108/114/12/R5

UMKHOMAZI WATER PROJECT

MODULE 3 – POTABLE WATER MODULE

Detailed Feasibility Study

Water Treatment Works Conceptual Design

Revision 02

October 2015

UMGENI
WATER•AMANZI

Planning Services
Engineering & Scientific Services
Umgeni Water

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UMGENI WATER **OCTOBER 2015**

uMkhomazi Water Project

Detailed Feasibility Study - Water Treatment Works Conceptual Design

Report No. 108/114/12/R5

Prepared by Knight Piésold Consulting

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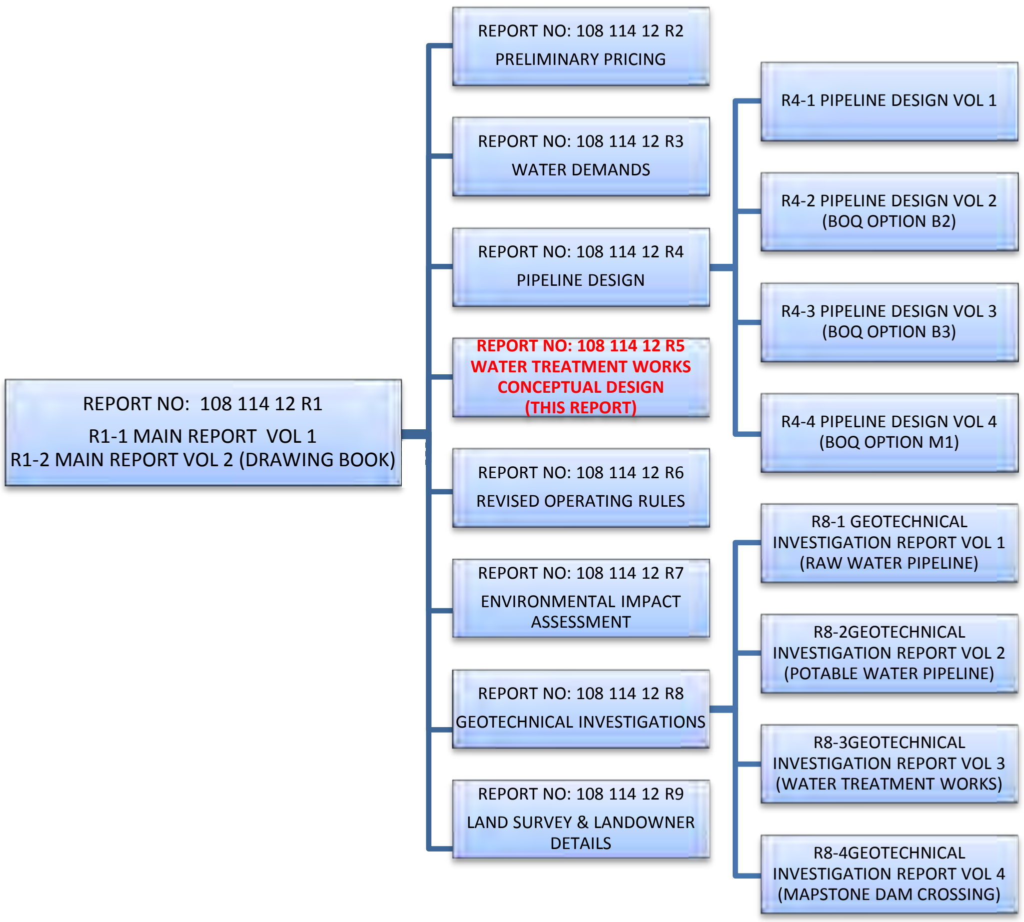
UMKHOMAZI WATER PROJECT

MODULE 3 – POTABLE WATER MODULE

Structure of Suite of Reports



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1 Introduction

The projected increase in water demand in the greater Durban region is predicted to exceed supply from all current sources within the next ten years. Several new water supply schemes are presently being investigated by Umgeni Water (UW) and eThekwini Water and Sanitation (EWS) as possible solutions to the predicted supply shortage. One of the schemes under investigation is the proposed uMkhomazi Water Project (uMWP-1).

The first phase of the proposed uMWP-1 comprises the Smithfield Dam, a raw water tunnel to Baynesfield, the Langa balancing dam at Baynesfield, a water treatment works (WTW) in the Baynesfield area and a potable water pipeline from the WTW to Umlaas Road, where it connects to the Western Aqueduct via UW's '57 Pipeline. The proposed scheme is depicted in **Figure 1**.

The feasibility investigations for this project have been split into three Modules.

Module 1 covers the raw water component of the study, i.e. Smithfield Dam, the raw water tunnels from Smithfield Dam to Baynesfield, a balancing dam in the Baynesfield area and a raw water pipeline from the tunnel outlet to the proposed WTW.

Module 2 covers the Environmental Impact Assessment for Modules 1 and 3.

Module 3 covers the potable water component of the study, i.e. a potable water treatment works (WTW), potable water storage reservoir and potable water pipelines from the WTW to Umlaas Road, where the proposed new pipelines connect into the existing '57 Pipeline owned by Umgeni Water.

The project proponents are:

- **Module 1 – Raw Water Module**: Department of Water and Sanitation
- **Module 2 – Environmental Impact Assessment**: Department of Water and Sanitation (Raw Water Module) and Umgeni Water (Potable Water Module).
- **Module 3 – Potable Water Module**: Umgeni Water

Umgeni Water appointed Knight Piésold in July 2012 to carry out a Detailed Feasibility Study for Module 3 of the proposed uMkhomazi Water project, i.e. the Potable Water Module. This report covers the Water Treatment Works Conceptual Design component of the Module 3 study.

Figure 1: Proposed uMkhomazi Water Project

2 Background

Detailed information on the background and need for the uMWP-1 project is included in Report No. **108/114/12/R1 – Main Report**.

3 Scope of report

Module 3 of the uMWP comprises several different reports as depicted in the structure of the suite of reports in the opening pages of this report. This report, No. 108/114/12/R5, is the fifth in a suite of nine reports that make up the uMWP-1 Module 3 study and focuses on the water treatment works and covers the following:

- Raw water quality sampling and testing
- WTW conceptual design
- WTW plant layouts
- WTW hydraulic profile
- WTW capital and operation cost estimates

The site selection for the WTW is integral with the raw and potable water pipeline routing. This portion of the WTW study is therefore covered in **Report No. 108/114/12/R5 Pipeline Design**.

4 Water Demand and Phased WTW Capacity Implementation

A 30-year demand projection as developed in **Report No. 108/114/12/R3** is reflected in **Figure 2**. The Low Road scenario was adopted for the purposes of sizing the potable water infrastructure for the uMWP-1.

The projected potable water demands and WTW phasing developed in the above report were taken into consideration in determining the overall treatment capacity for the proposed WTW and the phased implementation thereof.

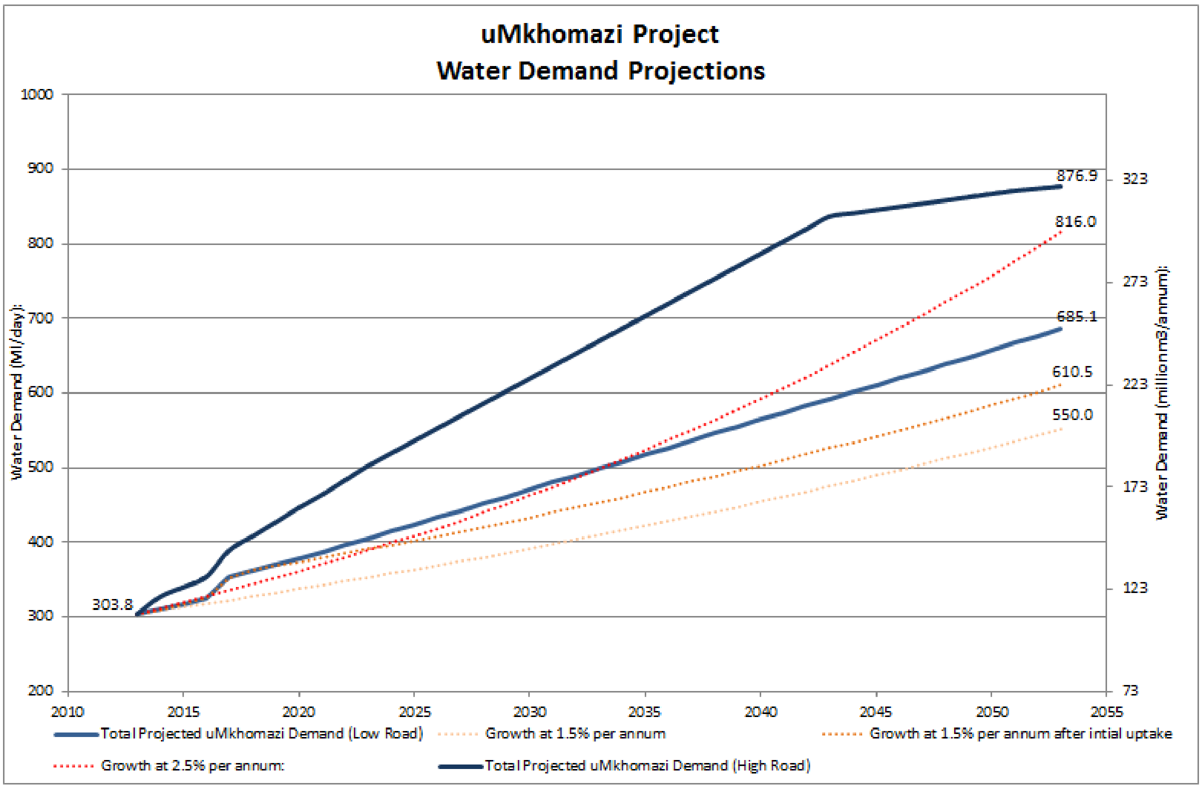


Figure 2: uMkhomazi Project – Total water demand projections

Based on the findings of a Department of Water & Sanitation report titled "*The uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study, Raw Water - Water Resources Planning Model Report*" relating to the risk of failure in the Mgeni system in relation to the utilisation of the Smithfield Dam, the uMWP-1 Project Management Committee (PMC) adopted a capacity of 500 Ml/d for the first phase of the WTW. A further 125 Ml/d increase to 625 Ml/d would be required in 2044. This proposed phasing is represented in **Figure 3** by the curve titled **PROPOSED WTW CAPACITY (BY AGREEMENT BETWEEN UW & DWS)**.

An initial capacity of 500 Ml/d will make allowance for one spare train of 125 Ml/d upon commissioning as requested by UW (Subramanian, 2014).

