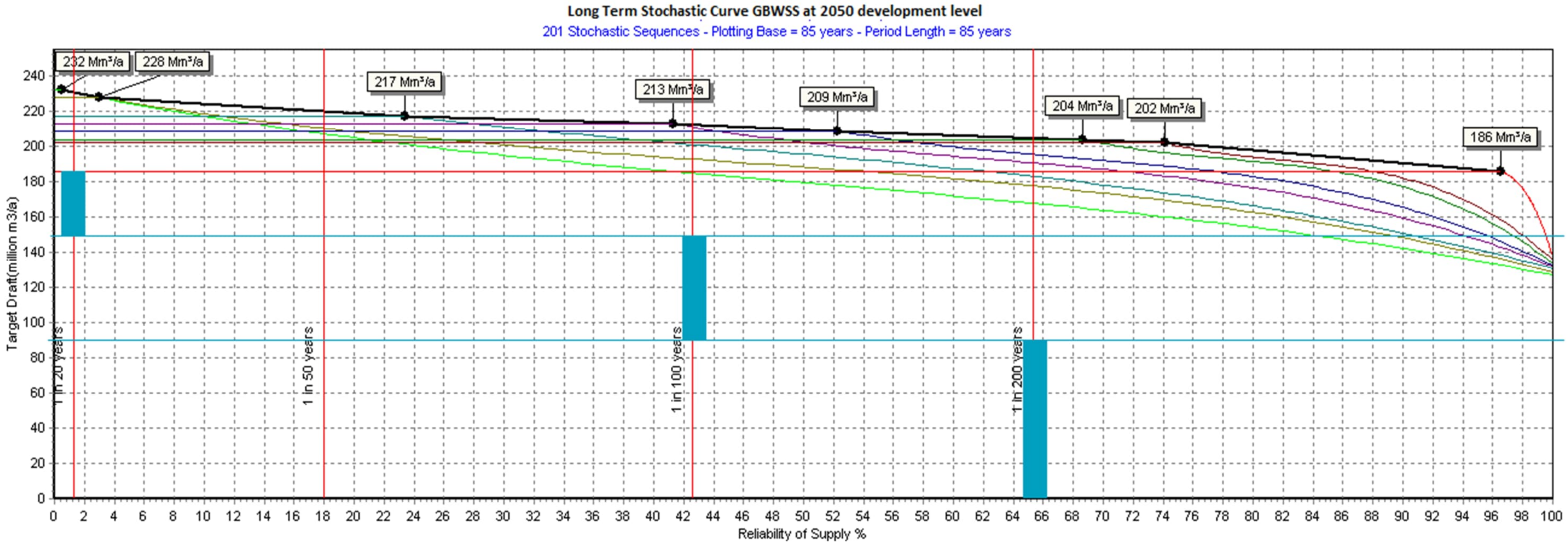

Figure 12-4: Stochastic yield analysis for Scheme 1B

Table 12-6: Results of financial comparison for optimization of scheme 1B (Hybrid)

Scheme Number	Pipeline length	Pipe diameter	HLPS Average duty	HLPS Maximum duty	Booster PS Avg duty	Booster PS Max duty	WTW upgrades	Total volume treated (2050 demands)	Volume pumped (2050 demands)	HFY for URV (6% over 45 yrs)	Total Capital Cost	Net Present Value of O&M	Total Net Present Value	O&M URV	Total URV	Comparison to lowest option cost
Scheme 1B optimization	km	mm	ℓ/s | m	ℓ/s | m	ℓ/s | m	ℓ/s | m	Mℓ	Mm³/annum	Mm³/annum	Mm³	Million Rands	(Million Rands)	(Million Rands)	R/m^3	R/m^3	%
Scheme 1B1A [Sc5b] Potable water from Gariep Dam to Rustfontein	186.9	2 x DN1400	2776 | 363	4294 | 403	2776 | 106	4294 | 138	330	182.2	87.5	2208	21493	22965	44458	10.40	20.13	-
+ Gravity pipeline to Brandkop reservoir from command reservoir	31.4	DN1600	-	-	-	-	-	-	69.2	2208	1100	239	1339	0.11	0.61	-
+ Gravity pipeline to Rustfontein from command reservoir	24.5	DN1400	-	-	-	-	-	-	68.9	2208	648	141	789	0.06	0.36	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	-
										Total	**23241**	**26920**	**50161**	**12.19**	**22.72**	**+1.2**
Scheme 1B1B [Sc5b] Potable water from Gariep Dam to Botshabelo	186.9	2 x DN1400	2776 | 363	4294 | 403	2776 | 106	4294 | 138	330	182.2	87.5	2208	21493	22965	44458	10.40	20.13	-
+ Gravity pipeline to Brandkop reservoir from command reservoir	31.4	DN1600	-	-	-	-	-	-	69.2	2208	1100	239	1339	0.11	0.61	-
+ Gravity pipeline to Botshabelo from command reservoir	30.3	DN2000	-	-	-	-	-	-	68.9	2208	1571	342	1913	0.15	0.87	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	579 | 180						18.3	2208	0	506	506	0.23	0.23	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	-
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	
										Total	**24164**	**26230**	**50394**	**11.88**	**22.82**	**+1.7**
Scheme 1B2 [Sc5b] Potable water from Gariep Dam to Rustfontein	180.2	2 x DN1400 & DN1600	2776 | 363	4294 | 403	-	-	330	182.2	87.5	2208	22081	20695	42776	9.37	19.37	-
+ Pumped pipeline to Brandkop reservoir from command reservoir	36.4	DN1500	1489 | 49	2193 | 85	-	-	-	-	69.2	2208	1171	777	1948	0.35	0.88	-
+ Gravity pipeline to Rustfontein from command reservoir	29.3	DN1600	-	-	-	-	-	-	68.9	2208	1025	223	1248	0.10	0.57	-
+ Rustfontein pumping operation cost (to Botshabelo)	~29 km	Assumed DN1100	1445 | 199	-	-	-	-	-	45.6	2208	0	1398	1398	0.63	0.63	-
+ Welbedacht pumping operation cost	~5.2 km to reservoir	DN1170	1493 | 249	-	-	-	-	-	47.1	2208	0	1803	1803	0.82	0.82	
+ Novo transfer pumping operation cost	~14.4 km to reservoir	DN1200	575 | 134	-	-	-	-	-	18.1	2208	0	375	375	0.17	0.17	
										Total	**24277**	**25271**	**49547**	**11**	**22.44**	**100**

Table 12-7: Summary of main infrastructure components of Scheme 1B1(A)

Infrastructure component	Capacity / Size	Length (km)
Low-lift pump station	3,797 m^3/s @ 102 m	-
Raw water pipeline	1800 mm	10.5 km
Water treatment works	312 Mℓ/d	-
High-lift pump station	3,616 m^3/s @ 325 m	-
1st command reservoir	80 Mℓ	-
Booster pump station	3,616 m^3/s @ 124 m	-
2nd command reservoir	80 Mℓ	-
Potable pipeline from high-lift pump station to 2nd command reservoir	1800 mm	176.4 km
Potable pipeline from 2nd command reservoir to Bloemfontein	2000 mm	31.4 km
Potable pipeline from 2nd command reservoir to Rustfontein WTW	1400 mm	24.5 km

The total capital cost of Scheme 1B1(A) is estimated at R 23,24 million (excluding VAT) at pre-feasibility level of detail.

A high-level comparison between Schemes 1A, 1B and 4B were undertaken based on the following considerations:

- Socio-economic,
- Financing arrangements,
- Institutional arrangements, and,
- Environmental impacts.

It was concluded from this high-level comparison that none of these considerations will dictate the scheme to be implemented. The decision on which scheme to implement must therefore be based on strategic, financial and operational considerations.

Based on the above conclusions, it is recommended that the detailed feasibility study proceed based on Scheme 1B1(A) for the following reasons:

- Financially it is comparable to all other schemes that were considered,
- It is the only scheme that can satisfy 100% of the 2050 demands to all the demand centres located within GBWSS, and,
- It is the scheme with the greatest operational flexibility and resilience, even when failures at any of the WTWs are experienced.

13 References

AWWA. (2000). *American Water Works Association Water Quality and Treatment A Handbook on Drinking Water.* McGraw-Hill Professional.

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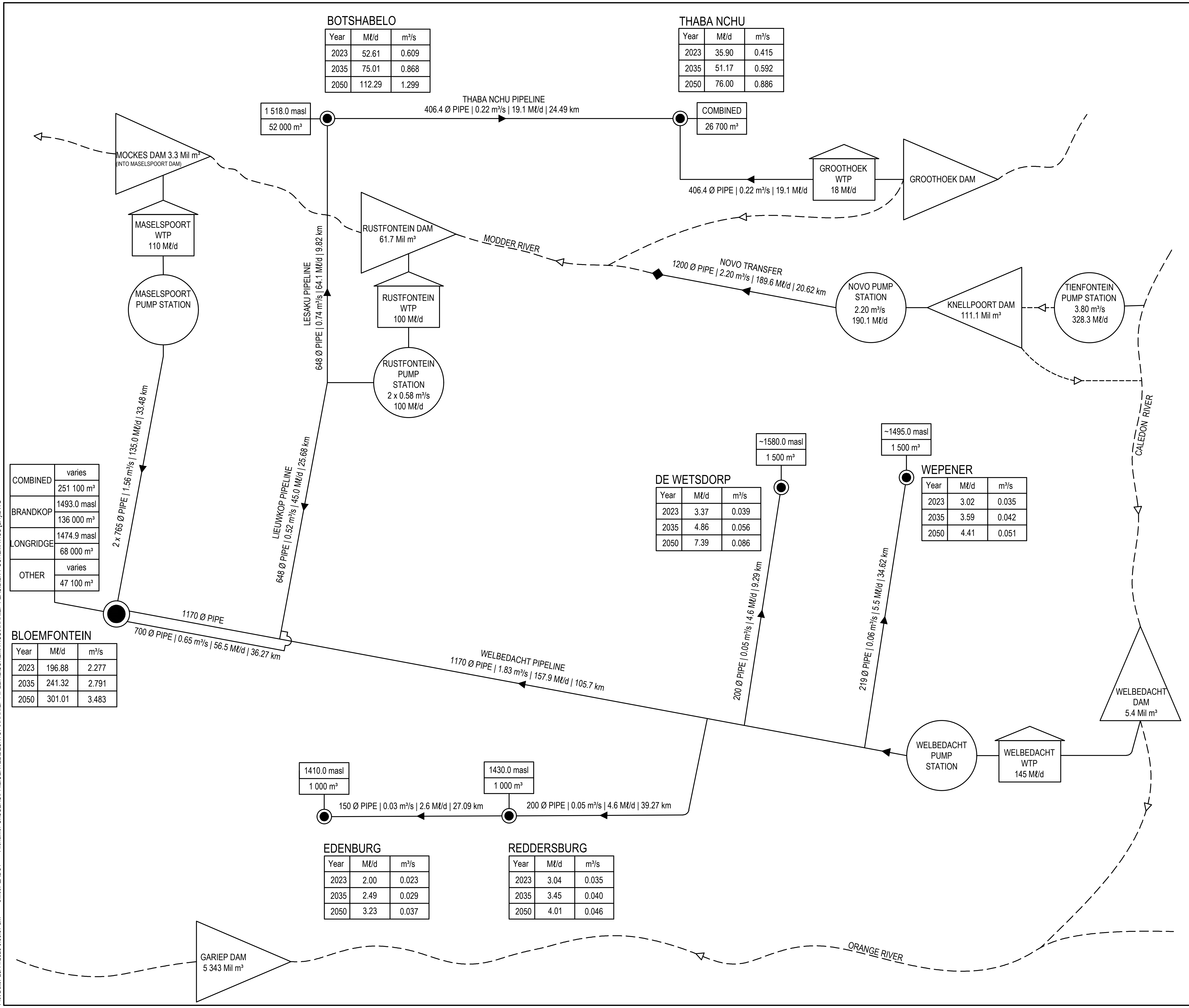
Appendix A

Book of Existing, Common Upgrades and Scheme Layouts



BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

COMBINED	varies	251 100 m³
BRANDKOP	1493.0 masl	136 000 m³
LONGRIDGE	1474.9 masl	68 000 m³
OTHER	varies	47 100 m³

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

DISCLAIMER

IF CONSTRUCTION DRAWINGS ARE ISSUED UNSIGNED, THE MASTER WITH THE ORIGINAL SIGNATURE OF APPROVAL WILL BE HELD AT THE ZUTARI OFFICE OF THE APPROVER

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. EXISTING PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.7 m/s EXCEPT WHERE VELOCITIES ARE KNOWN AND STATED BELOW.
2. NOVO TRANSFER VELOCITY OF 1.94 m/s.
3. LIEUWKOP PIPELINE VELOCITY OF 1.58 m/s (MAX).
4. LESAKU PIPELINE VELOCITY OF 2.25 m/s (MAX).
5. BLOEMFONTEIN INCLUDES DEMANDS OF DE BRUG.
6. BLOEMFONTEIN 'OTHER' RESERVOIR CAPACITY INCLUDES BLOEMDUSTRIA, BLOEMSPRUIT, DE BRUG AND BAINSVLEI RESERVOIRS.
7. THABA NCHU INCLUDES DEMANDS OF EXCELSIOR.
8. THABA NCHU COMBINED RESERVOIR CAPACITY INCLUDES TABALI, HOUTNEK, OK, MOTLATLA & BLYDSKAP RESERVOIRS.
9. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

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

REV	DATE	REVISION DETAILS	APPROVED
A	28/09/23	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: C.FEUZE
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT

XHARIEP PIPELINE FEASIBILITY STUDY

TITLE

SCHEMATIC OF EXISTING INFRASTRUCTURE

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
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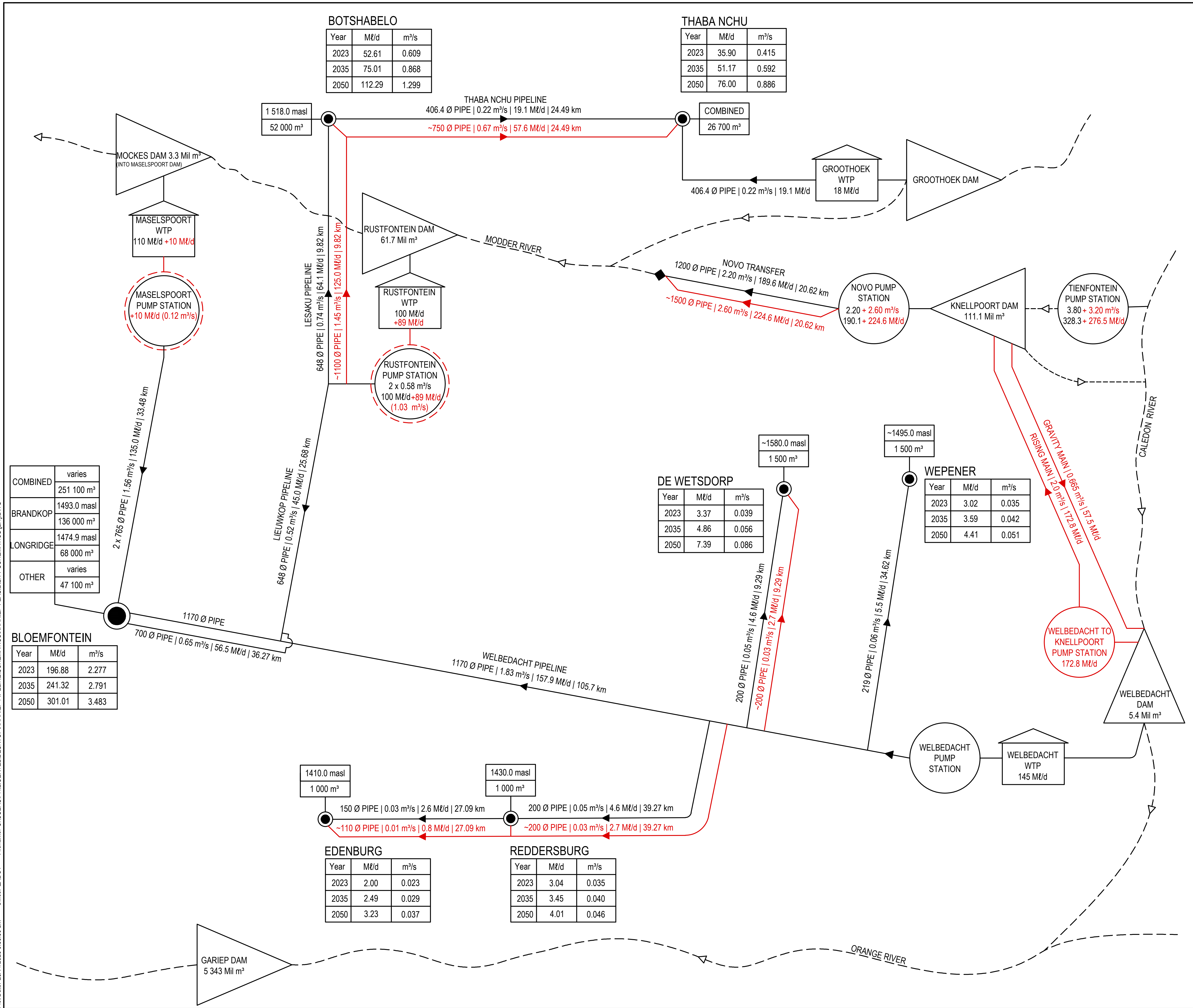

BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

COMBINED	varies	
	251 100 m³	
BRANDKOP	1493.0 masl	
	136 000 m³	
LONGRIDGE	1474.9 masl	
	68 000 m³	
OTHER	varies	
	47 100 m³	

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

DISCLAIMER

IF CONSTRUCTION DRAWINGS ARE ISSUED UNSIGNED, THE MASTER WITH THE ORIGINAL SIGNATURE OF APPROVAL WILL BE HELD AT THE ZUTARI OFFICE OF THE APPROVER

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. PROPOSED PIPELINE DIAMETERS BELOW DN500 ARE ASSUMED TO BE HDPE PE100 PN16 FOR CAPACITY CALCULATIONS.
3. WHERE ESTIMATED DIAMETERS EXCEED DN500 THE NOMINAL DIAMETER WAS USED FOR CAPACITY CALCULATIONS.
4. RUSTFONTEIN WTW TO DEDICATE FULL 100 Mℓ SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
5. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

ZUTARI
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CLIENT

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Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

REV	DATE	REVISION DETAILS	APPROVED
A	28/09/23	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: C.FEUZE
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT

XHARIEP PIPELINE FEASIBILITY STUDY

TITLE

SCHEMATIC OF COMMON UPGRADES TO ALL OPTIONS

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
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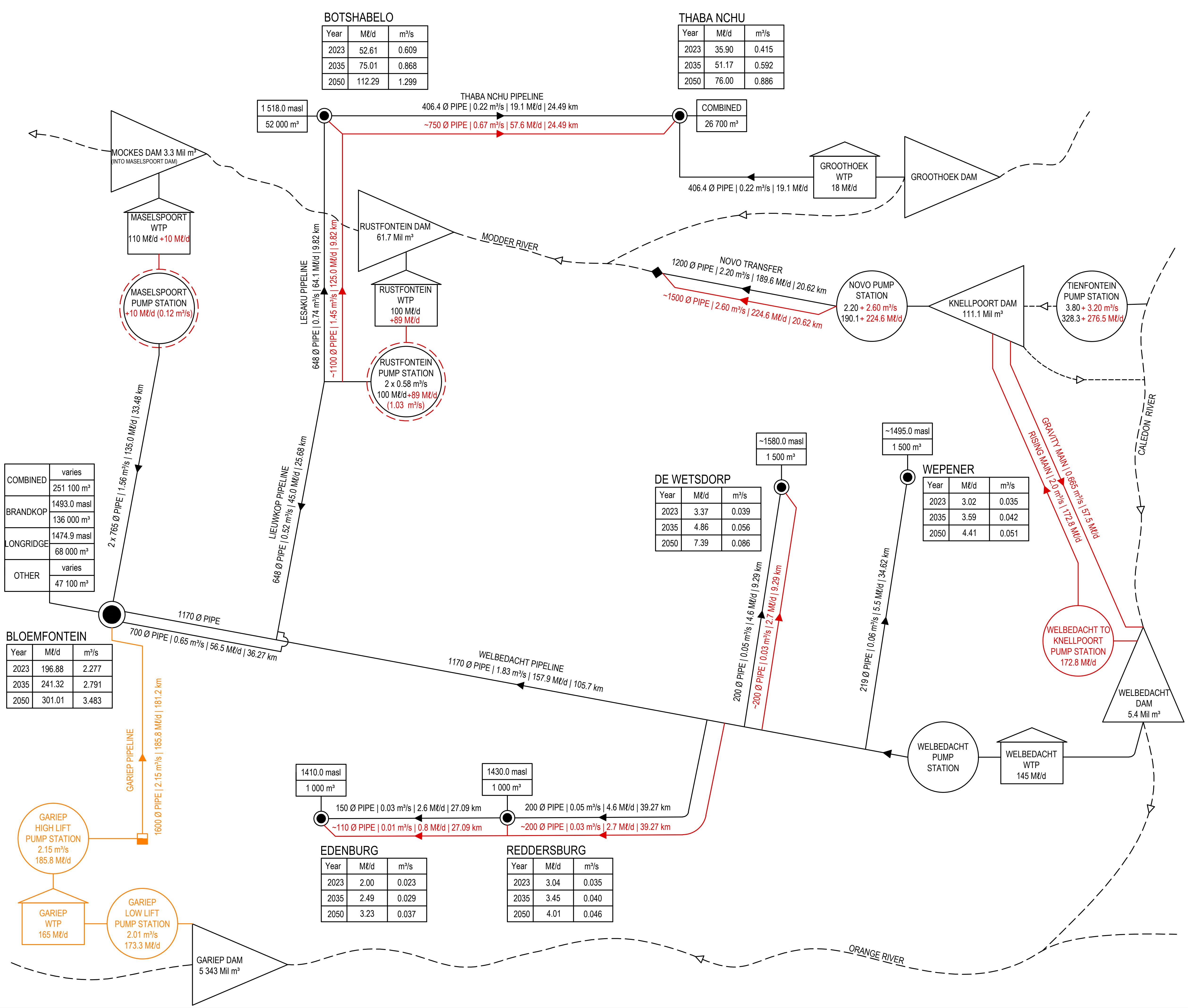

BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

COMBINED	varies	
	251 100 m³	
BRANDKOP	1493.0 masl	
	136 000 m³	
LONGRIDGE	1474.9 masl	
	68 000 m³	
OTHER	varies	
	47 100 m³	

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

DISCLAIMER

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY: LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO DEDICATE FULL SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1 (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION
RESERVOIR

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Department:
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REV	DATE	REVISION DETAILS	APPROVED
A	04/10/23	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS
DATE
ECSA 20010010

PROJECT

XHARIEP PIPELINE FEASIBILITY STUDY

TITLE

SCHEMATIC OF OPTION 1 [Sc5b]
POTABLE WATER TO BLOEMFONTEIN
(HISTORIC FIRM YIELD 137 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
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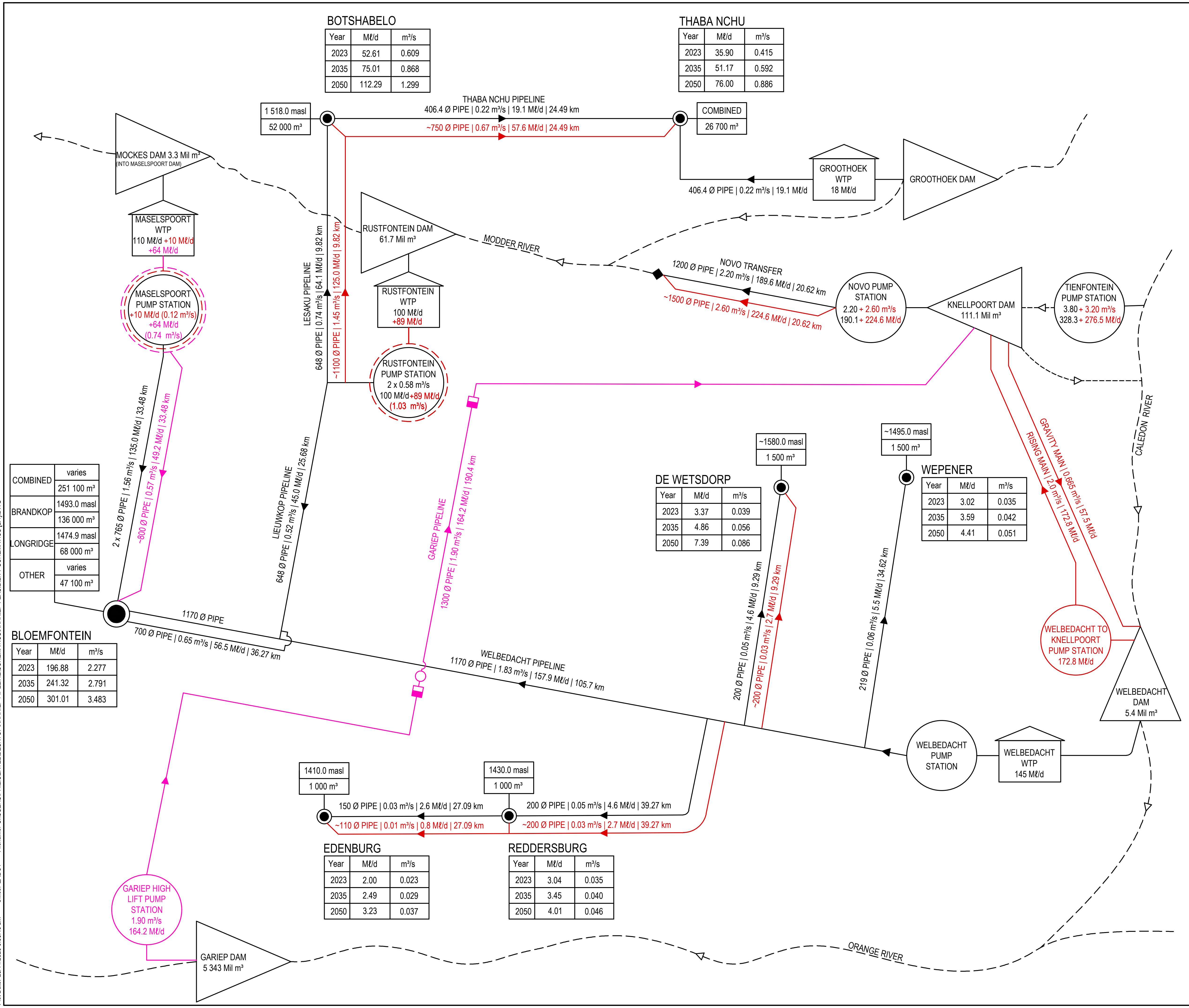

BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

DISCLAIMER

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LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO DEDICATE FULL SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1 (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION — RESERVOIR

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

REV	DATE	REVISION DETAILS	APPROVED
A	04/10/23	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT

XHARIEP PIPELINE FEASIBILITY STUDY

TITLE

SCHEMATIC OF OPTION 2 [Sc4b(ii)]
RAW WATER TO KNELLPOORT DAM
(HISTORIC FIRM YIELD 119 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
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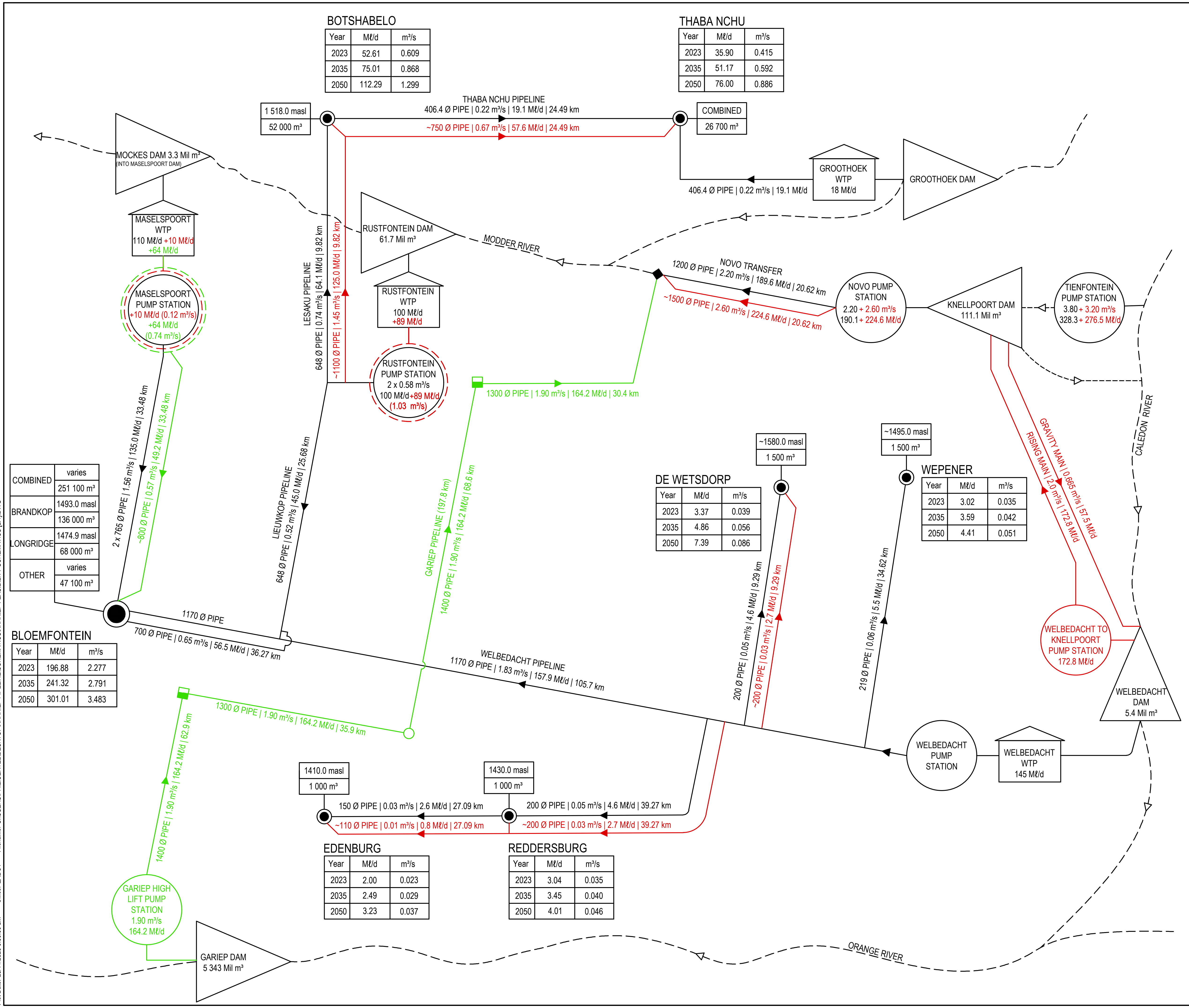

BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

COMBINED	varies	
	251 100 m³	
BRANDKOP	1493.0 masl	
	136 000 m³	
LONGRIDGE	1474.9 masl	
	68 000 m³	
OTHER	varies	
	47 100 m³	

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO DEDICATE FULL SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1 (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION
RESERVOIR

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

REV	DATE	REVISION DETAILS	APPROVED
A	04/10/23	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