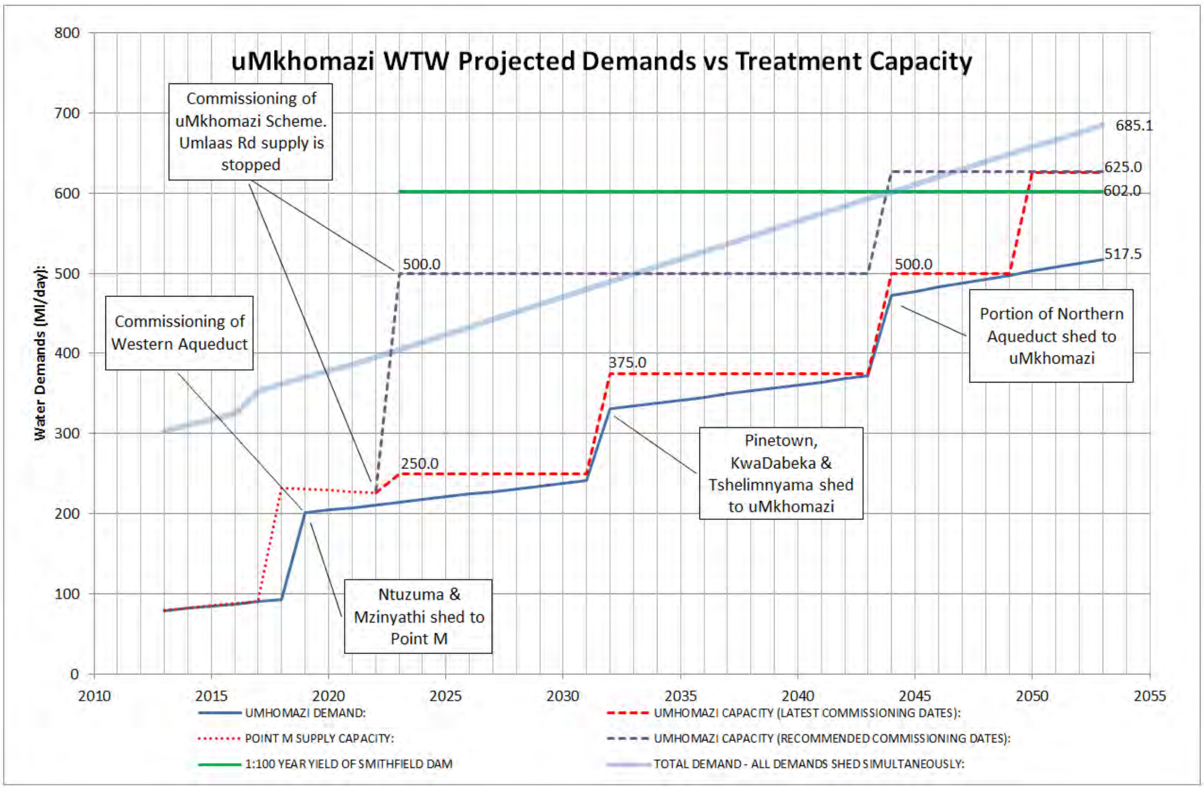


Figure 3: uMkhomazi Project - Water demand projections and treatment capacity requirements

This Conceptual Study for the new uMkhomazi WTW was therefore based on providing an initial treatment capacity of 500 Ml/d in four equal trains of 125 Ml/d each, which will be available at start-up in 2023. This will be undertaken under Phase 1 of the project and will provide sufficient treatment capacity to meet the scheme's projected water demands up to the year 2043. The initial 500 Ml/d WTW represents a partial development of the full uMWP-1 capacity. The WTW will reach the full capacity of the uMWP-1 in 2044 when the total capacity is planned to be increased to 625 Ml/d to meet the projected water demands in that year.

Phase 2 of the uMWP will entail increasing capacity of the plant to 1 250 Ml/d. This capacity allows for full development of the available yield of 1 020 Ml/d plus a 20% allowance for taking units out of operation for maintenance, servicing and cleaning and then rounding to 1 250 Ml/d to allow for expansion in standard trains of 125 Ml/d each. The fully developed uMWP will therefore have a WTW consisting of ten trains in parallel, each with a treatment capacity of 125 Ml/d and is planned to be constructed in phases as required.

The planning, process flow diagram and site layout drawings in this report incorporate the cumulative plant capacity for Phases 1 & 2 of 1 250 Ml/d. Cost estimates have been provided for the 500 Ml/d as well as the 1250 Ml/d WTW.

5 Raw Water Characterisation

The quality of the raw water that will supply the WTW was monitored by UW over a six-month period and typical minimum, median, average, 95th percentile and maximum values for crucial contaminants were established (Hodgson, 2013). The raw water quality information, together with basic laboratory tests such as flocculation and settling tests has been used to evaluate different unit processes suitable for treating the raw water to potable water quality.

Samples of the raw water that will be treated at the proposed WTW were analysed and characterised with regards to its water quality and physical/chemical parameters. Whereas the former included chemical and biological analyses of the main quality parameters, the latter included mainly flocculation, sedimentation and filtration tests conducted on the raw water, and thickening and dewatering of the sludge that accumulates during the treatment process.

5.1 Water Quality Assessment

The raw water infrastructure for the uMWP-1 comprises the Smithfield Dam, a raw water tunnel from Smithfield Dam to the Baynesfield area, the Langa Balancing Dam at Baynesfield and a raw water pipeline from the tunnel outlet to the proposed WTW. Raw water from Smithfield Dam will be transferred via the tunnel directly into the raw water pipeline which in turn will deliver raw water to the WTW. Langa Balancing Dam provides emergency storage but will only be used during a tunnel shut down for maintenance or repairs.

A detailed water quality assessment for the uMWP was undertaken by UW and is summarized in a report by Hodgson (2013). For characterisation of the raw water, the main sources of inflow to the Smithfield and Langa dams have been considered, i.e. the uMkhomazi River that serves the uMkhomazi Catchment Area and discharges into Smithfield Dam and the uMlazi River that serves the uMlazi Catchment Area and discharges into the Langa Dam.

Unless there is a tunnel shutdown, the water arriving at the WTW will be from the uMkhomazi River via the Smithfield Dam.

Smithfield Dam Inflow: There were two sampling points that best represented the expected raw water quality at the inflow to this dam:

- uMkhomazi Smithfield Inflow, sampling the uMkhomazi River at Lundy's Hill Weir. This will be the main source of supply for the envisaged new dam and data from March 1996 to date is available;
- Luhane Smithfield Inflow. Data collected from March 2007 to present is available.

Baynesfield Dam Inflow: Data collection only started in October 2012 and six samples have been considered to date. Therefore, the analyses must be viewed with caution. However, the samples were taken in the above average summer rainfall period and will generally be biased towards elevated results (Hodgson, 2013).

The Water Quality Assessment by Hodgson (2013) was used to only identify main constituents that have to be taken into consideration when defining a water treatment process for treating the raw water to reliably achieve product water conforming to UW's potable water standards. From the above report, minimum, maximum and average values for the different contaminants were defined using summary statistics from the water quality data as obtained at Lundy's Hill Weir, Luhane Smithfield Inflow and Baynesfield Dam Inflow. Whereas the water treatment plant needs to be able to cope with maximum expected contaminant levels, annual chemical consumption and operating costs are based on average contaminant levels.

Cognisance was taken of the fact that significant reductions in certain contaminants can be observed with impoundment. For example, the new Smithfield Dam is planned with retention time not less than 0.3 years and 11.6 km impounded river length at full supply level. This will reduce turbidity, suspended solids, iron, manganese and total phosphorus values by at least 50% (Hodgson, 2013).

Table 1 was drawn up using data extracted from the Water Quality Assessment Report (Hodgson, 2013). This table depicts minimum, average and maximum contaminant levels only for constituents that were identified from the Report that need to be considered for the design of the new water treatment plant. Final design values were then defined, taking into account reductions in certain parameters due to impoundment but also increases in other parameters due to eutrophication. Where Hodgson gave statistical predictions for a change in raw water parameters taking into account the effect of an integrated impoundment with retention time of 0.3 years, as envisaged for the proposed Smithfield Dam, these values were taken up in the table for plant design purposes.

Table 1: Main Parameters Considered for Design of New uMkhomazi WTW

Contaminant	Units	RAW WATER INFLOW*			PLANT DESIGN VALUES		
		Min.	Median	95th Perc.	Min.	Av.	Max.
Algal Count	Cells/ml	0	205	6 390	0	1147	6 400
Alkalinity	mg/l $CaCO_3$	12.9	31.6	47.7	10	32.6	48
Calcium	mg/l as Ca	2.0	6.3	9.0	2	6.3	9.0
Chlorophyll a	µg/l	0.5	0.5	2.2	0.5	0.8	2.2
E.coli	Count/100 ml	288	343	2 500	280	560	2 500
Iron	mg/l as Fe	0.6	1.5	2.9	0.6	1.6	3.0
Magnesium	mg/l as Mg	1.4	2.7	8.4	1.5	2.7	8.5
Manganese	mg/l as Mn	0	0.05	0.14	0	0.06	0.14
pH		7.1	7.8	8.4	7.1	7.8	8.4
Soluble Organic Carbon	mg/l as C	1.2	2.1	3.5	1.2	2.1	3.5
Suspended Solids	mg/l	7.2	25.4	267	7.2	91.6	270
Total Hardness	mg/l as $CaCO_3$	14.8	25.9	35.7	15	27	36
Total Organic Carbon	mg/l as C	1.3	2.4	4.4	1.3	2.8	4.5
Total Phosphorus	µg/l as P	16.4	70.1	133.9	16	69	135
Turbidity	NTU	13.6	25.8	328	14	91	330

* The raw water inflow value reflects the highest value from the three inflow sources under consideration.

When choosing the most appropriate unit processes to treat the above raw water, the following water quality aspects, highlighted in the Water Quality Assessment Report by Hodgson (2013), need to be taken into consideration:

- Significant elevated turbidities can be expected to occur occasionally at the abstraction point due to high peak inflow values and under severe storm conditions. These turbidity peaks may not be sufficiently reduced in the envisaged Smithfield Dam because of the relatively low retention period of 0.3 years;
- The envisaged impoundment size is, however, sufficiently large to significantly reduce suspended material, notably silt particles, which will be removed by sedimentation;
- The bacteriological quality of the inflow will also improve due to in-dam processes when an impoundment as envisaged is provided;
- The envisaged impoundment (Smithfield Dam) is anticipated to be mesotrophic, i.e. enriched with nutrients, which will result in occasional blooms of nuisance algal species. This will initially be manageable with proper dam operation such as spilling, scouring and abstracting raw water from the aerobic zone for treatment in the WTW. However, raw water quality in the impoundment may deteriorate in future due to increased nutrient discharge into the catchment area of the river. This will result in the envisaged dam becoming eutrophic and will require treatment in a WTW to reduce mainly organic carbon and microbial by-products. The soluble organic carbon, at 1.2 mg/l as minimum for the three sources, is already over the ideal limit of 1.0 mg/l for drinking water;
- Thermal stratification during summer with dam turnover (de-stratification) in autumn is highly likely. This will result in elevated metal concentrations, notably iron and manganese, which will be liberated from the sediments under anoxic conditions and must be removed in the treatment plant;
- The raw water is very soft with average Total Hardness of only 27 mg/l as $CaCO_3$. Untreated, the final water will be very aggressive and will therefore require lime stabilisation during treatment.

The raw water quality data in this section has been used to define appropriate unit processes and a treatment train for the envisaged uMkhomazi WTW.

5.2 Physical-Chemical Assessment

Laboratory tests to simulate physical-chemical processes were conducted on the main raw water sources that will feed the envisaged uMkhomazi WTW, being the uMkhomazi River (sample taken at Lundy's Hill Weir), Luhane Smithfield inflow and Baynesfield Dam. Stabilisation, iron and manganese removal, turbidity reduction and disinfection to achieve potable water standards is addressed in this section. Sludge dewatering and thickening is also addressed, since it is anticipated that large volumes of clarifier underflow and filter washwater will be produced by the new WTW.

5.2.1 Stabilisation

Since the raw water is soft and thus aggressive (**Section 5.1**), lime stabilisation would in all cases be required. UW uses commercially available white or brown lime in their current treatment plants and this was therefore also used in the beaker tests. The required lime dosage for stabilisation to a CCPP of 1 mg/l was determined using the Stasoft III (Friend and Loewenthal, 1992) computer program and was found to be 10 mg/l of white lime [78% (m/m) $Ca(OH)_2$]. The latter was added first to all samples before standard beaker tests were performed.

5.2.2 Iron (Fe) and manganese (Mn) removal

These two metal ions are found to occur naturally in concentrations above aesthetically acceptable levels in raw waters throughout the Kwa-Zulu Natal Midlands. Also, Fe and Mn accumulates in lower layers of impoundments and are then released at relatively high concentrations when thermal stratification and inversions occur with seasonal changes as was described in **Section 5.1**. **Table 1** indicates that the envisaged average raw water Fe and Mn concentrations into the new WTW will be 0.8 mg/l and 0.1 mg/l respectively, which exceeds drinking water standards and therefore need to be removed. Potassium permanganate ($KMnO_4$), a strong oxidant, is very effective in precipitating both Fe and Mn and generates manganic oxides, which have a further accelerating effect on manganese removal (Barnes & Wilson, 1983):

$$3\,Fe^{2+} + MnO_4^- + 7\,H_2O \rightarrow 3\,Fe(OH)_3 + MnO_2 + 5\,H^+ \qquad \ldots\ldots 1$$

$$3\,Mn^{2+} + 2\,MnO_4^- + 2\,H_2O \rightarrow 5\,MnO_2 + 4\,H^+ \qquad \ldots\ldots 2$$

Fe removal is very effective already at pH values above 7, while Mn removal requires the pH to be around 8. Since the average raw water pH is expected to be around 7.8, it will increase to above 8 due to lime addition for stabilisation, if latter is added at the beginning of the treatment process and before potassium permanganate is added. For Fe and Mn precipitation at average concentrations of respectively 0.8 mg/l and 0.1 mg/l in the raw water, the required $KMnO_4$ dosage will be ca 1 mg/l.

5.2.3 Turbidity reduction

Standard beaker tests (Rencken, 1997) to establish basic physical design parameters for a new potable water treatment plant for flocculation, settling and filtration were conducted on said raw water samples from 22 to 31 October 2012 by Dr Lempert.

Alum [$Al_2(SO_4)_3.18\ H_2O$], ferric chloride ($FeCl_3$) and many different poly-electrolytes were tested as coagulants and flocculants. Turbidity of the uMkhomazi River and Luhane Smithfield inflow sample was moderately high at 255 NTU and 162 NTU, respectively, due to clay-bearing colloidal matter in the raw water. However, turbidity of the Baynesfield Dam raw water sample was very low at 3.2 NTU. Tests with especially latter water were very important to assess the difficulties of treating low turbidity waters from a typical impoundment from that area, since median raw water turbidity at the new, envisaged Smithfield Dam with a retention time of 0.3 years is predicted to be only 6.7 NTU, although 10 NTU was allowed for estimating purposes in this study.

It was found that alum, dosed either separately or in combination with a specific polyelectrolyte, Ultrafloc® U3500 (from NCP), gave good flocculation results (**Table 2**). However, the floc that is formed is light and slow to settle and, when treating medium to high turbidity raw waters from the uMkhomazi River and Luhane Smithfield inflow, the target turbidities of below 10 NTU after settling and 0.5 NTU after filtration were in most cases not reached when dosing only alum. Poly had to be added to achieve latter. Alum dosages could be reduced by 5 mg/l for each 1 mg/l of Ultrafloc® U3500 added.

On the low turbidity raw water from Baynesfield Dam alum addition resulted in a very light floc that did not want to settle, although settling rates improved when alum was used in combination with Ultrafloc® U3500. Substantial improvements in settling rates were seen when either Bentonite or fine sand (ES < 150 µ) was added to augment coagulation and settling. Latter two constituents are typically used in the water field when the coagulating water contains little mineral turbidity, and addition thereof results in a heavier floc being formed that allows higher settling rates in a subsequent clarifier (AWWA, 1999). UW already uses Bentonite at their Midmar, Wiggins and Durban Heights WTW for this purpose. **Table 2** summarises the results regarding most effective chemical(s) or additives used, optimum dosing rates and final water turbidity obtained when conducting these beaker tests.

Table 2: Optimum Beaker Test Results Obtained With Various Chemicals and Additives

	Units	uMkhomazi River (Lundy's Hill Weir)			Luhane Smithfield Inflow			Baynesfield Dam Impoundment		
		A	B	C	A	B	C	A	B	C
Chemical Added										
Lime	mg/l *	10	10	10	10	10	10	10	10	10
Alum	mg/l *	25	20	18	40	35	35	15	15	15
Poly – U3500®	mg/l *	0	1	1	0	2	1	1	1	1
Bentonite	mg/l	0	0	0	0	0	0	0	5	0
Fine sand(ES < 150 µ)	mg/l	0	0	3	0	0	3	0	0	3
Turbidities:										
Raw Water	NTU	255	255	255	162	162	162	3.2	3.2	3.2
After 10 min Settling	NTU	10.6	8.8	8.9	7.8	3.7	4.4	0.7	0.7	1.1
After Filtration	NTU	0.6	0.5	0.3	0.5	0.4	0.4	0.1	0.1	0.1
Observed settling rate		poor	fair	good	poor	fair	good	poor	good	good

* mg/l as commercially delivered product:

- Lime = 78% (m/m) $Ca(OH)_2$
- Alum = 15-22% (kg/l) Aluminium sulphate [$Al_2(SO_4)_3.18\ H_2O$]

The above results obtained with alum, poly and Bentonite addition were compared with dosing rates as applied at UW's Midmar and Wiggins WTW when raw waters with similar turbidities were treated (Mdlalose, Thompson, Trollip, 2013). It was found that they conform fully to the chemical usage at these large-scale plants and are therefore a reliable basis to calculate envisaged chemical demand and operational costs for the new uMkhomazi WTW.

5.2.4 Enhanced flocculation and settling

To be able to employ high-rate clarification processes, a heavier floc is required so that higher upflow rates can be employed in the clarifier and thus allows a clarifier to be provided with smaller footprint area. A ballasted floc is typically obtained when adding Bentonite or fine sand (ES < 150 µ) during flocculation as explained in the previous section. Whereas UW has been using Bentonite already for many years, even better results are nowadays achieved in Europe and North America when fine sand is added. Latter was also tested on the raw water samples in beaker tests.

Figure 4 and **Figure 5** show pictures taken during a typical beaker test that was conducted on low turbidity raw water from the Baynesfield Dam to assess flocculation and settling when using a ballasted floc. Whereas good floc forming was observed [**Figure 4**, first beaker] when adding 15 mg/l alum and 1 mg/l U3500® the floc did not settle well [**Figure 5**, first beaker]. Enhanced flocculation and improved settling was observed when 5 mg/l Bentonite was added [**Figure 4** and **Figure 5**, middle beaker]. When fine sand was added instead of Bentonite, flocculation improved and substantially faster settling rates were observed as could be seen from **Figure 4** and **Figure 5**, last beaker. This aspect will be further discussed when dealing with ultra high-rate clarification/settling processes in **Section 6.3.3** of this report. However, whereas bentonite addition has been allowed for in the Conceptual Study, the aspect of ballasted floc with sand addition was not further exploited, but can be considered by UW in future to increase throughput of their clarifiers.

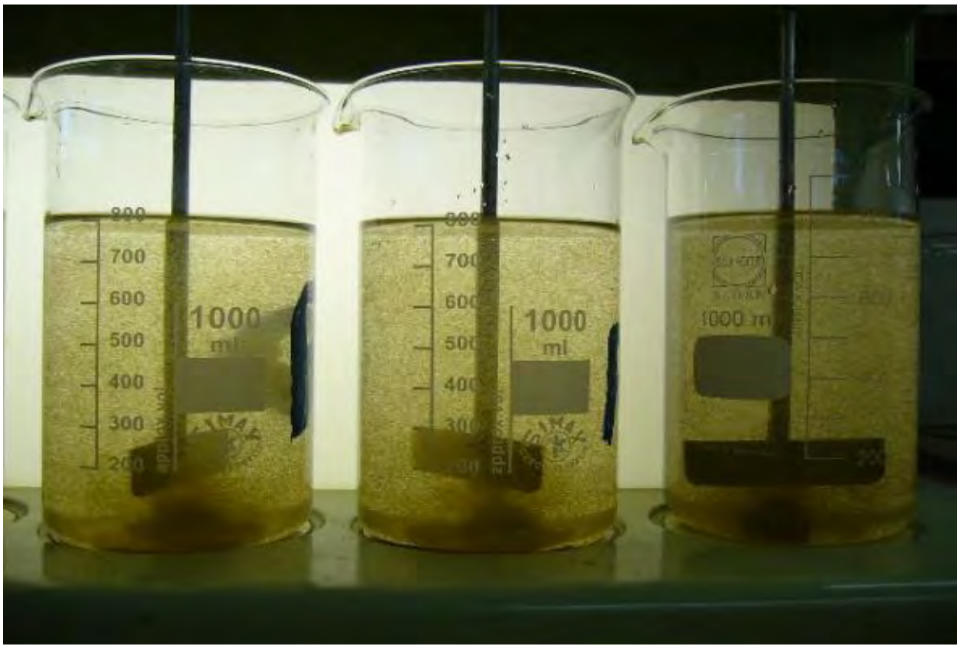


Figure 4: Beaker Test During Flocculation

a) Beaker test during flocculation (after 5 min. of slow stirring):
   - First jar (left) – 15 mg/l alum plus 1 mg/l poly;
   - Second jar (middle) – 15 mg/l alum plus 1 mg/l poly plus 5 mg/l Bentonite;
   - Third jar (right) – 15 mg/l alum plus 1 mg/l poly plus 3 mg/l fine sand.
     Note aglomoration of floc already in centre, at bottom of jar.