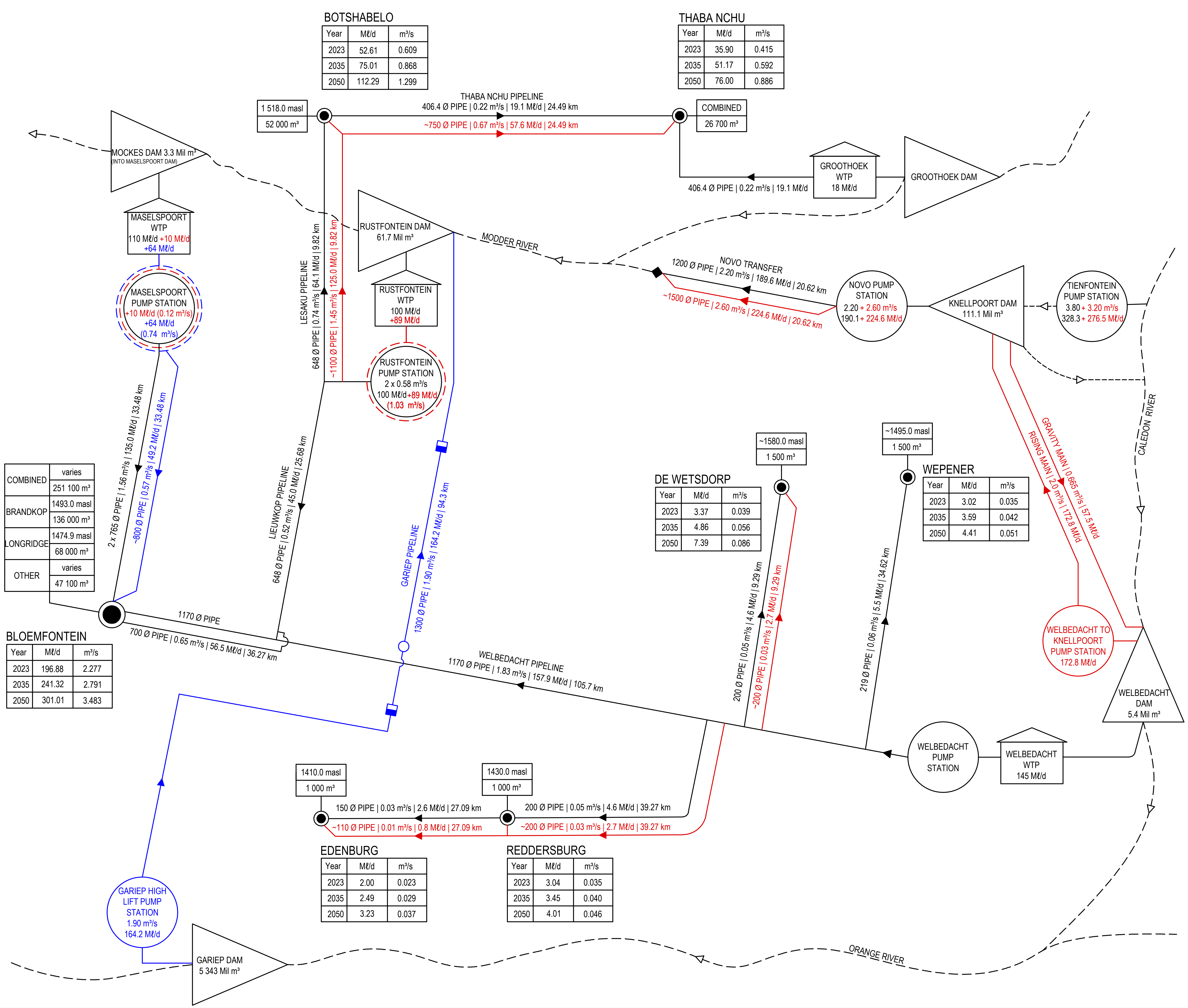
PROJECT
XHARIEP PIPELINE FEASIBILITY STUDY

TITLE
SCHEMATIC OF OPTION 3 [Sc4c(i)]
RAW WATER TO NOVO OUTLET
(HISTORIC FIRM YIELD 120 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
1002533	0000	DRG	CC	0006	A

Plot Date: 23/11/2023 9:59:09 am Office: ZADC1 Filename: C:\USERS\PHIL.OERTEL\DESKTOP\XHARIEP PIPELINE\SCHEMATICS\XHARIEP FEASIBILITY SCHEMATICS [2.4].DWG



BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

COMBINED	varies	
	251 100 m³	
BRANDKOP	1493.0 masl	
	136 000 m³	
LONGRIDGE	1474.9 masl	
	68 000 m³	
OTHER	varies	
	47 100 m³	

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO DEDICATE FULL SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1 (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION
RESERVOIR

ZUTARI
IMPACT. ENGINEERED.

CLIENT

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

REV	DATE	REVISION DETAILS	APPROVED
A	04/10/23	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT

XHARIEP PIPELINE FEASIBILITY STUDY

TITLE

SCHEMATIC OF OPTION 4 [Sc5f]
RAW WATER TO RUSTFONTEIN DAM
(HISTORIC FIRM YIELD 134 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
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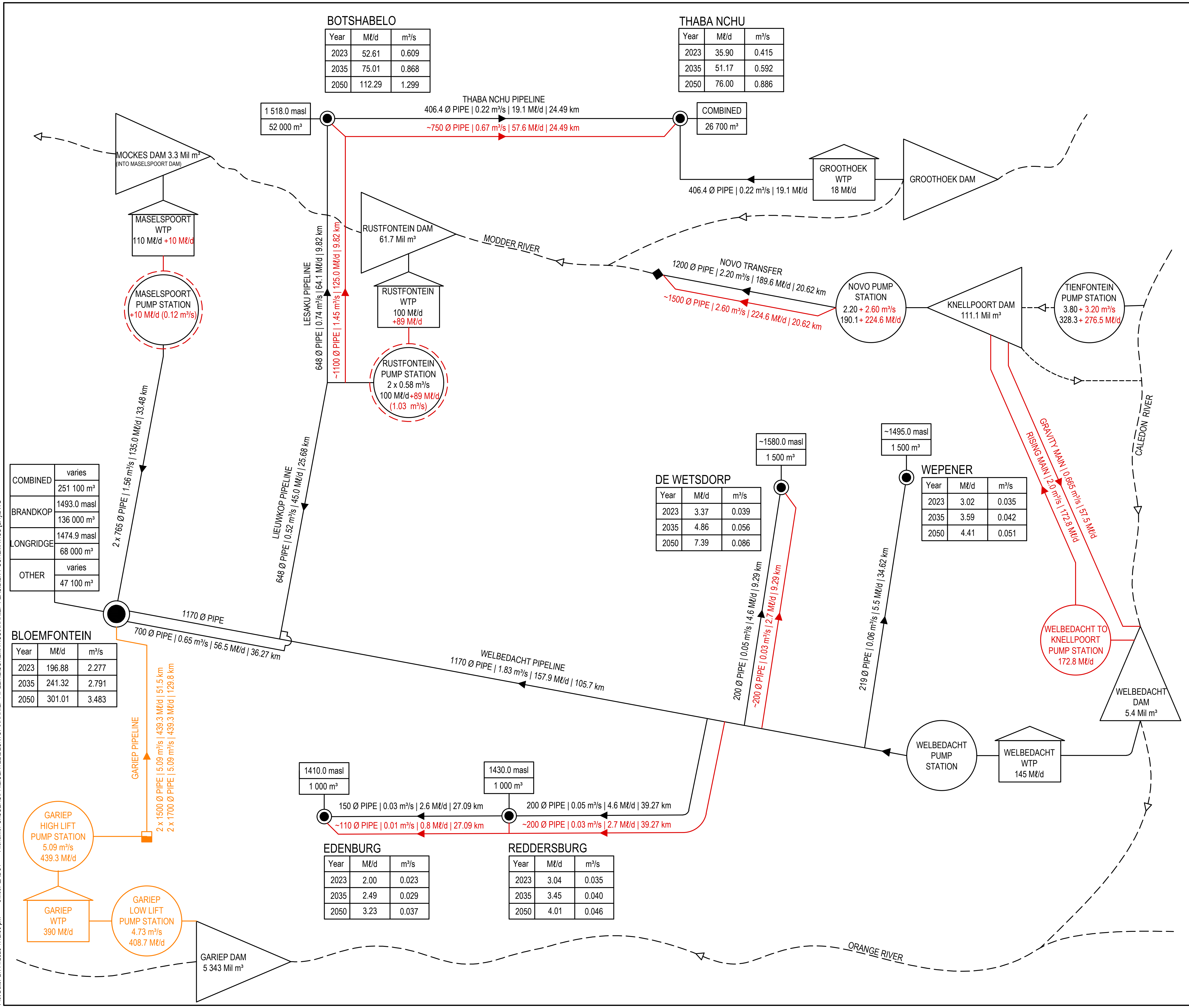

BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO DEDICATE FULL SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1 (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION
RESERVOIR

ZUTARI
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CLIENT

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

REV	DATE	REVISION DETAILS	APPROVED
A	22/11/23	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT

XHARIEP PIPELINE FEASIBILITY STUDY

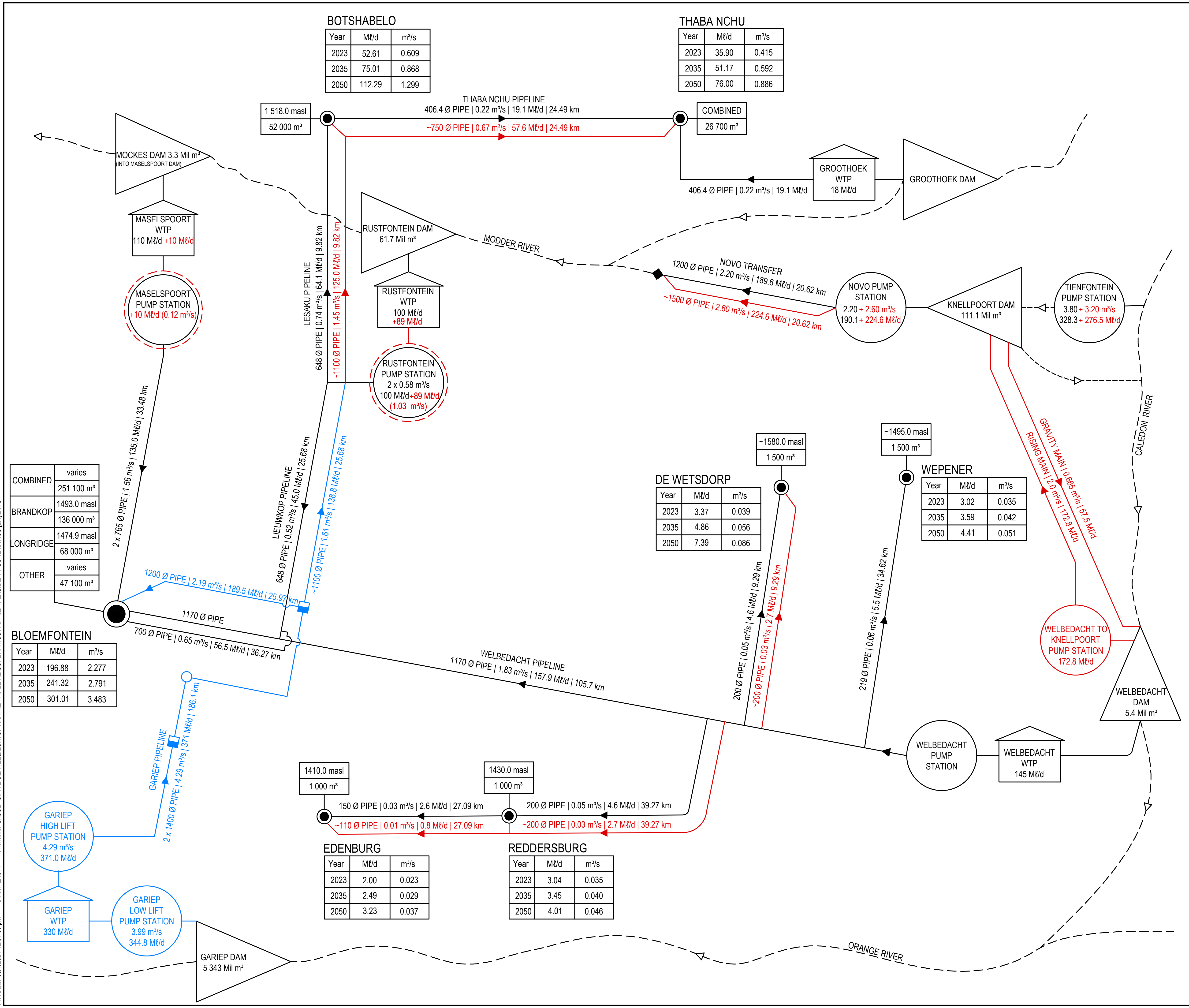
TITLE

SCHEMATIC OF OPTION 1A [Sc5b]
POTABLE WATER TO BLOEMFONTEIN
(HISTORIC FIRM YIELD 186 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
1002533-	0000 -	DRG -	CC -	0008	- A

Plot Date: 27/11/2023 1:12:00 pm Office: ZADC1 Filename: C:\USERS\PHIL.OERTEL\DESKTOP\XHARIEP PIPELINE\SCHEMATICS\XHARIEP FEASIBILITY SCHEMATICS [2.4].DWG



BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

COMBINED	varies	251 100 m³
BRANDKOP	1493.0 masl	136 000 m³
LONGRIDGE	1474.9 masl	68 000 m³
OTHER	varies	47 100 m³

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

DISCLAIMER

IF CONSTRUCTION DRAWINGS ARE ISSUED UNSIGNED, THE MASTER WITH THE ORIGINAL SIGNATURE OF APPROVAL WILL BE HELD AT THE ZUTARI OFFICE OF THE APPROVER

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO DEDICATE FULL SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1 (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION   RESERVOIR

ZUTARI
IMPACT. ENGINEERED.