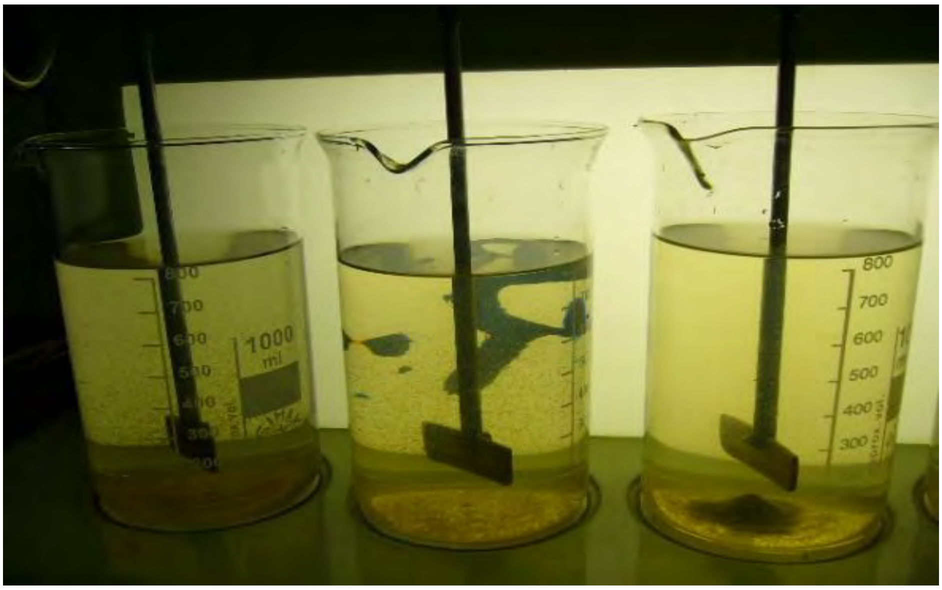


Figure 5: Beaker test showing effect of ballasted floc

b) The same beaker test during settling (after 3 min.). Note hovering floc in left jar and compact floc in right jar, bottom centre .

5.2.5 Disinfection

Chlorine gas is currently used for disinfection as pre and post-chlorination chemical on all major treatment works of UW. For ease of operation and logistical reasons, UW (Thompson, 2013) requested that this chemical is also used at the new uMkhomazi WTW.

5.2.6 Sludge thickening and dewatering

Batch sedimentation and thickening tests as described by Rencken (as in: Van Duuren, 1997) were conducted on sludge collected from beaker tests on uMkhomazi River raw water. Only alum (20 mg/l) and poly (1 mg/l U3500) was used for flocculating the raw water, as per **Table 2** in Section 1.3.2.4 above, whereas only poly (U3500) was used for subsequent dewatering, which would later take place in a gravity thickener and centrifuge.

The concentration of sludge that was produced during sedimentation in the standard beaker tests was collected and found to be 1.75% (m/m) DS. Latter was then dosed with Ultrafloc® U3500, mixed up and flocculated before left to settle. The sludge dewatered well and settled fast when adding poly (U3500) at a concentration of 0.8 kg/t DS. **Figure 6** shows the well-defined water/sludge interface, with a very clear liquid phase that could be observed after 12 minutes settling.

With gravity thickening a final sludge concentration of 4.9 % (m/m) DS could be achieved.

The above dewatering/thickening results are in line with current, large-scale operational results obtained at UW's Midmar and Wiggins WTW, where pre-clarification followed by poly addition and sludge thickening is used to obtain a dewatered sludge of 4% to 5% (m/m) DS. Latter is then again dosed with poly before fed to centrifuges for mechanical dewatering to 30 to 33% (m/m) (Mdlalose, Thompson, Trollip, 2013).

Figure 6: Typical sludge settling test

5.2.7 Overall chemical requirements

Based on the above findings, chemicals that will be used at the new uMkhomazi WTW as well as the annual consumption for a 500 Ml/d (Phase 1) treatment works have been established based on the above water quality and physical-chemical assessments. **Table 3** reflects envisaged minimum, average and maximum chemical dosages that the new WTW will have to apply to treat the raw water to potable water standard. These dosages are also very much in line with what UW is currently using at their other plants dealing with similar river water, e.g. Midmar, Wiggins and Durban Heights WTW (Mdlalose, Thompson, Trollip, 2013).

Table 3: Envisaged Chemicals and Application Range for Phases 1 (500 ML/d) and 2 (1 250 ML/d)

Chemical/Additive	Units	Envisaged Application Range (mg/l)			Phase 1 Annual Average Consumption (Ton/year)	Phase 2 Annual Average Consumption (Ton/year)
		Min.	Av.	Max.		
Alum	mg/l *	10	15	25	2 738	6 844
Bentonite	mg/l*	0	3	5	548	1 369
Lime						
- Stabilisation	mg/l *	8	10	16	1 825	4 563
- Sludge treatment	mg/l sludge	120	150	180	126	318
Chlorine (gas):						
- Pre-chlorination	mg/l as Cl_2	1.0	1.5	3.0	274	684
- Final chlorination	mg/l as Cl_2	2.0	2.0	2.0	365	913
Poly electrolyte (U3500®):						
- Flocculation	mg/l *	0	1	2	183	456
- Sludge treatment	kg/T DS	4.5	9	13.5	1 188	2 988
Potassium Permanganate	mg/l as $KMnO_4$	0.6	1.0	1.6	183	456

* mg/l as commercially delivered product

For calculations regarding annual chemical consumption and operational costs (Section 2), only the average values in **Table 3** were considered.

6 Process and Plant Design

6.1 Basic Design Philosophy

The general and specific design aspects that have been taken into consideration when selecting specific unit processes for the uMkhomazi WTW include:

6.1.1 Raw water source

The proposed WTW must cater for all typical river water conditions and changes in raw water quality due to seasonal changes in inflow, stratification and inversion of a dam. It is, however, envisaged that the impact thereof will be smoothened through optimum dam management, such as regular dam scouring and spilling, and controlling abstraction depth to ensure that only water from the aerobic zone will be supplied to the new WTW. The rather short impoundment retention time of 0.3 years, as currently envisaged for the Smithfield Dam, will result in more extreme fluctuations in raw water quality reaching the plant than, for example at UW's Midmar WTW, whose supply dam has a 1.25 year retention time.

6.1.2 Operation and maintenance

Emphasis was placed on simplicity of operation, ease of maintenance and minimal process adjustments, coupled to familiar processes as also used at other plants operated by UW personnel. In order not to complicate UW's existing WTW operations, it would be beneficial if the proposed WTW employed the same unit processes as the Midmar, Durban Heights and Hazelmere WTWs.

6.1.3 General design aspects

The following aspects have been taken into account for choosing a specific unit treatment process:

- The design is to include proven unit processes that are familiar to UW;
- Availability of electricity is limited and power is expensive – energy-intensive unit processes were therefore avoided where practically possible;
- Simplicity of operation and maintenance;
- Limited reliance on skilled personnel;
- Routine maintenance must be able to be performed by UW personnel or a South African based company;
- Duplication of critical equipment such as pumps and valves to ensure limited stocks of spares can be kept on site.

6.1.4 Specific design aspects

The Technical Feasibility Study, as Phase 1 of the uMkhomazi Water Project required specific attention to be given to the following important aspects for a new WTW:

6.1.4.1 *Small footprint.*

Whereas several locations have been identified as possible sites for the new WTW, all of these sites will require expropriating and compensating current land owners for their valuable, productive agricultural land and/or could negatively impact the scenic landscapes for which the Natal Midlands are well known. Public meetings conducted in October 2013 as part of the Environmental Impact

Assessment (EIA) for this project highlighted the necessity of minimizing the land area to be expropriated and reducing the overall size of the plant to reduce visibility thereof and to better blend in with the surrounding natural landscape. Reducing the footprint of a unit process substantially can only be achieved when employing high-rate technology. Therefore, even where conventional treatment processes were chosen, an in-depth investigation of latest, high-rate technology in that field was undertaken in order to reduce the overall footprint of the plant.

6.1.4.2 Limited head available

With reference to the flow of water through the WTW, all main processes from the head of works to the discharge of treated water from the potable water reservoir utilise gravity flow.

The WTW forms part of a larger gravity water supply system that originates at Smithfield Dam and ends at the terminal supply points of the Western Aqueduct pipeline. Based on the worst case scenario of 876.6 msl residual head at the raw water tunnel outlet (Badenhorst, 2014) and the requirement to maintain a residual head of 838 msl at the Umlaas Road tie-in (Doorgapershad, 2015), the maximum hydraulic loss that could be allowed through the WTW was 10 metres after allowing for friction losses in the raw and potable water pipelines as well as the lowest acceptable water level in the potable water reservoir.

6.2 Treatment Processes & Design Capacity for New WTW

Based on Water Quality (**Section 5.1**) and Physical-Chemical Assessment (**Section 5.2**) of the raw water it was decided to employ conventional water treatment processes as typically applied in river water treatment plants for the proposed uMWP WTW. The final water quality will comply with the SANS 241: 2011 Drinking Water Guidelines.

6.2.1 Basic treatment process train selected

The basic unit processes that were chosen and need to be incorporated will be:

- Chemical dosing, allowing for:
    - Oxidation of iron and manganese;
    - Stabilization;
    - Addition of a coagulant/flocculant;
    - Addition of a ballasting agent;
    - Chlorination – pre and post chlorination is required.
- Flash mixing and coagulation;
- Flocculation;
- Sedimentation;
- Filtration;
- Disinfection;
- Sludge dewatering and thickening.

The process schematic in **Figure 7** depicts the unit processes planned to be employed for the proposed WTW. Each process is further elaborated on individually in this report.

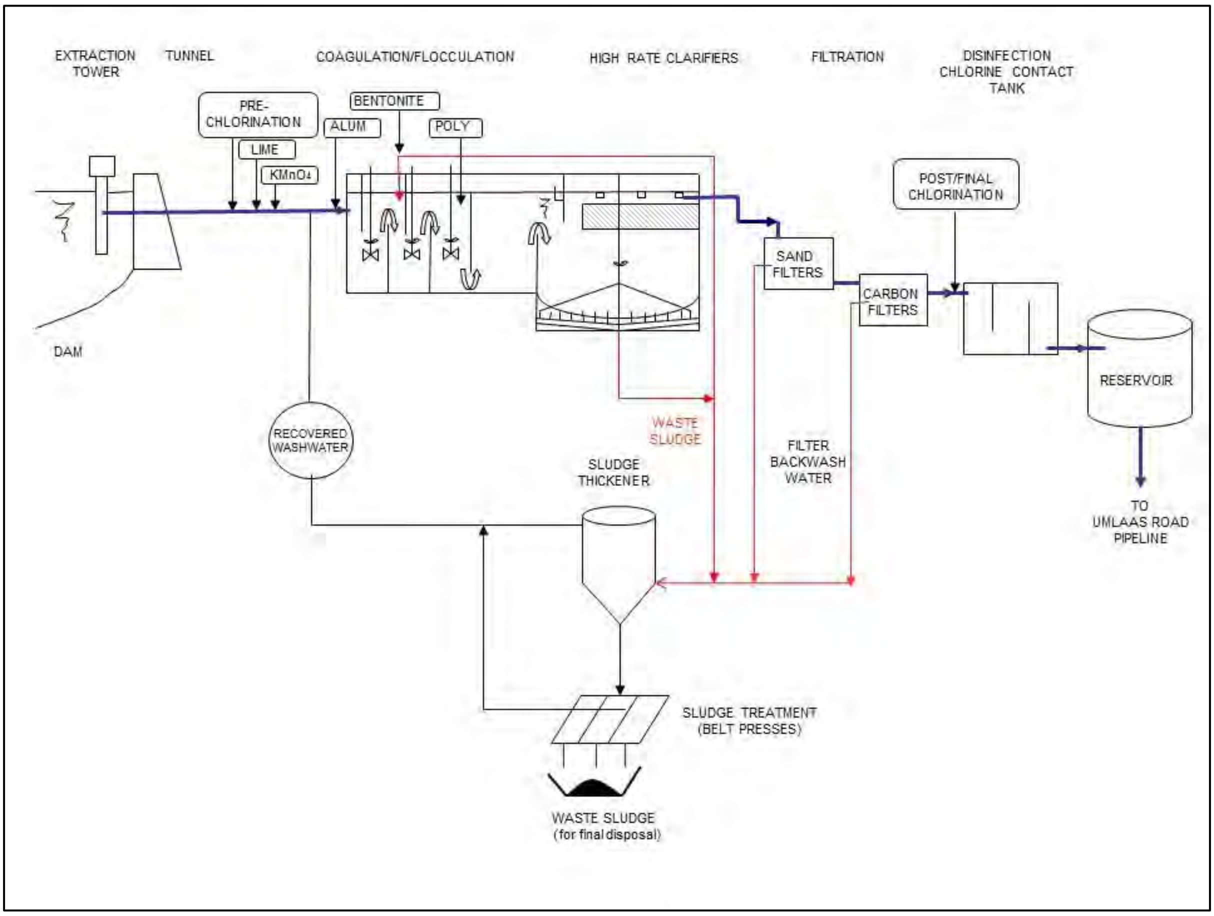


Figure 7: Process Schematic for the Proposed uMkhomazi WTW

6.2.2 Plant design capacity

The ultimate capacity of the WTW will be 1250 Ml/d when Phases 1 and 2 are fully developed. Phase 1 will initially provide 500 Ml/d as elaborated on in **Section 4**. **Table 4** reflects how the actual available capacity will then correspond with projected future demand and recommended minimum availability. For "Recommended Availability" in the below table, the actual demand plus 20% is used, which corresponds to UW's design philosophy, viz. to have 20% excess capacity available to take process units such as filters and/or clarifiers out of operation for cleaning and maintenance purposes.

Table 4: Water Demand, Recommended Plant Capacity and Actual Design Capacity for Phase 1

	Units	Projected Demand	Recommended Availability*	Actual Availability (as per Design)
Water Demand:				
• Up to 2022:	Ml/d	up to 215	0	0
• 2023 to 2031	Ml/d	215 to 240	288	500
• 2032 to 2043	Ml/d	335 to 375	450	500

* Recommended Availability = Expected Demand plus 20%

From **Table 4** it can be seen that spare treatment capacity will be available from the envisaged first inception of Phase 1 in 2023. This spare capacity is important to have, since it will serve as emergency capacity to augment supply to consumers if a serious breakdown is encountered at any of the other big treatment plants of UW.

Although the complete plant capacity of 1 250 Ml/d has been considered when drawing up process flow diagrams (PFDs), setting aside the required plant area and planning the plant layout, the Conceptual Design allows for Phase 1 requiring only 500 Ml/d. The Phase 1 capacity of 500 Ml/d will be provided in four parallel trains.

A process flow diagram (PFD) based on the selected treatment processes and above design capacity has been drawn up and is shown in Drawing No 1301.X01.UW-002. The unit processes employed are discussed in detail in the following sections.

6.3 Unit Processes to be Employed

The individual unit processes that have been chosen for the proposed uMkhomazi WTW are discussed below in the sequence that the water flows through the plant and is treated. The discussion is supported by a basic process/pipe and instrumentation diagram (P&ID, Drawing No 1301.X01.UW-001, Sheets 01 to 05), which should be read in reference to the relevant process descriptions.

6.3.1 Inlet Works

Raw water will be gravity fed *via* a raw water pipeline to the new WTW. No allowance for silt or sand removal at the plant needs to be made. A waterworks of the size and treatment capacity as envisaged cannot be provided without sufficient raw water storage up front to maintain an uninterrupted supply of water to the WTW. The planned Smithfield Dam coupled with the facility to change the raw water abstraction depth in the abstraction tower, will ensure that silt, fine sand and heavier particles will settle out in the dam before reaching the abstraction point and will therefore not be transported to the plant.

The Inlet Works will cater for the full plant capacity, *viz* 1 250 Ml/d, although only used at 500 Ml/d capacity for Phase 1. A hexagonal distribution tower will be provided, which will evenly split the incoming flow into the downstream treatment trains. Raw water will enter the box at the bottom, in the center, and can discharge into six outlet chambers, each fitted with an adjustable weir and outlet pipe that feeds into a treatment train downstream. Although six chambers will be provided, only five will be maximally in use. The sixth will serve as stand-by unit, should maintenance require one to be taken out of operation. The weirs will be manually adjustable with a handwheel, and will therefore be fitted with a gearbox and rising spindle. This will allow selecting specific trains to be operated and equal and accurate flow splitting to the different trains in operation.

The following chemical dosing will be applied at the inlet works:

- Pre-chlorination. Chlorine gas will be added to the raw water, at least 200 m upstream of the inlet pipe before the distribution box, at an average dosage rate of 1.5 mg/l. This will be mainly for disinfection of the raw water, prevention of biofilm formation in pipes and tanks downstream and oxidation of reduced iron and manganese when the latter occur in low concentrations only. To cater for concerns regarding the formation of chlorine by-products such as THMs, pre-chlorination will be supplemented with permanganate as oxidant when higher Fe and Mn concentrations are experienced in the raw water.

- Lime. Slaked lime [78% (m/m) $Ca(OH)_2$] will be made up and dosed as a slurry to the raw water at the inlet pipe to the distribution box at an average dosage rate of 10 mg/l, to raise the pH and alkalinity that is reduced during chlorine addition and for stabilization of the final water.
- Potassium permanganate ($KMnO_4$). Reduced iron and manganese concentrations in the raw water are expected to be relatively high, at 0.8 mg/l Fe and 0.1 mg/l Mn on average, which will require addition of a strong oxidant and $KMnO_4$ has been chosen to oxidize and precipitate these ions. It is estimated that, on average, 1 mg/l $KMnO_4$ will have to be added to the raw water. A 1% (m/m) $KMnO_4$ solution will be dosed to the raw water, where it falls over the overflow weir and is aerated, after which the water flows to each individual train.

6.3.2 Flash Mixing, Coagulation and Flocculation

Suspended matter in raw surface waters consists mainly of clay particles that exist as stable or near-stable colloidal particles in suspension. Coagulants and flocculants are added to first destabilize these suspensions in a rapid mixing step, followed by slow mixing to form a larger floc during flocculation that can then be removed in subsequent treatment processes. The rate of destabilization, aggregate formation and the size and structure of flocs formed are primarily controlled by the intensity of agitation. During rapid agitation micro-particles are formed, whereas slow agitation results in macro-particles being formed. The collision of particles is effected by their drag velocities, which depend on the velocity differences between neighboring layers of liquid and are therefore difficult to calculate (Polasek, 1979). The root mean square velocity gradient, G, is therefore used as guideline to express the mean intensity of agitation, which is calculated from the work per unit time put into a unit volume:

$$G^2 = \frac{P}{V.\mu} \qquad \text{......} 3$$

with
G = Root mean square velocity gradient (s^{-1})
P = Work per unit time ($m^2.kg.s^{-3}$; Watt)
V = Liquid volume (m^3)
μ = Dynamic viscosity ($kg.m^{-1}.s^{-1}$)

Alum has given good coagulation and flocculation results in beaker tests (**Section 5.2.3**) and will be used as primary coagulant for the new plant. Alum reacts as coagulant according to the following reaction.

$$Al_2(SO_4)_3 + 6(HCO_3)^- \rightarrow 2Al(OH)_3\downarrow + 3SO_4^{2-} + 6CO_2\uparrow \qquad \text{......} 4$$

Flash mixing. Typical G-values for flash mixing used at other Umgeni Water WTW's lie in the range of 600 s^{-1} to 1 000 s^{-1} (UW, 2012). Flash mixing can be achieved with a variety of technologies, of which the following were considered for the new uMkhomazi WTW:

- Mechanical mixing, using mechanical agitators or mixers. For operational flexibility, these need to be provided with Variable Speed Drives (VSD);
- Hydraulic mixing, achieved using hydraulic jumps to cause turbulence and mixing;
- Static in-line mixing, using fixed installations that obstruct the flow of water in such a way as to cause turbulence and mixing.

Each of these alternative technologies carries inherent advantages and disadvantages which are summarised in **Table 5** below. Capital and O&M Costs have been calculated for unit production costs taking Net Present Values (NPV) into account as discussed in **Section 9** of this report. A treatment train for a capacity of 62.5 Ml/d is used for comparative purposes.

Table 5: Comparison of Flash Mixing Technologies

Operational Parameter	Mechanical Mixing with VSD	Hydraulic Mixing	Static In-line Mixing
Operational control	High	Low	Low
Adaptability to changing Operation	High	Low	Low
Skilled maintenance required	Medium	Low	Low
Sophistication of operation	Low	Low	Low
Headloss required [m]	0	1	1
Power absorbed [kW] for 62.5 Ml/d treatment unit	38.5	47.2*	47.2*
Capital cost [R] • Civil • Mechanical **TOTAL**	 112 000 455 000 **567 000**	 123 000 625 000* **748 000**	 105 000 625 000* **730 000**
Annual cost • Civil redemption • Mechanical redemption • Power (62.5 Ml/d) • Maintenance **TOTAL**	 5 405 40 923 175 499 18 760 **240 588**	 5 936 56 213 218 714* 25 615 **306 479**	 5 068 56 213 218 714* 25 525 **305 520**

* Theoretical only: Additional pumping will be required to make up for headlosses due to hydraulic and static mixing.

Hydraulic flash mixing is achieved with hydraulic jumps that cause significant headlosses but is widely used due to its low maintenance and operational input as well as reasonable total costs compared to the other alternatives. However, the limited headloss gradient available (see **Section 6.1.4**) for the envisaged overall plant would require any additional headloss created due to a unit process employed to be compensated for by lifting the treated water by the amount of head so lost. For the proposed uMkhomazi WTW, an additional one metre of elevation will be necessary for hydraulic flash mixing and the costs to compensate for this by providing additional pumping capacity were included when comparing different mixing technologies as given in **Table 5**.

Similarly, a static, in-line flash mixer that is also often used for especially flash mixing but also for flocculation (see later) will result in additional headlosses, which will then also require the final water to be pumped to the UW bulk distribution system at Umlaas Road, instead of the preferred method of gravity discharge from the WTW to this system. Any headloss occurring over the flocculation units (see **Table 7**) will also need to be recovered by these pumps.

For mechanical flash mixing axial flow impellers installed in a mixing chamber with ca 120 s retention time needs to be provided. The relatively short impoundment time for dam inflow water of 0.3 years will result in raw water quality fluctuations at the plant inlet and thus require flexibility in mixing intensity, both for flash mixing and coagulation. This should be allowed for by providing VSD's for the mixer motors, which will allow operators to adjust the mixing intensity according to the process requirements. Regular maintenance and diagnostics will ensure optimum performance and operational lifetime of these units.