CLIENT

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

REV	DATE	REVISION DETAILS	APPROVED
A	22/11/23	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT

XHARIEP PIPELINE FEASIBILITY STUDY

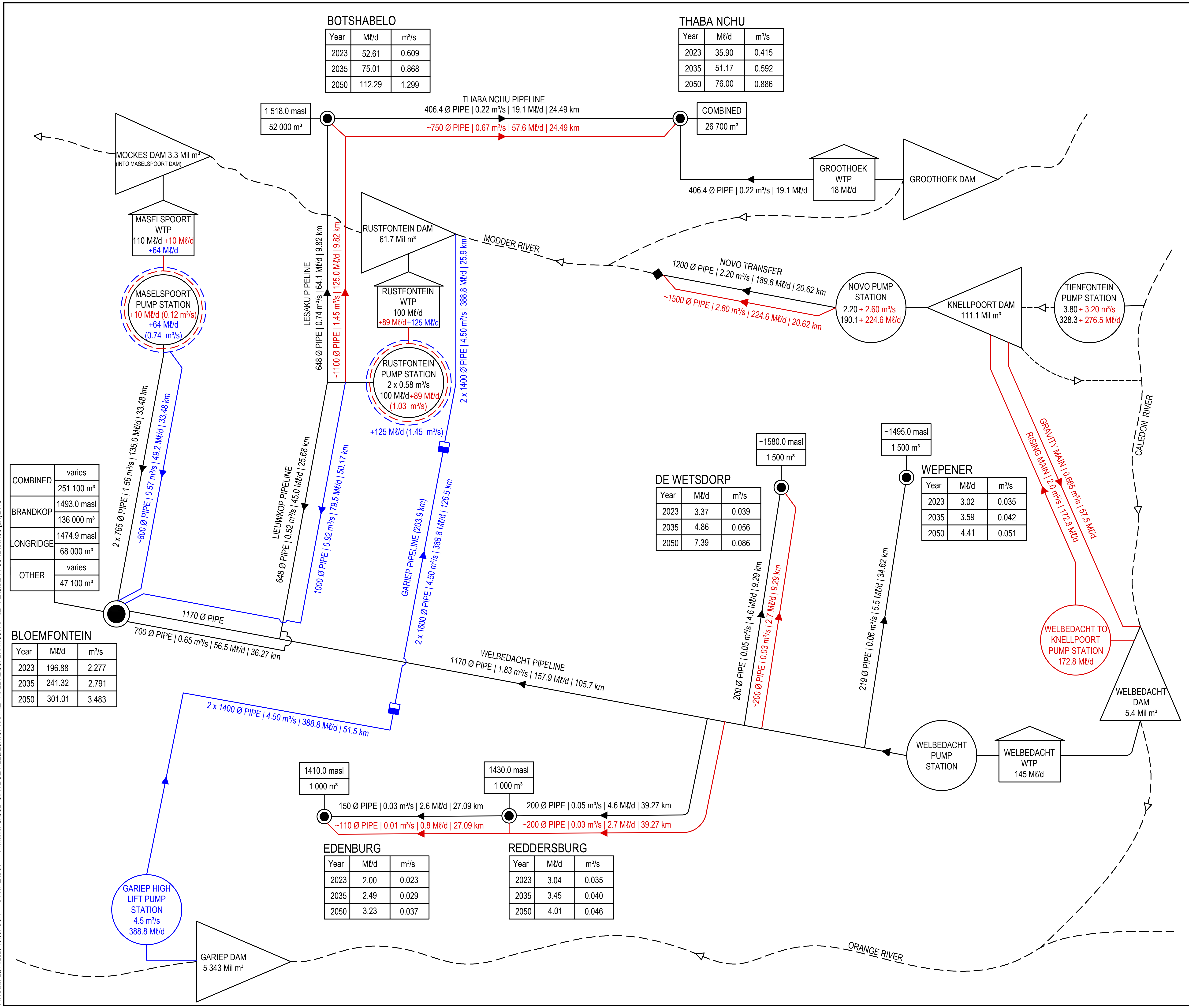
TITLE

SCHEMATIC OF OPTION 1B [Sc5f]
POTABLE WATER TO COMMAND RESERVOIR
(HISTORIC FIRM YIELD 186 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
1002533	0000	DRG	CC	0009	A

Plot Date: 30/1/2024 12:31:05 pm Office: ZACPT Filename: C:\USERS\PHIL.OERTEL\DESKTOP\XHARIEP PIPELINE\SCHEMATICS\XHARIEP FEASIBILITY SCHEMATICS [2.4].DWG



BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO SPLIT SUPPLY TO BLOEMFONTEIN AND BOTSHABELO
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1 (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION   RESERVOIR

ZUTARI
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Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

REV	DATE	REVISION DETAILS	APPROVED
A	22/11/23	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

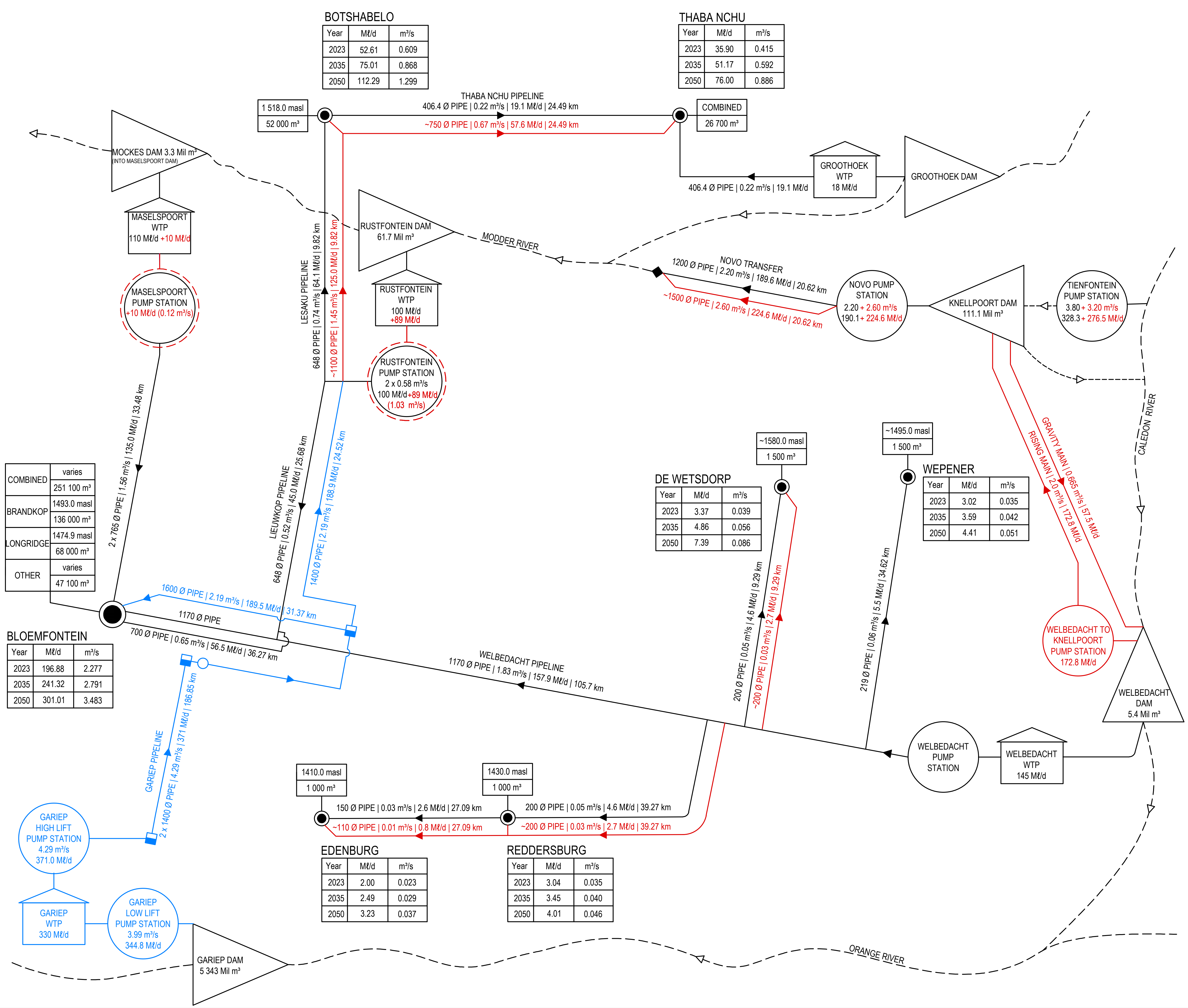
DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT
XHARIEP PIPELINE FEASIBILITY STUDY

TITLE
SCHEMATIC OF OPTION 4B [Sc5f]
RAW WATER TO RUSTFONTEIN DAM
(HISTORIC FIRM YIELD 186 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
1002533	0000	DRG	CC	0010	A



BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO DEDICATE FULL SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1B1A (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION   RESERVOIR

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Department:
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REPUBLIC OF SOUTH AFRICA

REV	DATE	REVISION DETAILS	APPROVED
A	06/02/24	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS DATE ECSA 20010010

PROJECT

XHARIEP PIPELINE FEASIBILITY STUDY

TITLE

SCHEMATIC OF OPTION 1B1A [Sc5f] POTABLE WATER TO 1530 MASL COMMAND RESERVOIR (HISTORIC FIRM YIELD 186 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
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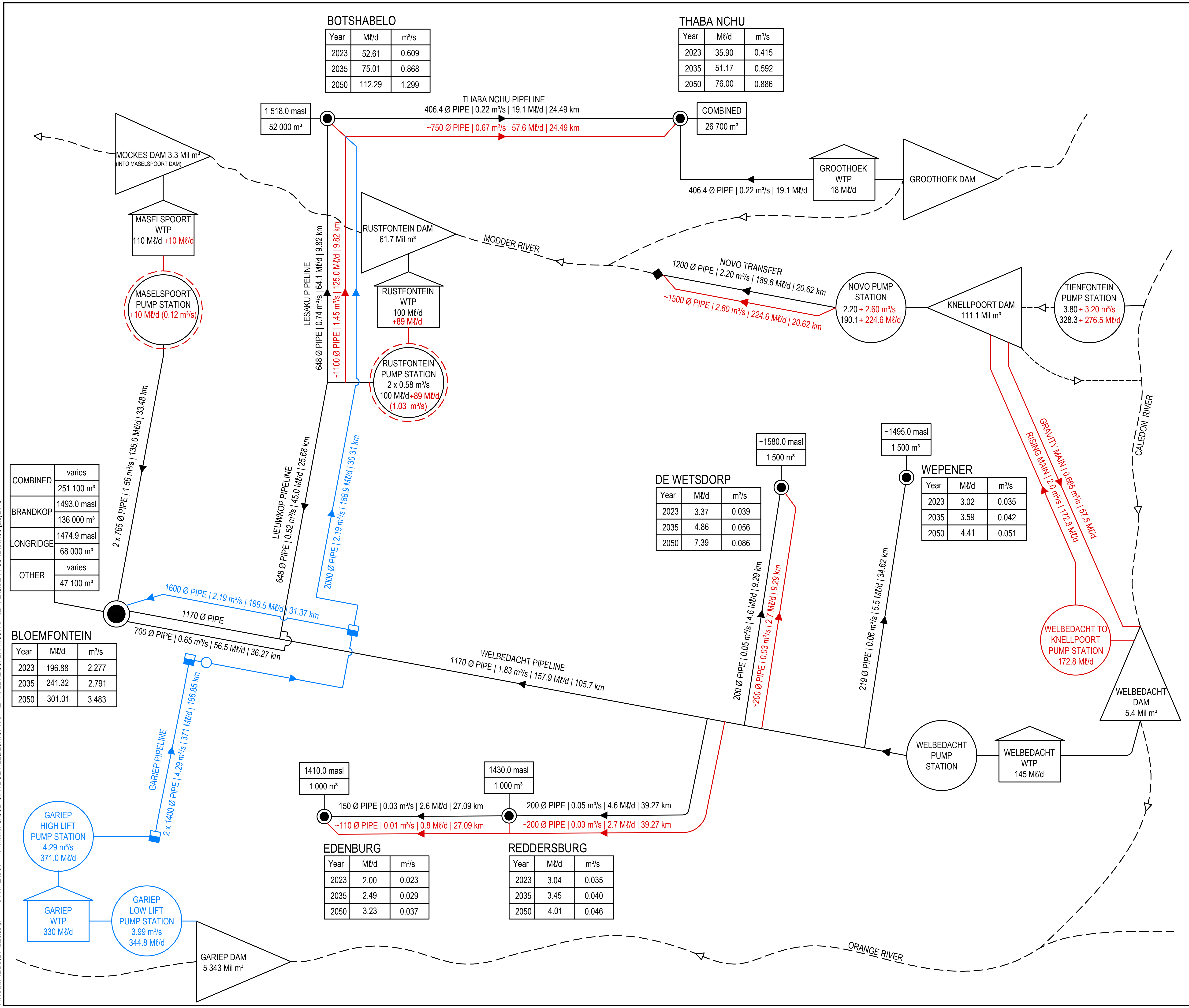


BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

COMBINED		varies
		251 100 m³
BRANDKOP		1493.0 masl
		136 000 m³
LONGRIDGE		1474.9 masl
		68 000 m³
OTHER		varies
		47 100 m³

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

DISCLAIMER

IF CONSTRUCTION DRAWINGS ARE ISSUED UNSIGNED, THE MASTER WITH THE ORIGINAL SIGNATURE OF APPROVAL WILL BE HELD AT THE ZUTARI OFFICE OF THE APPROVER

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO DEDICATE FULL SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1B1B (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION   RESERVOIR

ZUTARI
IMPACT. ENGINEERED.

CLIENT

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

REV	DATE	REVISION DETAILS	APPROVED
A	06/02/24	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

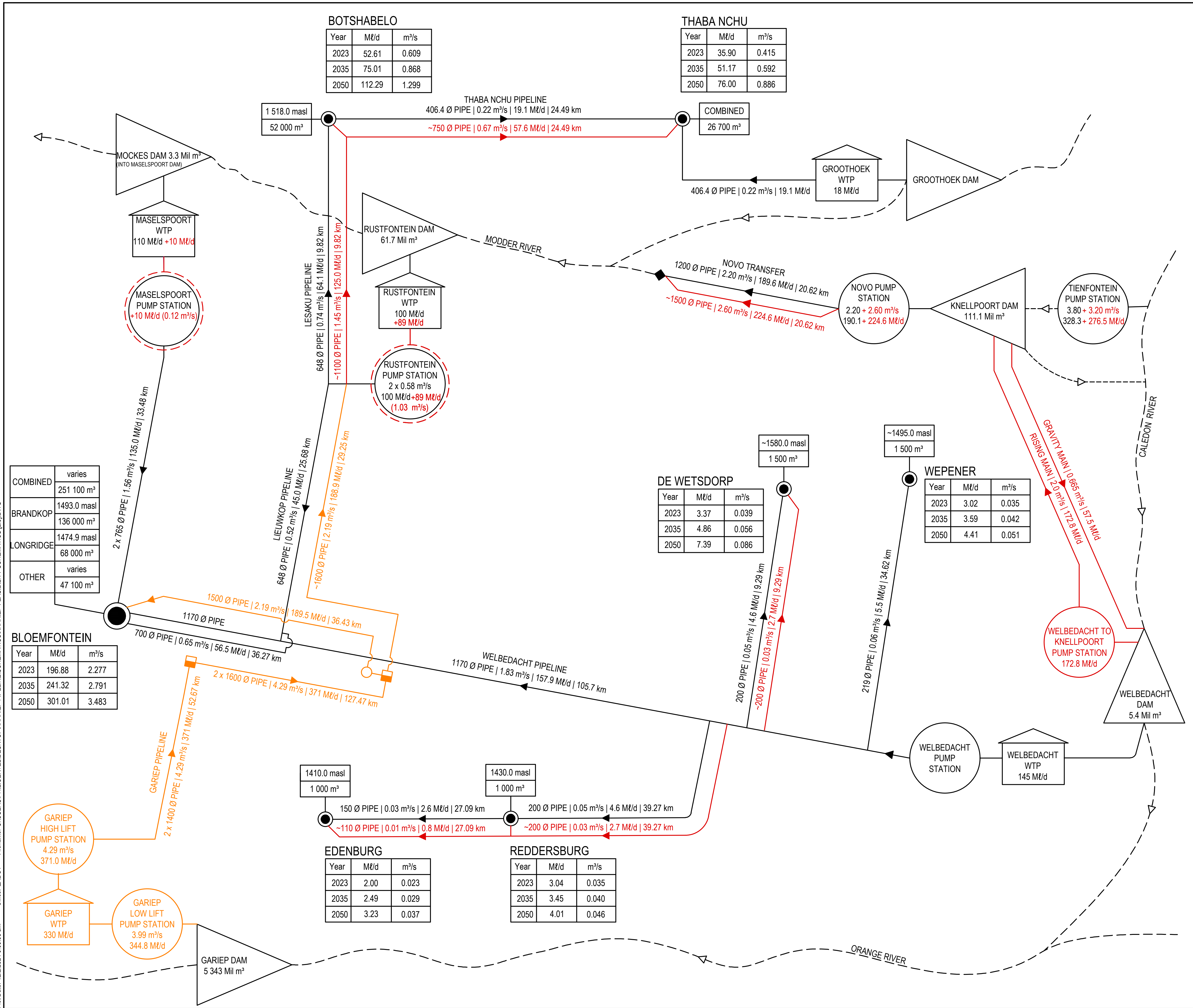
DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT
XHARIEP PIPELINE FEASIBILITY STUDY

TITLE
SCHEMATIC OF OPTION 1B1B [Sc5f]
POTABLE WATER TO 1530 MASL COMMAND RESERVOIR (HISTORIC FIRM YIELD 186 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
1002533-	0000 -	DRG -	CC -	0012 -	A



BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

DISCLAIMER

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. RUSTFONTEIN WTW TO DEDICATE FULL SUPPLY CAPACITY TO BOTSHABELO AND THABA NCHU.
3. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES ACROSS ALL OPTIONS (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

REQUIRED UPGRADES FOR OPTION 1B2 (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION — RESERVOIR

ZUTARI
IMPACT. ENGINEERED.