Thus, for comparative purposes in **Table 5**, the costs for hydraulic and static in-line mixing include a provision made for and operation of final water pumps to transfer the final water to the Umlaas Road distribution system, which would not be required if mechanical flash mixing is used. Hydraulic or static in-line mixing would not only increase the plant's power consumption and maintenance costs due to final water pumping, but also require extensive additional bulk excavations to be performed for the entire WTW downstream of the flash mixing process to accommodate the resulting headloss.

When taking the above factors into account, mechanical flash mixing will result in slightly more than 20% annual savings, if compared with static in-line mixing and/or hydraulic mixing. It is thus recommended that mechanical mixing is employed for flash mixing at the uMkhomazi WTW and we have based our conceptual design on employing this technology.

The flash mixing aspect of the conceptual design should be reviewed at detailed design stage to assess whether the overall system hydraulics could be adjusted to accommodate hydraulic mixing. If an additional one metre of elevation can be made available, hydraulic flash mixing would be the process of choice.

<u>Coagulation</u>. Adsorption-destabilization and sweep coagulation are two distinct mechanisms by which alum coagulation can take place. The prevailing mechanism will depend on the alum dosage and raw water characteristics, mainly pH:

- Adsorption-destabilization. Alum forms a number of intermediate hydrolysis products, which attach to the surface of clay particles. Interparticle bridging then takes place and clay particles coagulate to form flocs.
- Sweep coagulation. Aluminum hydroxide precipitate captures and encloses clay particles in the precipitation process. Latter forms floc particles, which settle and thereby remove the clay particles.

Depending on which mechanism rules, specific design values such as retention time (t), G-value, Gt-value and flowrates will give optimal coagulation and flocculation results. **Figure 8** indicates the prevailing mechanism for a particular alum dosage and raw water pH.

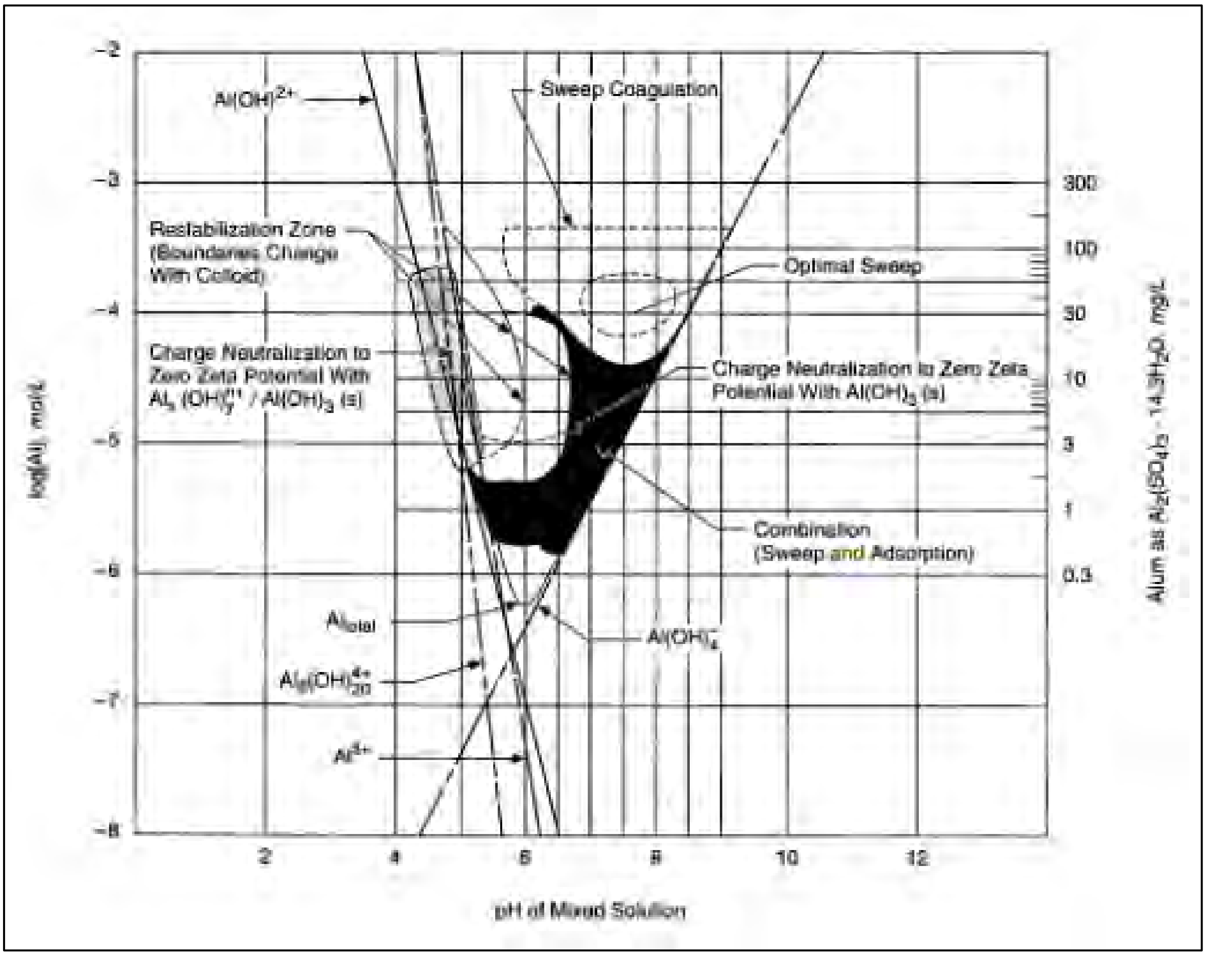


Figure 8: Alum coagulation diagram (Amirtharajah and Mills, 1982)

The expected operating range during coagulation with alum and corresponding pH, after lime stabilization and without poly addition, obtained from beaker tests (Section 1.3.2.3), is reflected in **Table 6**.

Table 6: Expected operating range with alum dosing, without poly addition at uMkhomazi WTW

Parameter	Units	Minimum	Average	Maximum
Alum dosage	mg/l	15	20	35
pH		7.4	8.0	8.5

From **Figure 8** it can be observed that alum sweep coagulation will be the determining mechanism during coagulation for the expected operating range of the new plant and thus formed the basis for selecting typical flash mixing design parameters.

Flocculation. After coagulation with alum, destabilization of clay particles is completed *via* flocculation, whereby optimal conditions are created for contact between small flocs and individual clay particles to agglomerate into larger, settleable flocs. The control of mixing intensities, quantified by the G-value, is of high importance during flocculation as there are two opposing phenomena to consider:

- Increasing the mixing intensity increases inter-particle contact, which results in floc growth;
- Increasing the mixing intensity too much results in disintegration of previously formed larger flocs.

Therefore, there is a very narrow range of mixing intensities that will result in optimal floc formation and clay particle destabilization. The flocculation process usually consists of an initial rapid mixing stage followed by a reduction in mixing intensity to allow for floc growth without disintegration of already formed larger flocs. Typically, G-values for flocculation can range from 10 s^{-1} to 100 s^{-1}. UW typically operates at a velocity gradient of 70 s^{-1} for initial rapid mixing, followed by 20-25 s^{-1} for slow mixing (UW, 2012).

As is the case for coagulation, flocculation can be achieved using different mixing methods:

- Hydraulic static flocculators such as baffled channels or spiral flocculators (**Figure 9**: Typical circular static hydraulic flocculator (Rand Water – from Waterwise)) cause mixing due to frictional head loss around 180° bends or tapered channels;
- Mechanical flocculators or rotor/stator flocculators, which transfer energy into the fluid through electrically driven axial agitators, mixers or paddle wheels;

Figure 9: Typical circular static hydraulic flocculator (Rand Water – from Waterwise)

The advantages and disadvantages of each technology are summarised in **Table 7**. Capital and O&M Costs have been calculated for unit production costs taking Net Present Values (NPV) into account as discussed in Section 2 of this report. A treatment train with 62.5 Ml/d capacity is used for comparative purposes. As discussed previously, the limited headloss gradient available (see **Section 6.1.4**) for the envisaged overall plant would require any additional headloss created due to a unit process employed to be compensated for by lifting the treated water by the amount of head lost. This has also been taken into consideration in the cost comparison in **Table 7**.

Table 7: Comparison of flocculation technologies

Operational Parameter	Hydraulic/Static Flocculator	Mechanical Flocculator
Simplicity of Operation	High	Medium
Control over mixing intensity	Low	Medium
Adaptability to changing operation	Low	High
Skilled maintenance required	Low	Medium
Power absorbed [kW] for 62.5 ML/d plant	96*	5.4
Capital cost		
• Civil	1 438 000	473 000
• Mechanical	625 000*	120 000
TOTAL	**2 063 000**	**593 000**
Annual cost		
• Civil	69 316	22 828
• Mechanical	56 213	10 793
• Power (62.5 Ml/d)	216 602*	19 882
• Maintenance	27 190	7 165
TOTAL	**369 321**	**60 668**

* Theoretical only: Additional pumping will be required to make up for headlosses due to hydraulic and static mixing.

Hydraulic, static flocculators achieve mixing due to frictional forces around bends or tapered channels. This results in an overall hydraulic headloss over the flocculator. As with flash mixing, this head lost during flocculation needs to be recovered using final water pumps, due to the fact that only very limited headloss is available over the entire plant site before discharge to the potable water distribution system. A hydraulic, static flocculator will require final water pumps with ca 2 m of hydraulic head to be provided. This will significantly increase both the capital costs and the power consumption when compared to mechanical flocculation. Mechanical flocculation does not cause a headloss and therefore no final water pumping is required to make up for the headloss created due to hydraulic mixing. From **Table 7** it can be seen that annual cost involved with pumping the final water after hydraulic, static flocculation far exceeds the cost of mechanical mixing.

We therefore recommend and have based our conceptual design on flocculation using mechanical, axial mixers in two separate but equal sized flocculation tanks with diminishing energy intensity. A total retention time of 13.5 minutes will be provided, with the first tank operating at a G-value of 70 s^{-1} and the second tank with G-value of 25 s^{-1} for slow mixing.

Enhanced Flocculation. The above flocculation technologies are all considered to be "conventional" treatment processes. More recent developments in flocculation and clarification processes allow for changes to the floc structure to form a much denser and faster settling floc than with conventional methods. These include: Ballasting agent addition during flocculation; sludge recirculation from the clarifier to the start of the flocculation process; addition of an organic polymer. These processes all facilitate higher clarification rates than achieved in conventional treatment processes and thus decrease the size and cost of clarifiers employed downstream to remove larger floc particles.

A very brief description of enhanced flocculation and settling methods used in high-rate clarifiers for enhancing floc forming efficiency and settling rates is given below:

- Ballasting agent - an inert agent such as micro-sand, magnetite or bentonite is added as seed for the formation of dense and rapid settling flocs (Budhram et. al, 2013).
- Sludge recirculation - a portion of the sludge from the clarifier is recycled back to the start of the flocculation chamber. This increases inter-particle bridging for the fast formation of larger flocs.
- If micro-sand is used as the ballasting agent it can be recovered from the sludge using a hydrocyclone. The sand can then be reused as ballasting agent, although a small fraction (typically 5%) is lost with the waste sludge. If bentonite is used it cannot be recovered and thus a portion of the sludge is recirculated to aid in the formation of flocs.
- Chemical flocculant addition - an organic flocculant aid can be added to enhance floc forming and increase the settling velocity of flocs.

The new envisaged uMkhomazi WTW will make use of high rate clarifiers which will be discussed in more detail in **Section 6.3.3**. This will result in a much smaller footprint of the WTW, which conforms to the specific design aspects of the design philosophy as discussed in **Section 6.1.4**. In order to be able to employ high-rate clarification processes, enhanced flocculation and settling will be required. Bentonite as ballasting agent will be used, as this is currently also used at some of the other UW WTW's (Thompson, 2013). An organic polymer will also be added at the start of the first flocculation chamber in order to increase the settling velocity of flocs. Sludge wasted from the clarifier will be partially recirculated to the flocculation chamber. This will enhance flocculation, which will result in a heavier floc being formed allowing higher settling velocities and maintaining higher upflow velocities in clarifiers with a smaller footprint. The high-rate clarification process is discussed in **Section 6.3.3**. Average alum, polyelectrolyte and bentonite dosing rates with associated chemical demand and operational costs are discussed in **Section 9**.

The mixing chambers for coagulation and flocculation will each be fitted with mechanical mixers. These are designed to achieve the desired mixing intensity for each agitation step and can be controlled using variable speed drives. This design allows for easy process adjustments in case of varying inlet water quality. The coagulation, rapid mixing flocculation and slow mixing flocculation chambers will be built in modular units connected to high-rate clarifiers. Each such unit will be able to handle a hydraulic throughput of 62.5 ML/d and thus 8 off such combined coagulation, flocculation and clarification units will be constructed for the Phase 1 plant with a capacity of 500 ML/d. The mixers will have the following specifications, per 62.5 ML/d unit (**Table 8**):

Table 8: Coagulation and Flocculation Mixer Specifications

Mixer	Required G-value [s-1]	Power Installed [kW]	Retention time [s]
Coagulation Mixer	600	45	138
Fast Flocculation Mixer	70	3	406
Slow Flocculation Mixer	25	2.2	406

6.3.3 Clarification

Clarification processes have developed considerably over the past fifty years and fall into two main categories, conventional clarification and high-rate processes:

Conventional clarification processes. These require flocs to settle large distances, typically about 3 m. This is achieved by reducing the upflow velocity in the clarification basin low enough so that the gravitational downward velocity of a floc particle is larger than the vertical upflow velocity of the entrained particle. As long as the upward force on the particle caused by flow entrainment (Ff) is larger than the gravitational force on the particle (Fg) it will settle toward the bottom of the basin. There are thus only two opposing forces acting on a floc particle, as shown in

Figure 10 below. This conventional clarification method requires slow linear upflow rates, typically in the order of 1 m3/m2.h, in order to achieve settling of particles and thus requires a large surface area.

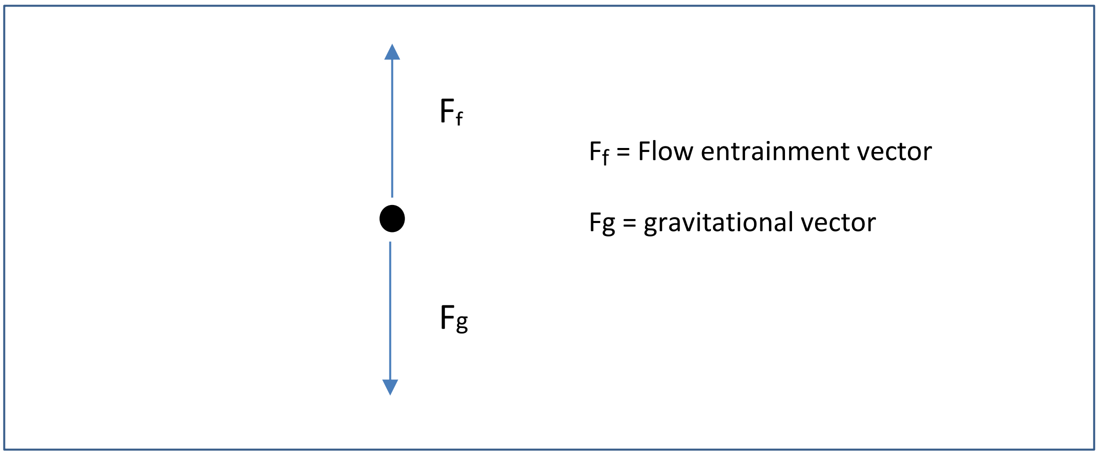


Figure 10: Forces acting upon floc particle in conventional settling process

Conventional clarification is often enhanced by employing a "sludge blanket" to entrap flocs and serve as seed to form larger flocs. The HR-CSAV high-rate clarifier developed by Polasek (Polasek, 2005) and Pulsator developed by Degrémont (Degrémont, 2007) use extended rapid agitation during flocculation coupled with the addition of a chemical flocculant aid to enhance flocculation and employ a sludge blanket to give better settling characteristics. The sludge blanket compartment is maintained within the clarifier in order to act as deposit site for incoming floc particles. The HR-CSAV has, however, been reported to be less efficient for the clarification of low turbidity water (<25 NTU)(WRC, 2013). From our own experience, sludge blanket clarifiers are limited with regard to their linear upflow velocities to below 12 m/h and are generally difficult to operate above 6 m/h.

- High-rate clarification processes. These significantly reduce the plant footprint when compared to conventional sedimentation processes. Two different processes are generally employed, *viz* flotation and sedimentation:
    - Flotation. Dissolved air flotation (DAF) is typically used when a floc is formed that is light and consists predominantly of organic particulate matter such as algae. Also, the DAF process should not be employed if turbidities, caused by colloidal matter such as river water runoff, exceed 400 NTU. Since the maximum plant design values for the new uMkhomazi WTW will be 800 NTU (**Table 1**), DAF will not be suitable as clarification process and was therefore not further addressed in this study.
    - Sedimentation. High-rate clarification processes employ lamella, which are typically hexagonal tubes arranged on a 60° angle to the horizontal, to assist with and enhance the sedimentation process. The angled flow direction reduces the vertical upward vector of the force acting on the particle due to flow entrainment. In addition, the angled lamella channel creates a very short distance for the particle to settle downward before colliding with the channel wall. Flow channeling inside the tubes creates a laminar flow pattern, which results in a boundary layer with zero flow velocity at the channel wall. A floc particle therefore only has to settle onto the channel bottom wall at which time the flow entrainment velocity becomes zero and the only remaining force acting on the particle is gravity, resulting in a downward slide of the particle along the lamella channel. **Figure 11** depicts this scenario:

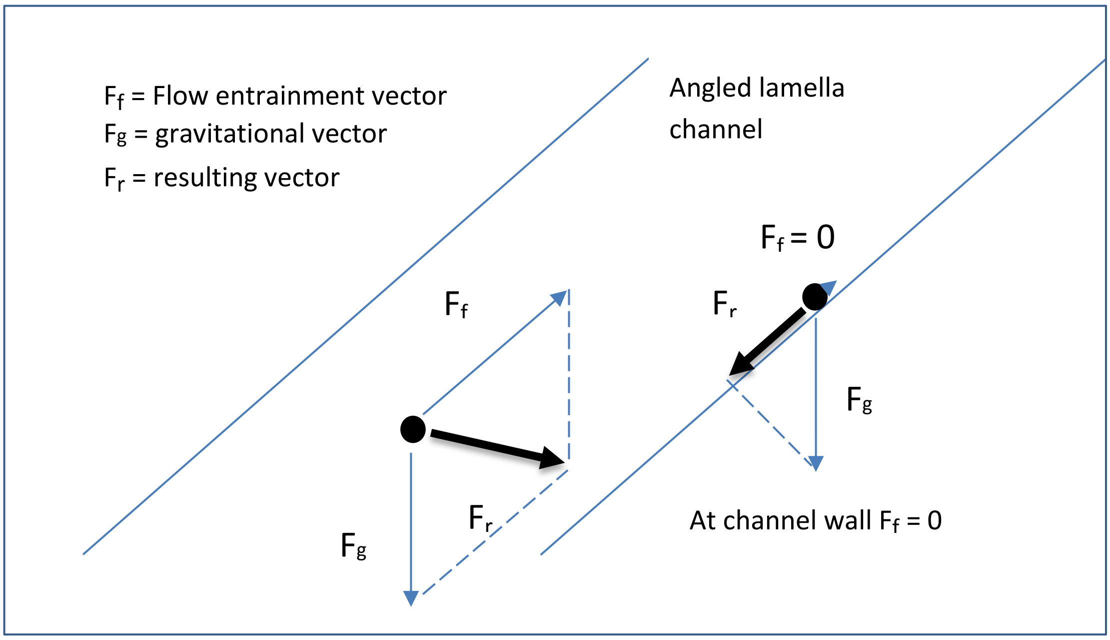


Figure 11: Forces acting upon floc particle in lamella clarifier

From **Figure 11** it can also be seen that the flow entrainment forces in a lamella clarifier will be much larger than in a conventional clarifier, which again translates into much higher surface loading rates that can be maintained in lamella clarifiers. This reduces the total area required for lamella clarifiers significantly.

Particles will settle at the bottom of the clarification basin, where the sludge is then removed using a rotating scraper. To further enhance the settling process, a portion of the settled sludge is recirculated back to the flocculation basin, where it acts as seed to form larger, heavier particles thereby allowing higher throughput rates.

The settling rate can be further enhanced with a ballasting agent such as bentonite or fine sand (microsand – ES < 150 µm). When microsand is used, the bulk thereof is recovered by employing a hydrocyclone in the recycle stream, to wash the particulate matter off the sand. The sand is then returned to the flocculation tank, whereas other particulate matter is discharged as sludge, as depicted in **Figure 12** for Veolia's Actiflo® clarifier. Latter can already be classified as an ultra high-rate clarifier and needs more skill to operate.