CLIENT

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

REV	DATE	REVISION DETAILS	APPROVED
A	06/02/24	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT

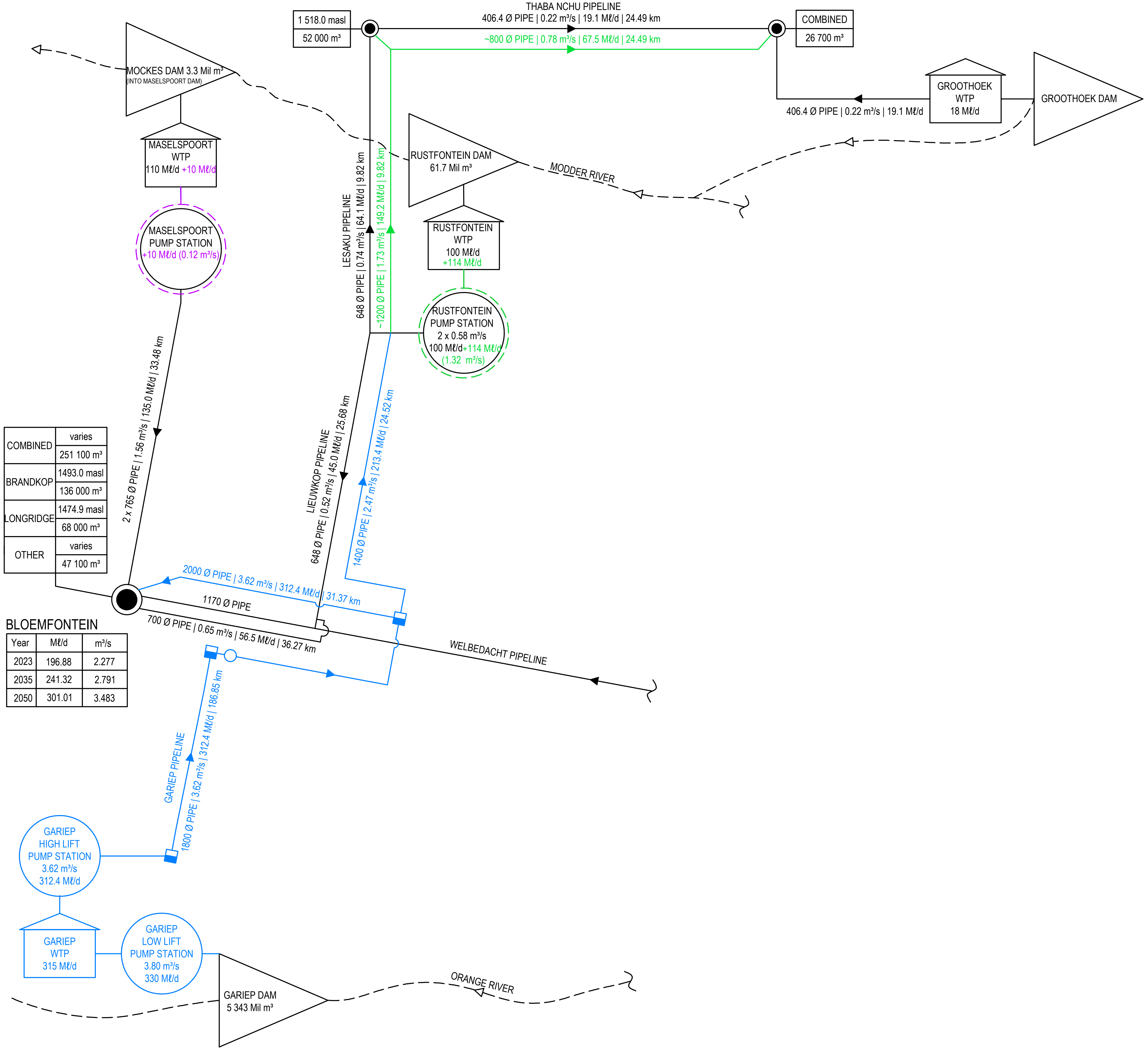
XHARIEP PIPELINE FEASIBILITY STUDY

TITLE

SCHEMATIC OF OPTION 1B2 [Sc5f]
POTABLE WATER TO 1475 MASL COMMAND RESERVOIR (HISTORIC FIRM YIELD 186 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
1002533	0000	DRG	CC	0013	A



BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

COMBINED	varies	
	251 100 m³	
BRANDKOP	1493.0 masl	
	136 000 m³	
LONGRIDGE	1474.9 masl	
	68 000 m³	
OTHER	varies	
	47 100 m³	

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

DISCLAIMER

IF CONSTRUCTION DRAWINGS ARE ISSUED UNSIGNED, THE MASTER WITH THE ORIGINAL SIGNATURE OF APPROVAL WILL BE HELD AT THE ZUTARI OFFICE OF THE APPROVER

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PROPOSED PIPELINE CAPACITIES ARE ESTIMATED BASED ON A VELOCITY OF 1.5 m/s.
2. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

XHARIEP TRANSFER INFRASTRUCTURE (2050 SCENARIO)
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)
BOOSTER PUMP STATION — RESERVOIR

UPGRADES REQUIRED BY MMM
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES REQUIRED BY VCWB
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

ZUTARI
IMPACT. ENGINEERED.

CLIENT

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

REV	DATE	REVISION DETAILS	APPROVED
A	19/02/24	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT

XHARIEP PIPELINE FEASIBILITY STUDY

TITLE

SCHEMATIC OF UPGRADES REQUIRED BY MMM AND VCWB TO ACCOMODATE XHARIEP WATER TRANSFER SCHEME

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
1002533-	0000 -	DRG -	CC -	0014 -	A

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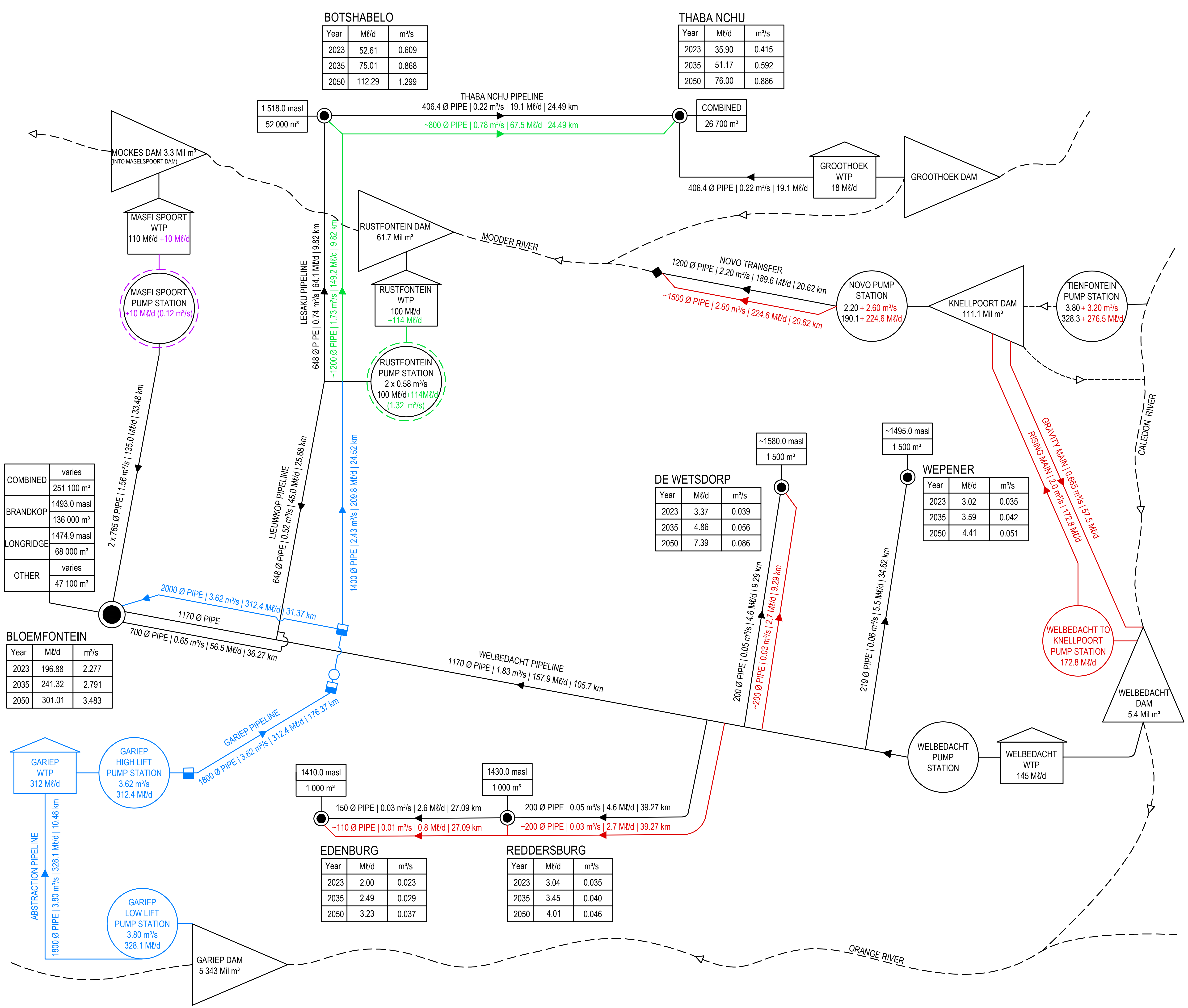


BOTSHABELO

Year	Mℓ/d	m³/s
2023	52.61	0.609
2035	75.01	0.868
2050	112.29	1.299

THABA NCHU

Year	Mℓ/d	m³/s
2023	35.90	0.415
2035	51.17	0.592
2050	76.00	0.886

COMBINED	varies	
	251 100 m³	
BRANDKOP	1493.0 masl	
	136 000 m³	
LONGRIDGE	1474.9 masl	
	68 000 m³	
OTHER	varies	
	47 100 m³	

BLOEMFONTEIN

Year	Mℓ/d	m³/s
2023	196.88	2.277
2035	241.32	2.791
2050	301.01	3.483

DE WETSDORP

Year	Mℓ/d	m³/s
2023	3.37	0.039
2035	4.86	0.056
2050	7.39	0.086

WEPENER

Year	Mℓ/d	m³/s
2023	3.02	0.035
2035	3.59	0.042
2050	4.41	0.051

EDENBURG

Year	Mℓ/d	m³/s
2023	2.00	0.023
2035	2.49	0.029
2050	3.23	0.037

REDDERSBURG

Year	Mℓ/d	m³/s
2023	3.04	0.035
2035	3.45	0.040
2050	4.01	0.046

DISCLAIMER

IF CONSTRUCTION DRAWINGS ARE ISSUED UNSIGNED, THE MASTER WITH THE ORIGINAL SIGNATURE OF APPROVAL WILL BE HELD AT THE ZUTARI OFFICE OF THE APPROVER

LEGEND

- PRESSURISED DELIVERY PIPELINE
- OVERLAND FLOWS
- ABSTRACTION PIPELINES
- NOVO OUTFALL
- T.W.L / CAPACITY — LOCAL STORAGE RESERVOIR
- DEMAND CENTER (WITH TABLED DEMANDS)

NOTES

1. PIPELINE INFORMATION IS SET OUT AS SHOWN BELOW:

EXISTING INFRASTRUCTURE
DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

ADDITIONAL INTERVENTIONS AND UPGRADES
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES REQUIRED BY MMM
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

UPGRADES REQUIRED BY VCWB
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

SCHEME 1B PRE-FEASIBILITY DESIGN
~DIA (mm) | CAPACITY(m³/s) | CAPACITY (Mℓ/d) | LENGTH (km)

BOOSTER PUMP STATION
RESERVOIR

ZUTARI
IMPACT. ENGINEERED.

CLIENT

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

REV	DATE	REVISION DETAILS	APPROVED
A	23/02/24	FOR INFORMATION	S.KLEYNHANS

SCALE	SIZE	
NTS	A3	FOR INFORMATION

DRAWN: P.OERTEL
DESIGNED: P.OERTEL
REVIEWED: S.KLEYNHANS
APPROVED: S.KLEYNHANS — DATE — ECSA 20010010

PROJECT
XHARIEP PIPELINE FEASIBILITY STUDY

TITLE
PRE-FEASIBILITY DESIGN SCHEMATIC OF OPTION 1B WITH STOCHASTIC FLOW RATES (HISTORIC FIRM YIELD 186 Mm³/a)

DRAWING NUMBER

PROJECT No.	WBS	TYPE	DISC	NUMBER	REV
1002533-	0000 -	DRG -	CC -	0015 -	A

Appendix B

Sub-Option Hydraulic Gradelines

Appendix B

Bulk water infrastructure scheme development

Scheme sub-option descriptions

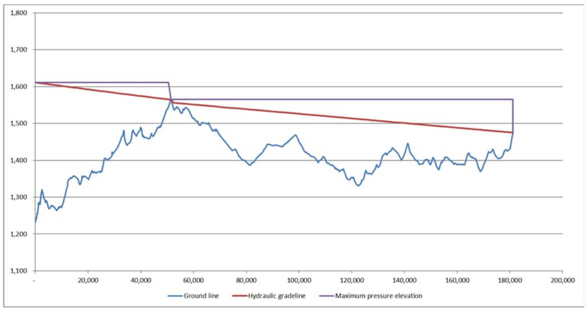
Scheme No.	Sub-option	Nominal diameters	Description of configuration	Minimum velocity (m/s)	Maximum velocity (m/s)
1 & 1A*	1	1. 1500, 1600 1A. 1400, 1700	High-Lift pump station (HLPS) and rising main to reservoir (~CH51485 m). Gravity line to Longridge reservoir.	0.72 0.66	1.29 1.80
	2	1. 1400 1A. 1500	HLPS and rising main to first reservoir at high point (~CH51485 m). Gravity line to second reservoir (~CH141156 m) and booster pump station to Longridge reservoir.	0.95 0.85	1.52 1.52
	3	1. 1600 1A. 1500, 1700	High-Lift pump station (HLPS) and rising main to reservoir (~CH51485 m). Gravity line to Longridge reservoir.	0.72 0.66	1.12 1.52
	4	1. 1400, 1700 1A. 1300 & 1700	High-Lift pump station (HLPS) and rising main to reservoir (~CH51485 m). Gravity line to Longridge reservoir.	0.64 0.66	1.52 2.08
	5	1. 1300, 1400 1A. 1300, 1500, 1400	HLPS and rising main to first reservoir (~CH51485 m). Gravity line to second reservoir (~CH141156 m) and booster pump station to Longridge reservoir.	0.95 0.85	1.76 2.08
2	1	1300 & 1600	HLPS and rising main to first reservoir (~CH134713 m). Gravity line to booster pump station (~CH168940 m) and rising main to second reservoir (~CH177698 m). Gravity to Knellpoort Dam.	0.53	1.57
	2	1300	HLPS and rising main to one reservoir at high point (~CH177698 m). Gravity line to Knellpoort Dam.	0.81	1.57
	3	1300	HLPS and rising main to first reservoir at local high point (~CH107482 m). Booster pump station and rising main to second reservoir (~177698 m). Gravity to Knellpoort Dam.	0.81	1.57
	4	1400	HLPS and rising main to one reservoir at high point (~CH177698 m). Gravity line to Knellpoort Dam.	0.70	1.34
	5	1400	HLPS and rising main to first reservoir at local high point (~CH107482 m). Booster pump station to second reservoir (~177698 m). Gravity to Knellpoort Dam.	0.70	1.31
3	1	1300	HLPS and rising main to first reservoir (~CH62920 m). Gravity line to booster pump station (~98778 m) and rising main to second reservoir (~CH167427 m). Gravity to Novo outfall.	1.29	1.57
	2	1300	HLPS and rising main to first reservoir (~CH118672 m). Booster pump station and rising main to second reservoir (~CH167427 m). Gravity to Novo outfall.	1.29	1.57

Scheme No.	Sub-option	Nominal diameters	Description of configuration	Minimum velocity (m/s)	Maximum velocity (m/s)
3	3	1400	HLPS and rising main to first reservoir (~CH62920 m). Gravity line to booster pump station (~98778 m) and rising main to second reservoir (~CH167427 m). Gravity to Novo outfall.	1.11	1.31
	4	1300 & 1400	HLPS and rising main to first reservoir (~CH62920 m). Gravity line to booster pump station (~98778 m) and rising main to second reservoir (~CH167427 m). Gravity to Novo outfall.	1.11	1.49
	5	1200	HLPS and rising main to first reservoir (~CH62920 m). Gravity line to booster pump station (~96017 m) and rising main to second reservoir (~CH167427 m). Gravity to Novo outfall.	1.29	1.87
4 & 4B*	1	4. 1300 4B. 1400	HLPS and rising main to first reservoir (~CH51485 m). Gravity line to booster pump station (~135446 m) and rising main to second reservoir (~CH178024 m). Gravity to Rustfontein Dam.	1.12 1.01	1.57 1.59
	2	4. 1400 & 1500 4B. 1400, 1500, 1600	HLPS and rising main to first reservoir (~CH51485 m). Gravity line to booster pump station (~135446 m) and rising main to second reservoir (~CH178024 m). Gravity to Rustfontein Dam.	0.83 0.77	1.31 1.52
	3	4. 1200 & 1600 4B. 1300 & 1600	HLPS and rising main to one reservoir at high point (~CH51485 m). Gravity line to Rustfontein Dam.	0.73 0.77	1.85 1.86
	4	4. 1300 & 1500 4B. 1400 & 1600	HLPS and rising main to one reservoir at high point (~CH51485 m). Gravity line to Rustfontein Dam.	0.83 0.77	1.57 1.59
	5	4. 1200, 1300, 1500 4B. 1400 & 1600	HLPS and rising main to first reservoir at high point (~CH51485 m) and gravity to second reservoir (CH178024 m). Gravity line to Rustfontein Dam.	0.83 0.77	1.76 1.59
1B	1	Main 1400 Longridge 1200 Rustfontein 1100	HLPS and rising main to first reservoir (~CH50925 m). Gravity line to booster pump station (~151005 m) and rising main to command reservoir (~CH186119 m). Gravity to Longridge Reservoir and Rustfontein WTW.	0.93 2.03 1.77	1.52 2.03 1.81

Hydraulic grade lines of scheme sub-options

Scheme 1: Gariep Dam to Bloemfontein (Longridge reservoir)

Sub-option 1



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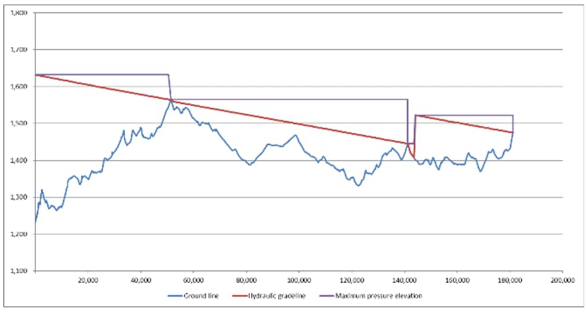

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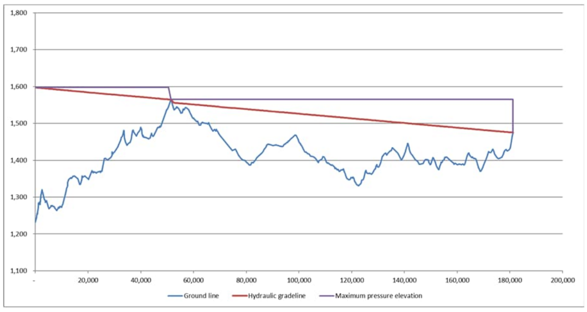

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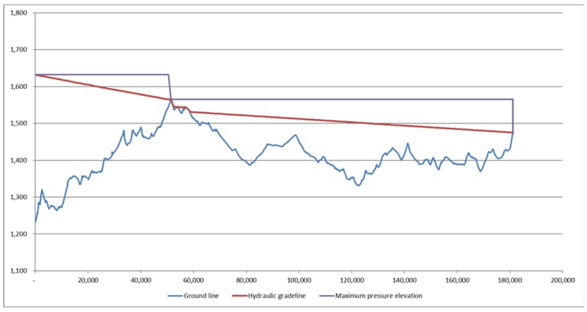

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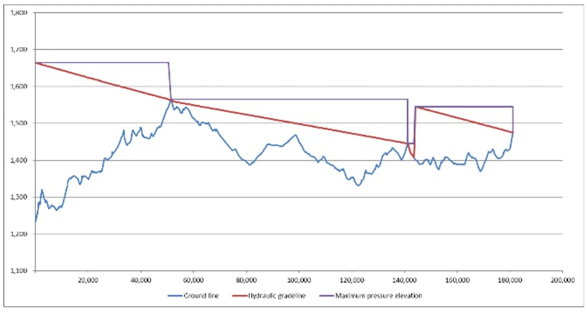
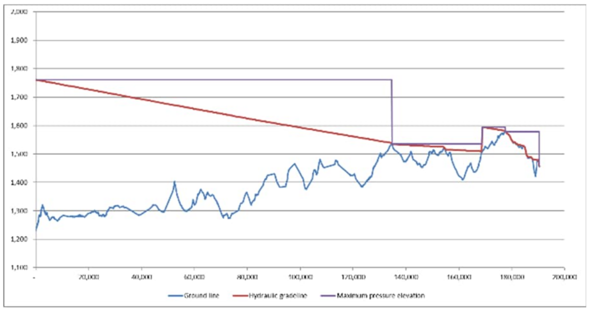


Scheme 2: Gariep Dam to Knellpoort Dam

Sub-option 1



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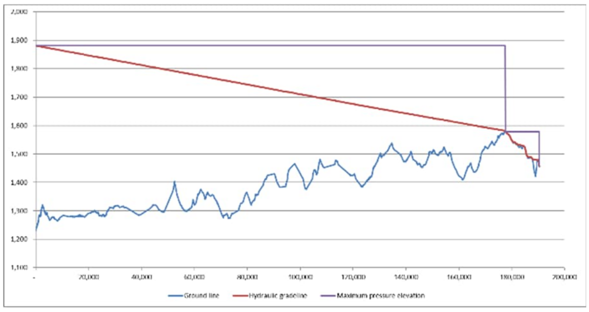

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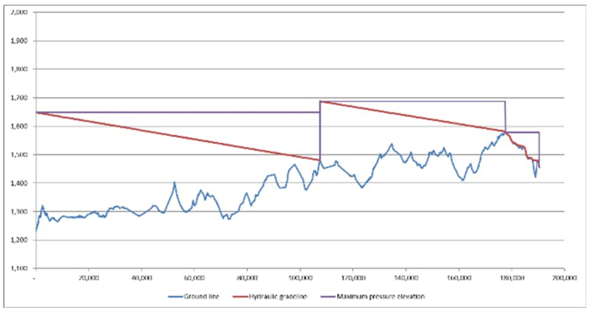

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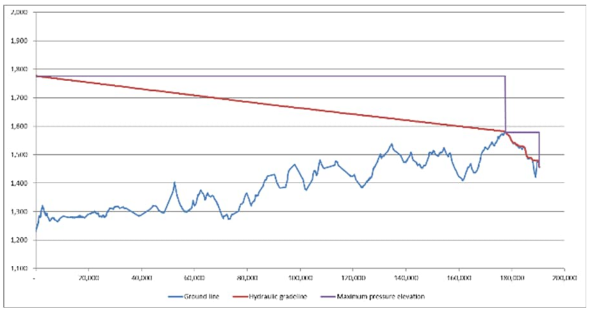

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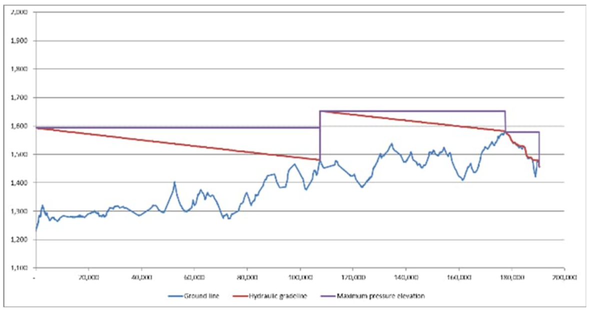
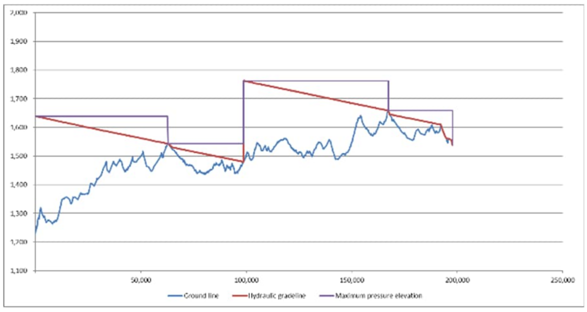


Scheme 3: Gariep Dam to Novo outfall

Sub-option 1



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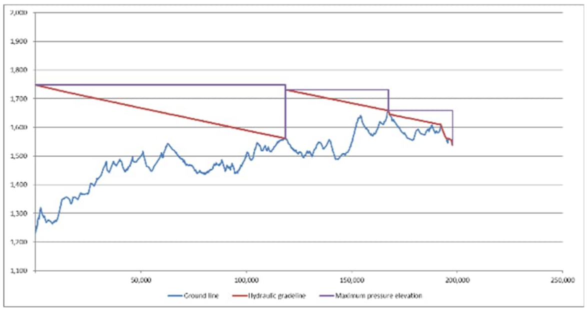

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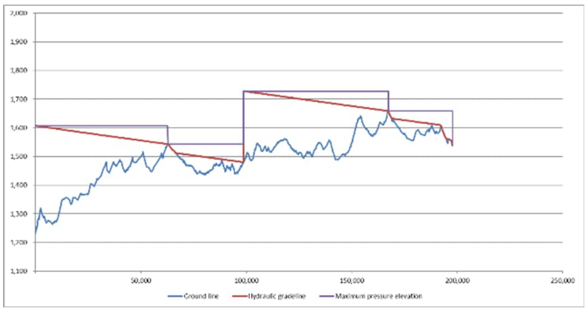

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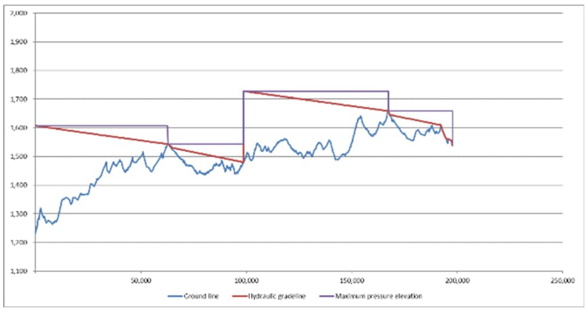

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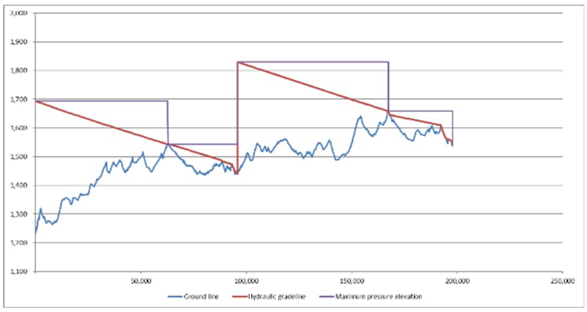
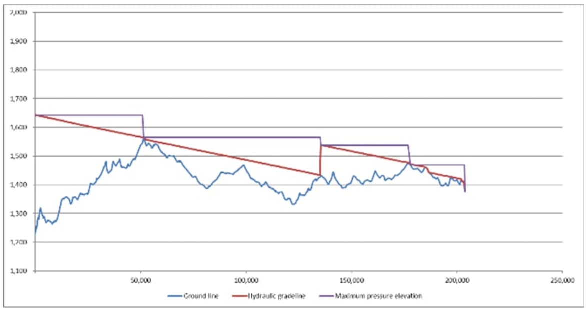


Scheme 4: Gariep Dam to Rustfontein Dam

Sub-option 1



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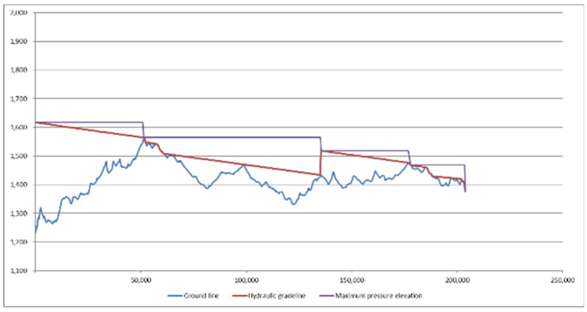

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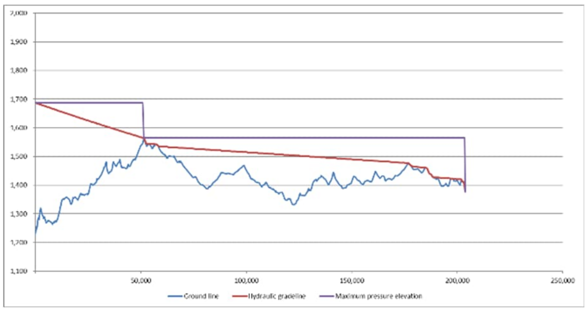

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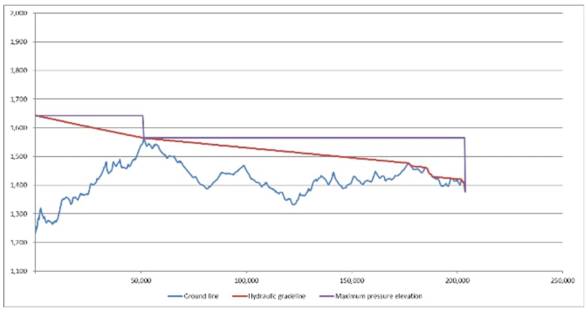

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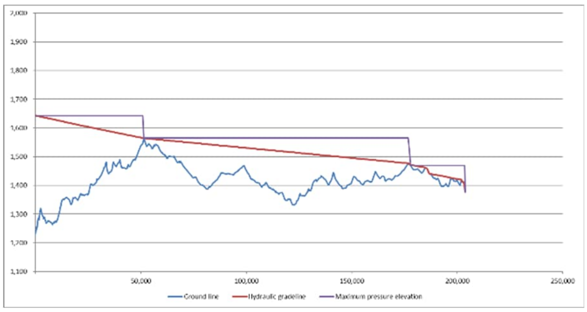
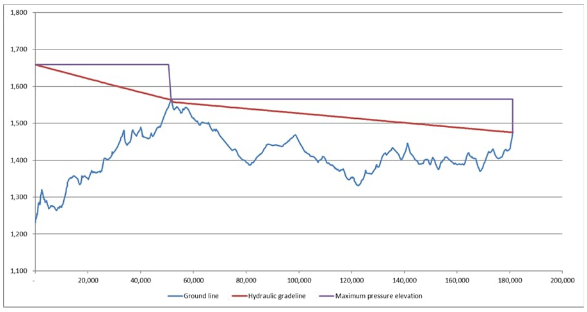


Scheme 1A: Gariep Dam to Bloemfontein (Longridge reservoir)

Sub-option 1



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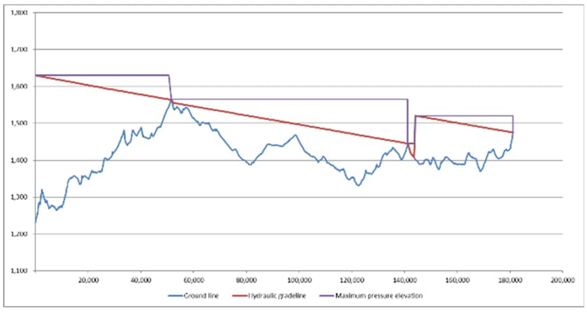

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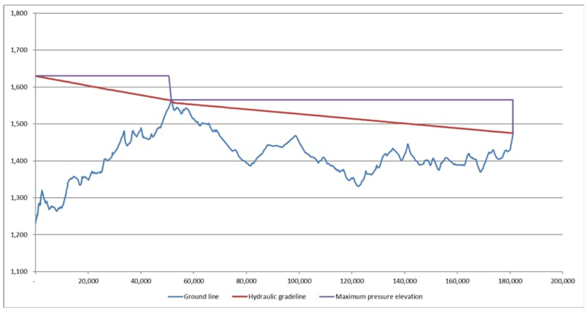

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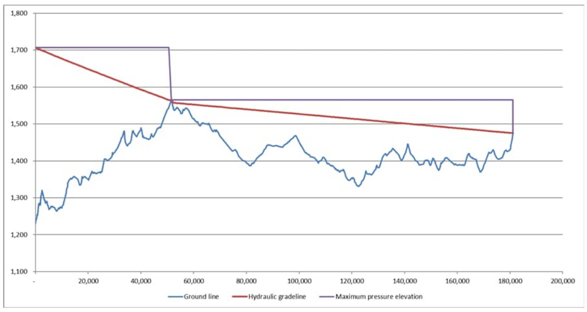

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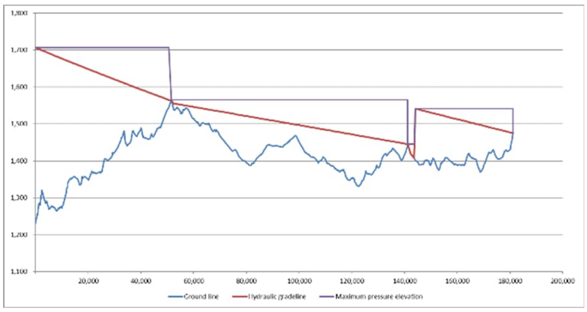
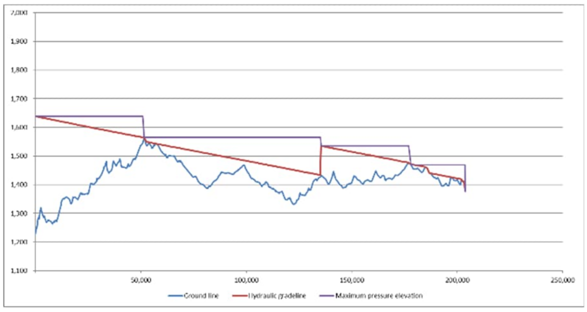


Scheme 4B: Gariep Dam to Rustfontein Dam

Sub-option 1



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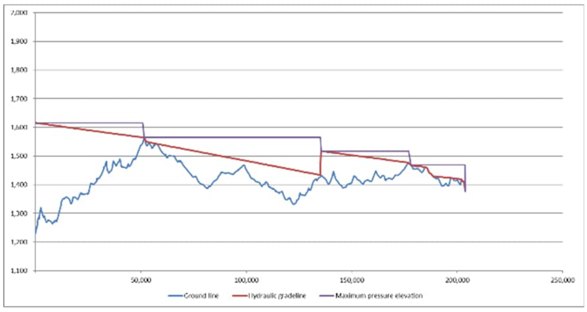

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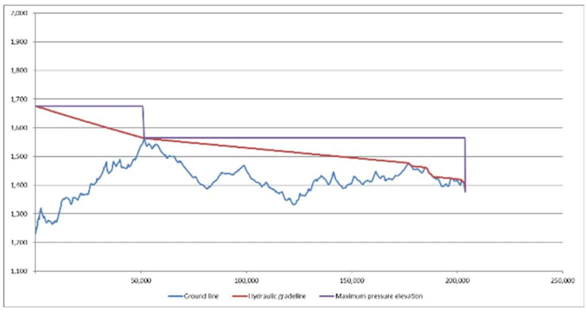

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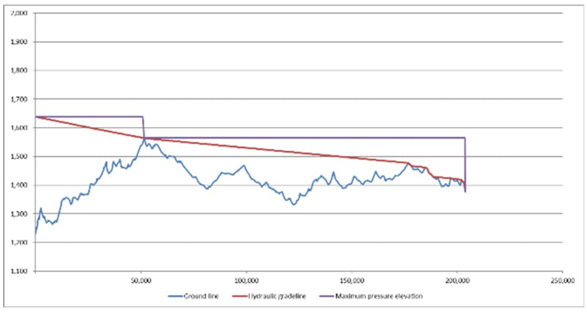

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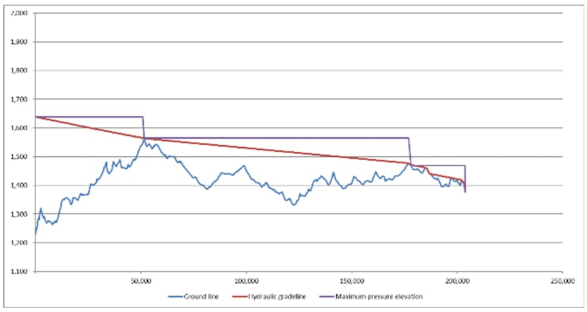
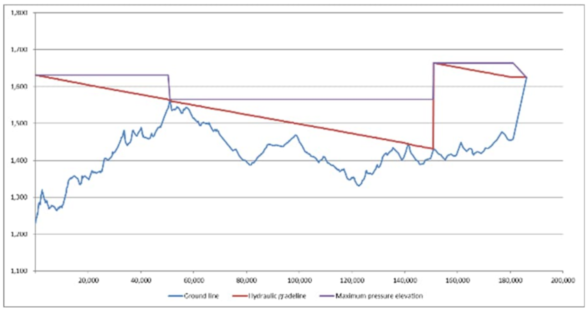
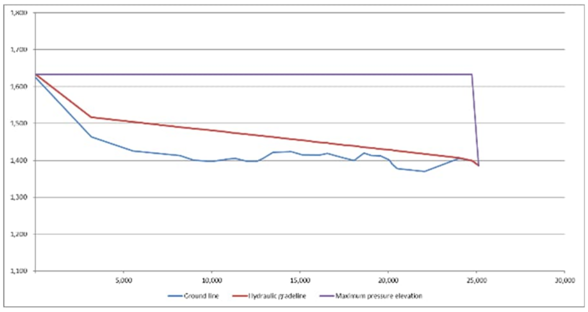


Scheme 1B: Gariep Dam to command reservoir and gravity to Rustfontein PS and Longridge Reservoir

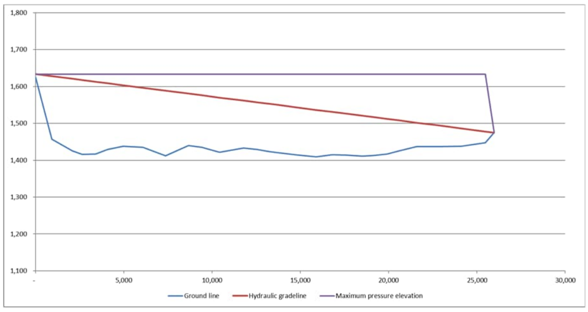
Main transfer Xhariep to Command



Gravity from Command to Rustfontein PS



Gravity from Command to Longridge Reservoir



Scheme sub-option selection tables

Scheme 1: Preferred sub-option 3

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
Pump station at start. DN1500 to reservoir at high point. DN1600 Gravity System, no BPT.	11950.6	12621.7	24572.3	6.4	12.37	OPTION 1	0.72	1.29
All DN1400. Pump station at start. Reservoir at high point. Booster station after 2nd reservoir.	10921.9	13544.3	24466.2	6.8	12.32	OPTION 2	0.95	1.52
Pump station at start. DN1600 to reservoir at high point. DN1600 Gravity System, no BPT.	11895.1	12542.7	24437.7	6.3	12.31	OPTION 3	0.72	1.12
Pump station at start. DN1400 to reservoir at high point. DN1700 Gravity System, no BPT.	12318.5	12802.2	25120.7	6.4	12.65	OPTION 4	0.64	1.52
Pump station at start. DN1300 to Reservoir at high point. DN1400 gravity. Booster station after 2nd reservoir.	10665.6	13846.2	24511.8	7.0	12.34	OPTION 5	0.95	1.76

Scheme 2: Preferred sub-option 3

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
DN1300 rising main, two reservoirs at high points, DN1600 stretch to booster pump station.	12885.0	12021.8	24906.8	6.7	13.9	OPTION 1	0.53	1.57
DN1300 Pump station at start. One reservoir at high point.	15448.2	12518.0	27966.1	7.0	15.6	OPTION 2	0.81	1.57
DN1300 rising main, two reservoirs and booster pump station	9948.2	11648.4	21596.6	6.5	12.02	OPTION 3	0.81	1.57
DN1400 Pump station at start. One reservoir at high point.	14824.3	12066.3	26890.6	6.7	15.0	OPTION 4	0.70	1.34
DN1400 rising main, two reservoirs and booster pump station	10288.9	11308.8	21597.7	6.3	12.02	OPTION 5	0.70	1.31

Scheme 3: Preferred sub-option 4

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
DN1300 Pump station at start. Two reservoirs at high points with booster pump station inbetween.	8833.0	14373.4	23206.4	8.0	12.8	OPTION 1	1.29	1.57
DN1300 Pump station at start. Two reservoirs at high points with booster pump station inbetween (2)	10233.2	14446.2	24679.4	8.0	13.7	OPTION 2	1.29	1.57
DN1400 Pump station at start. Two reservoirs at high points with booster pump station inbetween.	9117.7	13854.6	22972.3	7.7	12.7	OPTION 3	1.11	1.31
DN1400 and DN1300. Pump station at start. Two reservoirs at high points with booster pump station.	8917.9	13811.1	22729.0	7.6	12.6	OPTION 4	1.11	1.49
DN1200 rising main, two reservoirs at high points, DN1300 gravity end	9339.2	15921.4	25260.7	8.8	14.0	OPTION 5	1.29	1.87

Scheme 4: Preferred sub-option 1

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
All DN1300. Pump station at start. Two reservoirs at high points with booster pump station inbetween.	8473.2	12718.1	21191.3	6.5	10.84	OPTION 1	1.12	1.57
DN1400/DN1500 Pump station at start. Two reservoirs at high points with booster pump station inbetween.	9209.3	12558.1	21767.4	6.4	11.1	OPTION 2	0.83	1.31
DN1200, DN1300 & DN1500 Pump station at start. Two reservoirs at high points with booster pump station inbetwee	9939.7	12364.2	22303.9	6.3	11.4	OPTION 3	0.73	1.85
DN1300 to high point reservoir. DN1500 gravity thereafter.	9430.4	12009.5	21440.0	6.1	11.0	OPTION 4	0.83	1.57
DN1300 to high point DN1500 to second high point reservoir. DN1400 gravity thereafter.	9254.1	11976.1	21230.1	6.1	10.86	OPTION 5	0.83	1.76

Scheme 1A: Preferred sub-option 3

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
Pump station at start. DN1400 to reservoir at high point. DN1700 Gravity System, no BPT.	24621.8	22142.1	46763.9	10.0	21.18	OPTION 1	0.66	1.80
Pump station at start. DN1500 to Reservoir at high point. DN1500 gravity. Booster station after 2nd reservoir.	23345.3	23848.7	47194.0	10.8	21.37	OPTION 2	0.85	1.52
Pump station at start. DN1500 to reservoir at high point. DN1700 Gravity System, no BPT.	25120.5	21982.9	47103.3	10.0	21.33	OPTION 3	0.66	1.52
Pump station at start. DN1300 to reservoir at high point. DN1700 Gravity System, no BPT.	24681.8	22590.7	47272.5	10.2	21.41	OPTION 4	0.66	2.08
Pump station at start. DN1300 to Reservoir at high point. DN1500 gravity. Booster station after 2nd reservoir.	22775.1	24813.8	47588.9	11.2	21.55	OPTION 5	0.85	2.08

Scheme 4B: Preferred sub-option 5

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
All DN1400. Pump station at start. Two reservoirs at high points with booster pump station inbetween.	19800.2	22518.0	42318.2	10.2	19.2	OPTION 1	1.01	1.59
DN1400, DN1500 & DN1600 Pump station at start. Two reservoirs at high points with booster pump station	20701.0	22137.8	42838.8	10.0	19.4	OPTION 2	0.77	1.52
DN1300 to high point reservoir. DN1600 gravity thereafter.	21517.8	21369.5	42887.3	9.7	19.4	OPTION 3	0.77	1.86
DN1400 to high point reservoir. DN1600 gravity thereafter.	21518.7	20979.8	42498.4	9.5	19.2	OPTION 4	0.77	1.59
DN1400 to high point DN1600 to second high point reservoir. DN1400 gravity thereafter.	21317.3	20940.9	42258.2	9.5	19.1	OPTION 5	0.77	1.59

Scheme 1B (hybrid): Comparable sub-option

CONFIGURATION OPTIONS	Total Capital Cost (million)	NPV O&M (million)	TOTAL NPV (million)	O&M URV R/m^3	URV R/m^3	Option no.	V min m/s	V max m/s
Pump station at start. DN1500 to reservoir at high point.	35098.8	25546.9	60645.7	11.6	27.47	OPTION 1	0.81	1.31
All DN1400. Pump station at start to Reservoir at high point, gravity to booster station to termination reservoir.	21846.3	24675.1	46521.4	11.2	21.07	OPTION 2	0.93	1.52

Appendix C

Gariep WTW Site Inspection Report

Report Number: Appendix

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

Directorate: Water Resource Development Planning
Department of Water & Sanitation
Private Bag X313
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South Africa
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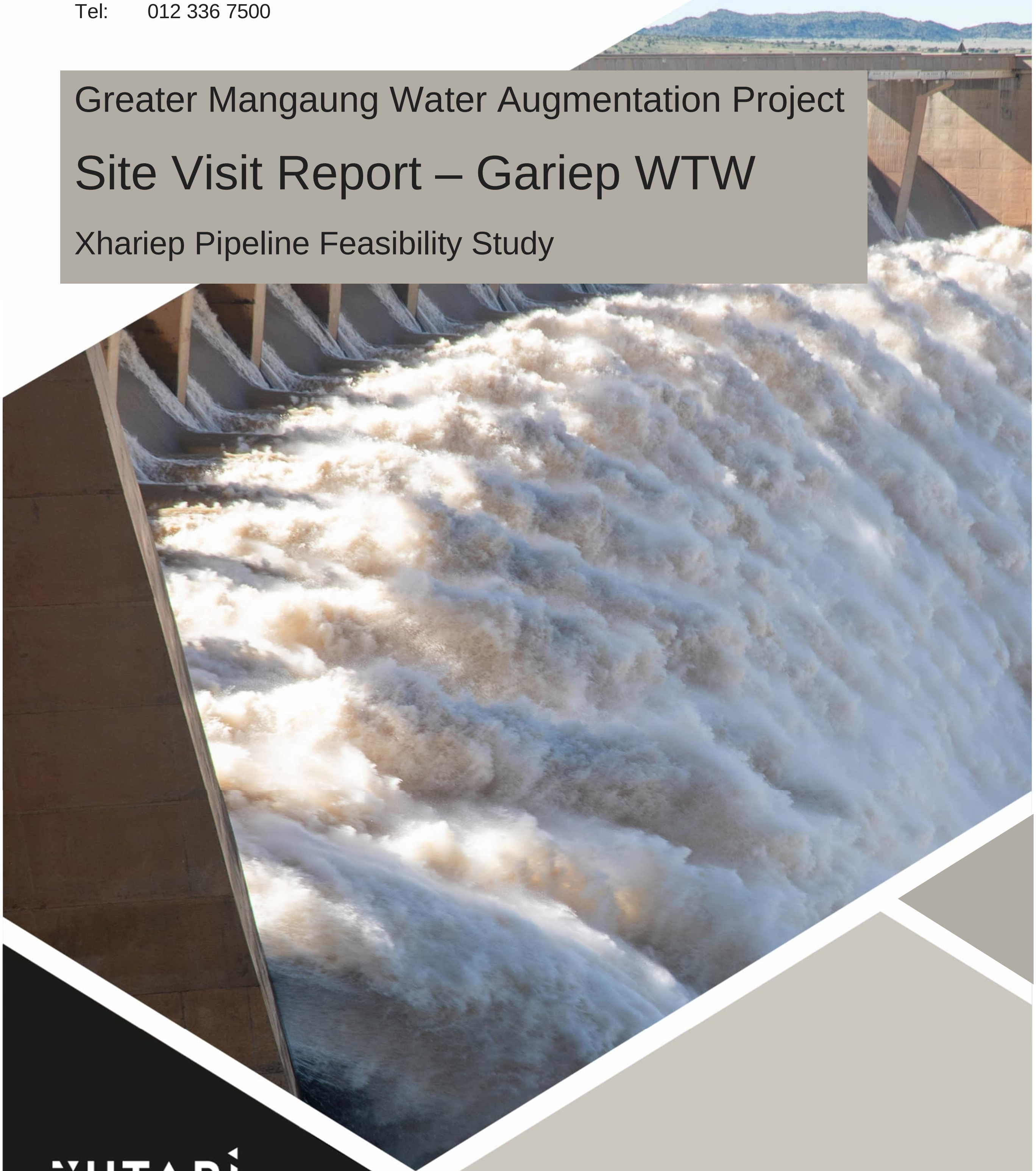
Greater Mangaung Water Augmentation Project

Site Visit Report – Gariep WTW

Xhariep Pipeline Feasibility Study



ZUTARI
IMPACT. ENGINEERED.

Submission date: 14 March 2024
Revision: A

Contents

Figures

Tables

1 Introduction

When developing a new treatment works, it is beneficial to have a pilot study undertaken, or alternatively a bench scale test in batch fashion in a laboratory. This project is fortunate to have a reference plant in the form of the Xhariep water treatment plant, which treats the same water as intended for the new Xhariep WTW. This report is a record of the Zutari team visit to the Gariep Water Treatment Works. The site visit took place on 30 January 2024. The purpose of the site visit was to inspect the existing treatment works, interview the staff of Vaal Central Water (VCW) on the operation of the works, their understanding of the raw water quality and their general experience in the treatment of the Gariep Dam water. During the visit, water quality measurement data for the raw and final water was gathered to supplement the design of the new water treatment works.

Table 1-1 provides a list of the site visit participants.

Table 1-1 List of site visit participants

Name	Institution
Louis Krouwkamp	Zutari (Tshwane)
David van der Westhuizen	Zutari (Bloemfontein)
Mike Brummer	Vaal Central Water

2 Observations of Existing Infrastructure

2.1 Description of the treatment works

The existing Gariep WTW has a design capacity of 2.8 Mℓ/day. The plant is located to the west of the Xhariep Dam, with the exact location of 30°37'33.98"S and 25°29'9.99"E. The image below provides the locality of the works.

Figure 1: Locality of Gariep WTP

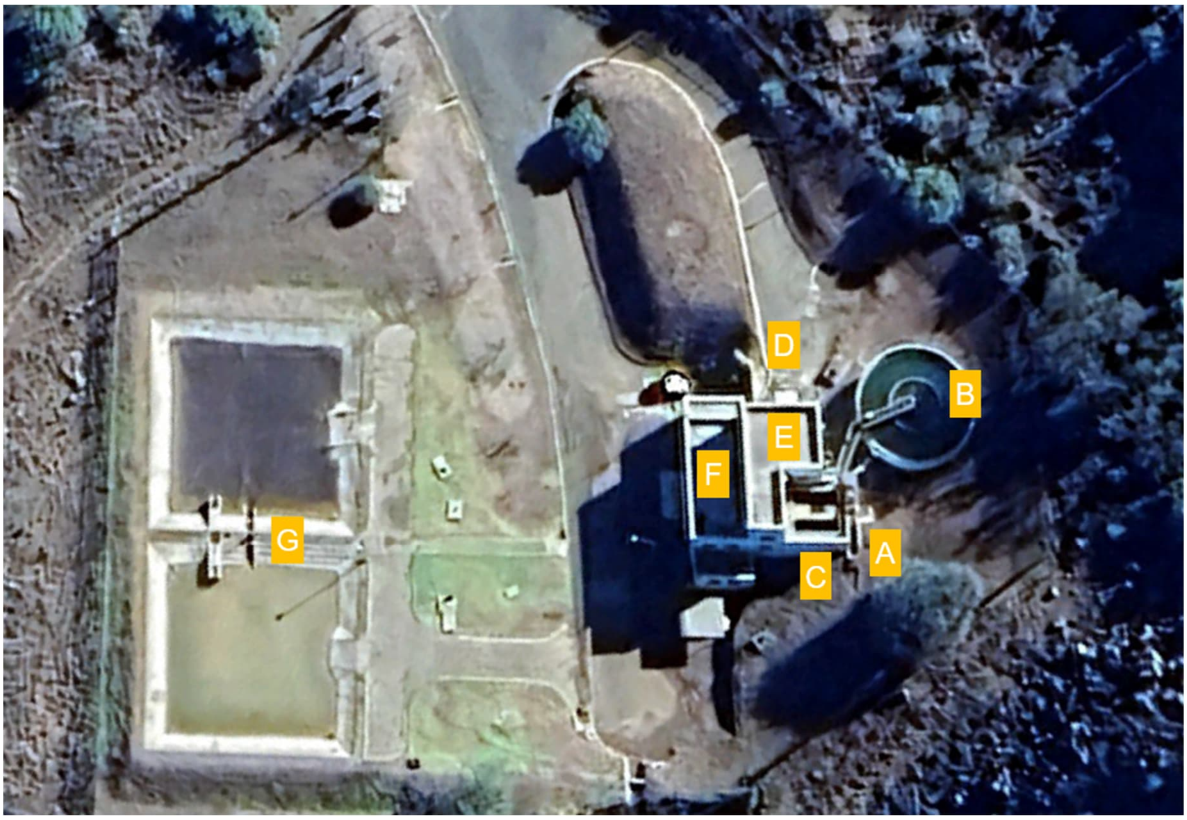
The treatment process is described as follows:

Water is abstracted from the Xhariep dam wall through a 2.1m Ø pipe which flows from east to west along the Orange river downstream of the dam wall. An offtake from this pipeline feeds to the inlet of the treatment works. Water enters the works through a single pipe into a receiving chamber. Water is dosed with a polymer (Ultrafloc U3500) in this chamber. Water flows to a second chamber equipped with a top entry mixer, where lime is dosed. The plant is equipped with lime dosing equipment, but reported no lime is being dosed due to good water quality and stability. From the lime dosing chamber, water flows to the single clarifier. The clarifier is a circular half bridge, sloped floor unit with a central mixing chamber. The mixing chamber is equipped with mixer arms rotating along with the bridge.

Clarified water flows over a concrete weir on the perimeter of the circular tank into a collection box. A pipe transfers the clarified water to the inlet of the filters. Intermediate chlorination is dosed in the pipe just upstream of the filter inlets. The plant is equipped with two rapid gravity sand filters. The filters are equipped with false floor underdrain system. Filtered water is collected in the clear water tank situated under the main admin building.

The high lift pump station supplies water to the Bergsig reservoir, Eskom and a small reservoir close to the treatment works. Backwash pumps and air blowers for the filters are situated in this pump station. The backwash pumps draw water directly from the clear water tank.

Sludge from the clarifier, and backwash water from the filters are discharged to the two sludge ponds on site. Supernatant water is returned to the inlet works and dried sludge is periodically removed from site. The image below provides identification of the main process units and their locations on site.



A	Inlet structure and chemical dosing point	B	Clarifier
C	Rapid gravity filters	D	Chlorine dosing equipment
E	Eastern building portion containing chemical dosing (top floor) and clear water tank (bottom)	F	Western building portion containing admin and laboratory (top) and pump station (bottom)
G	Residuals handling facility		

Figure 2: Site description

2.2 Site observations and plant condition

This section provides the site observations and high level plant condition. It is noted that this is not a condition assessment nor a process audit, and the observations noted on site was specific to determining operational conditions and/or challenges with the treatment of the source water.

Table 2-1: Summary of Novo Pump Station Key Parameters

Unit process	Equipment on site	Site observations
Inlet structure and chemical dosing		
	Single inlet pipe with sampling tap. Discharge into first mixing chamber where polymer is dosed	The pipe is in good condition up to the isolation valve. A flow meter is installed on the inlet of the pipe.
	Polymer (Ultrafloc U3500) is dosed directly into the chamber with an underflow to the next chamber.	During inspection there was no power and the dosing of polymer could not be observed.
		It would appear that the chamber is not regularly cleaned and some buildup is noted on the sides of the chamber. Minor etching is observed with exposed aggregate in the chamber.
	Two storage tanks within bunds and two dosing pumps installed for polymer dosing.	The dosing rates for the treatment works are recorded by the process controllers which provides good insight into the effectiveness of this chemical on site. Data will be used to supplement the water quality analysis and treatability testing
	Lime dosing facility is available on site. It consists of a bag loader and mixing bowl. The dosing chamber is equipped with a top entry mixer	Lime dosing is currently not being used. Process controllers report that the water stability is good and pH levels within preferred ranges. Water quality data will be scrutinized to confirm this claim.

Unit process	Equipment on site	Site observations
Clarification		
	ked in build-up, suspected to be overdosing of polymer.	Although the bridge is old, it is fairly well maintained and operational. Site personnel made on-site adjustments to improve the operation of the unit. Centre mixers were caked in build-up, suspected to be overdosing of polymer. There is a visible layer on the water surface, futher suggesting overdosing of polymer.
	Clarifier is equipped with a concrete weir and launder and a single draw off point feeding water to the filters.	There is evidence of etching on the concrete launder. The weirs are not cleaned and this might result in carry over of debris. Overall quality of the water appeared to be fair.
Intermediate chlorination		
	Chloring dosing point upstream of filtration. Dosing is directly into the feed pipe to the filters.	Dosing point appeared to be in fair condition. The condition of the pipework internals could not be determined. There is no post disinfection on the works. Process controllers reported sufficient chlorine residual through filters.
	Gas chlorination system used on site.	It is reported that chemical procurement is a challenge. Chlorine dosing equipment is still in good condition.
Filtration		
	Two rapid gravity sand filters with single media. Underdrain system is a false floor and the plant utilize sequential air water backwashing.	Backwashing could not be performed during inspection due to loadshedding. Both filters reported operational and providing good quality water. A film was observed on the surface of the filters suggesting overdosing of polymer, and the carry over from the clarifier.

Unit process	Equipment on site	Site observations
	Filter gallery equipped with isolation valves under water level.	Evidence of damage to chamber tiles. Valve placed under water level could fail prematurely. Water quality appeared to be good and clear. Water quality results will be scrutinized to determine the quality of the filtered water.
	Post chlorination (chip dosing) installed at the filter outlet.	If the gas chlorination system fails, the plant is able to dose chlorine into the filter outlet water upstream of the clear water tank. It is reported that this system is not generally in use.
Pump station		
	Three high lift pumps on the northern side of the pump station.	Pumps are in good condition and well maintained. Process controllers reported that they do not have major issues with pumps and water is not aggressive causing premature failure.
	Two Backwash pumps drawing water directly from the clear water tank.	Pumps are old but reported to be in good condition and well maintained.
	Single blower for air scour during backwashing	The plant has no redundancy for this unit. It was reported to be operational.

Unit process	Equipment on site	Site observations
Residuals handling		
	Two concrete sludge ponds with a return pump station	Ponds were new and in good condition. One pond was being cleaned during the inspection. Evidence of dried sludge piled outside of the plant perimeter.
Overall observations		

- The treatment plant has a simple process which is effective for the type of water treated. There were no reports of algae, but the plant does not test for chlorophyll-a to confirm.
- Although the plant does not use lime at present, there is evidence of etching on the inlet processes. This will be further investigated during the design process.
- According to the process controllers, the polymer used is effective for the source water.
- Overdosing of polymer is a concern, as data for collected on site might not be representative of the actual dosing requirements. No on-site jar testing is done to confirm dosing.
- Residuals handling system is simple and effective.
- No post disinfection is done on site, as it is reported the residual chlorine from intermediate dosing is sufficient. Distribution from the site is not extensive, and VCW maintains that the residual is sufficient for the bulk distribution from site.

3 Conclusions

The plant is functional and as per water quality data collected, performing within requirements. The observations and information collected from site will be used to supplement the design process. For reference, the water quality data collected from site and summarised is indicated below.

Parameter	Units	No. of analyses	5th percentile Raw Water Operational Data	50th percentile Raw Water Operational Data	95th percentile Raw Water Operational Data	SANS 241: 2015 and DWS/WHO Standards
Turbidity*	NTU	50	0.18	0.40	***4.25***	≤ 1
Colour	mg/ℓ as Pt	3	0.10	1.00	1.90	≤ 15
TDS	mg/ℓ	221	-1.73	-0.358	131.95	≤ 1200
Conductivity	mS/m	223	13.76	18.70	23.27	≤ 170
pH	[-]	223	6.51	7.60	8.18	≥ 5 to ≤ 9.7
Total Alkalinity	mg $CaCO_3$/ℓ	223	43.72	62.90	84.15	~40-120
Fluoride	mg/ℓ	220	0.05	0.17	0.31	≤ 1.5
Ammonia	mg/ℓ	220	0.02	0.02	0.14	≤ 1.5
Potassium	mg/ℓ	220	0.98	1.26	2.47	≤ 50
Sodium	mg/ℓ	220	4.00	5.68	7.45	≤ 200
Zinc	mg/ℓ	71	0.00	0.01	0.08	≤ 5
Calcium	mg/ℓ	220	***14.90***	20.00	29.80	≥ 16
Iron	mg/ℓ	117	0.01	0.10	0.10	≤ 0.3
Manganese	mg/ℓ	117	0.00	0.01	0.02	≤ 0.1
Magnesium	mg/ℓ	220	4.70	6.26	7.50	≤ 30
Chloride	mg/ℓ	219	1.50	4.75	8.43	≤ 300
Chlorine, free as Cl_2	mg/ℓ	180	1	1.20	2.60	≥ 0.5; ≤ 5
Nitrate as NO_3 - N	mg/ℓ	223	0.23	0.57	0.98	≤ 11
Nitrite as NO_2 - N	mg/ℓ	16	0.01	0.01	0.01	≤ 0.9
Sulphate as SO_4^{2-}	mg/ℓ	220	4.70	9.97	44.72	≤ 250
Ortho-Phosphate PO_4	mg/ℓ	217	0.00	0.02	0.10	
Calcium Hardness (calculated from above)	mg/ℓ as $CaCO_3$	220	14.90	20.00	29.80	
Magnesium Hardness (calculated from above)	mg/ℓ as $CaCO_3$	170	16.30	21.16	25.61	
Total Hardness (calculated from above)	mg/ℓ as $CaCO_3$	170	58.83	78.71	99.00	≤ 150
Langelier Index	-	169	***-1.88***	***-0.54***	-0.01	~ 0
Ryznar Index	-	169	***8.23***	***8.89***	***10.24***	6.5 – 7.0
Escherichia coli	MPN or CFU per 100 mL	155	0.00	0.00	0.00	0

Parameter	Units	No. of analyses	5th percentile Raw Water Operational Data	50th percentile Raw Water Operational Data	95th percentile Raw Water Operational Data	SANS 241: 2015 and DWS/WHO Standards
Heterotrophic plate count (HPC)	CFU	186	0.00	0.00	0.00	≤ 1000
Total coliforms	CFU	186	0.00	0.00	0.00	≤ 10
Calcium Carbonate Precipitation Potential (CCPP) (calculated from above)[2]	mg $CaCO_3$/ℓ		No data	No data	No data	2 to 5
TOC	mg/ℓ	49	1.58	2.83	3.78	<10
DOC	mg/ℓ	1	-	3.33	-	<10