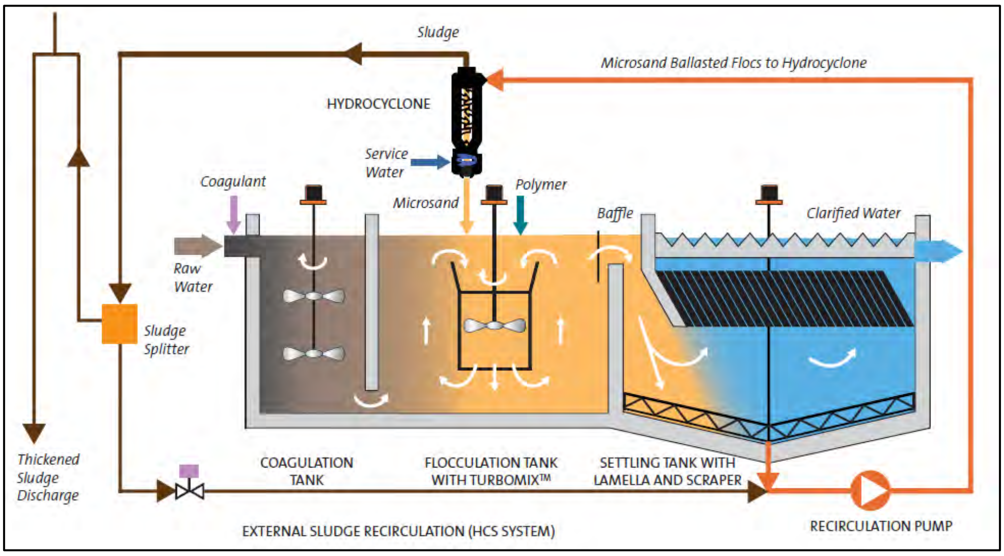
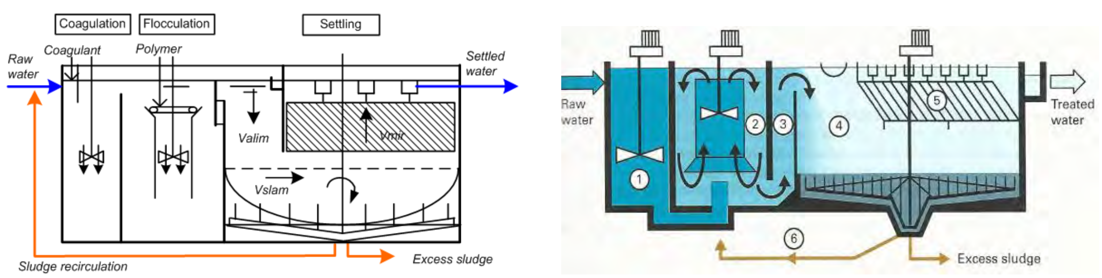


Figure 12: Veolia's Actiflo® ultra high-rate clarifier with microsand ballasting and recovery

However, various proprietary high-rate clarification technologies using lamella plates and sludge seeding either by micro-flocculation or sludge recycle (with or without ballasting agent) are available. These are easier to operate and, in South Africa, Veolia and Degrémont are currently amongst the market leaders with regard to this technology. Both Veolia and Degrémont clarifiers are well established technologies and have been proven to be very effective. Degrémont's Ultrapulsator® uses micro-flocculation and Veolia's Multiflo® and Degrémont's Densadeg® clarifiers use sludge recycle in order to increase inter-particle bridging for the rapid formation of larger flocs, and both are relatively easy to operate and classify as high-rate clarifiers.

Figure **13** depicts the similarity between Veolia's Multiflo® and Degrémonts Densadeg®, which are high-rate, sludge contact clarifiers.



a) Veolia's Multiflo®

b) Degrémont's Densadeg®

Figure 13: Schematics of the Multiflo® and Densadeg® sludge contact clarifiers

Table 9 summarizes typical linear clarification rates as achieved with various clarification processes and well-known suppliers currently on the market.

Table 9: Summary of typical clarification processes on the market

	Unit*	Typical Range	Reference
Conventional clarification	m/h	1	AWWA (1999)
Sludge blanket clarification: Degrémont ○ Pulsator ○ Pulsatube HR-CSAV	 m/h m/h m/h	 2 – 4 4 – 9 8 – 12	 Degrémont (2007) Polasek (2005)
High-rate clarification: DAF Degrémont ○ Superpulsator ○ Ultrapulsator Veolia ○ Multiflo ○ Actiflo (ballasted floc)	 m/h m/h m/h m/h m/h	 5 – 11 4 – 8 9 – 12 10 – 20 40 - 60	 Haarhoff & van Vuuren (1993) Degrémont (2007) Veolia (2007)

* Linear upflow rate ($m^3/m^2/h$) during clarification

Figure 14 gives an indication of the savings in surface area that can be achieved when employing high-rate clarification equipment. Successive upgrades on the Iver Plant, London, reduced the footprint of the required clarification area substantially.

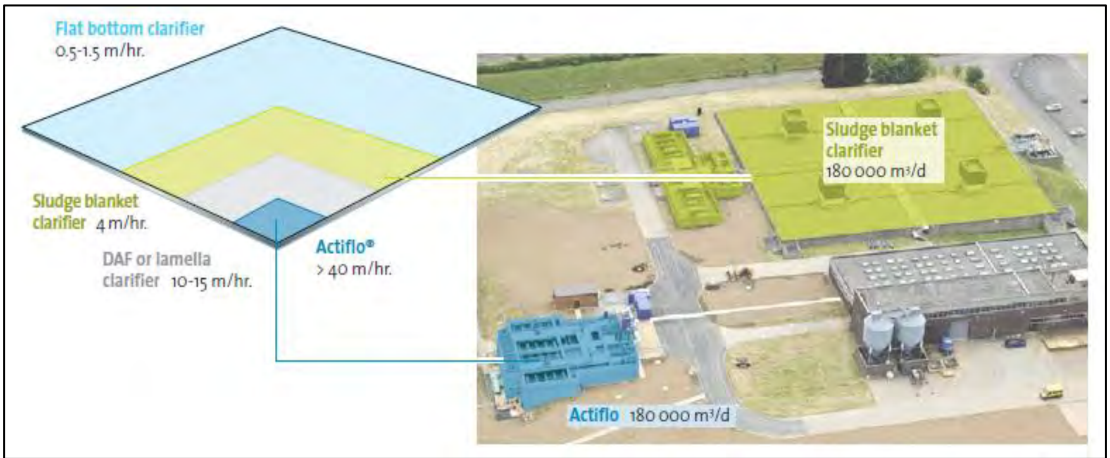


Figure 14: Footprint area required for clarification technologies – Iver, London (Veolia, 2007)

We do not recommend an ultra high-rate clarifier such as the Actiflo® be employed at the new uMkomazi WTW, because latter requires more sophisticated operational skills. However, a high-rate clarifier that employs sludge contact such as Veolia's Multiflow® or Degrémont's Ultrapulsator® can be operated relatively easily by semi-skilled operators and will result in substantial savings in the overall surface area required for clarification, compared to conventional clarifiers. Since land is not readily available, the design was based on a high-rate clarifier. This is one of the most important design considerations for this plant, as land will need to be expropriated from local farmers and therefore treatment processes were chosen that keep the footprint area to a minimum.

For the uMkhomazi WTW we have allowed for coagulation, flocculation and clarification units that will be constructed as modular units, each with a capacity to treat 62.5 Ml/d (refer to P&ID 1301.X01.UW-001 Sheet 1 of 5) based on Veolia's Multiflo®, but Degrémont's Ultrapulsator® or Densadeg® can also be fitted into the same area. Therefore, 20 off high-rate clarifiers to operate at rise-rates of 9 m/h and fitted with lamellas and rotating sludge removal scrapers are envisaged for the complete capacity of 1 250 Ml/d. However, since UW may be hesitant to operate at such high-rise rates initially, we recommend that, for Phase 1 of the plant to treat 500 Ml/d, double the number of high-rate clarifiers are provided in order to be able to operate these at only 4.5 m/h rise-rates. Thus, 16 off high-rate clarifiers, each with 288 m² lamella area to treat 31.25 Ml/d, will be provided under Phase 1.

Since Phase 2 is not envisaged for another 20 years after implementation of Phase 1, UW will have ample time to practice, test and assess if they are confident to operate these high-rate clarifiers at higher throughput rates than the initially implemented rise-rates of 4.5 m/h. Should UW **not** be comfortable operating at higher throughput rates, 40 off high-rate clarifiers (instead of 20 off) will be required and the layout drawings (1301.X01.UW-100 &110) indicate how the additional 20 clarifiers can be fitted in. The main design parameters as envisaged for Phase 1 and 2, respectively, are summarized in **Table 10**.

Table 10: High-rate Clarifiers – summary of design specifications

Type	High-rate clarifiers with microflocculation/sludge contact and lamella, e.g. Ultrapulsator®, Multiflow® or Densadg®
PHASE 1 – 500 Ml/d:	
Number	16 off
Normal capacity (each)	31.25 Ml/d (= 1 302 m^3/h)
Effective clarification area (each)	288 m^2 (lamella footprint area)
Linear (upflow) clarification rate	4.5 m/h
PHASE 1 & 2 – 1 250 Ml/d:	
Total Number	20 off
Normal capacity (each)	62.5 Ml/d (= 2 605 m^3/h)
Effective clarification area (each)	288 m^2 (lamella footprint area)
Linear (upflow) clarification rate	9.0 m/h

6.3.4 Rapid Gravity Sand Filtration

After clarification, the water is filtered using rapid gravity sand filters to further remove suspended impurities and particles remaining after the clarification process. Slow sand filtration is not considered for this plant due to the vast filter area that will be required. This technology relies on very slow linear filtration rates, typically 0.25 – 0.5 $m^3/m^2/h$ (m/h) (Schulz & Okun, 1984). For the uMkhomazi WTW, slow sand filters will be impractical as this technology will require vast areas for filtration and land is not readily available. Pressure filters were also not considered because this will require additional pumping and, due to the large size of the plant, would entail high mechanical costs, increase electrical energy consumption and need more maintenance. Since sufficient head is available to allow gravity filtration, rapid gravity sand filters (RGSF) will be the most cost-effective and suitable filtration system for such high throughputs as envisaged for this plant and the conceptual design was based on this technology. **Figure 15** shows a typical rapid gravity sand filtration plant.

Figure 15: Typical rapid gravity sand filter plant (Nampapa Nakhoneluang)

RGSF technology is also well-known to UW since all their large WTWs, viz. Midmar, Wiggins and Durban Heights employ this technology. In all gravity filtration systems, the pressure of the water above the filter bed forces the water through the media. Particulate impurities are then removed by the media through physical straining (sieve effect), adsorption and absorption processes. As the media bed becomes clogged the pressure required to force the water through the media increases until a point is reached, where the available head from the water above the media is insufficient to force the water though the filter bed. This may result in particle breakthrough and an increase in turbidity in the filtered water.

Backwashing is required to release and remove trapped particles before a certain headloss or before the filtered water quality deteriorates. After backwashing, the filter media will be very clean, which may result in protozoan cysts such as Cryptosporidium and Giardia cysts passing through the bed and ending up in the filtrate. These cysts are trapped in the filter bed during normal filter operation, due to the fact that flocs and suspended solids occupy the pore spaces within the filter bed and prevent the cysts from passing through the bed. With backwashing, these pore spaces are cleared, which allows cysts to be released from the bed. However, the cysts are often not fully transferred out of the filter bed with the backwash water. As soon as the filter is then put into normal filtration mode the cysts are freed along with the first filtrate due to the fact that the pore spaces in the bed have been cleared. Thus, the first filtrate after backwashing needs to be discharged for the filter bed to mature and we have allowed for a 30 minute discharged-to-waste step whereby the first filtrate will be discharged to the sludge treatment plant for washwater recovery.

For the uMkhomazi WTW it is recommended that, in addition to sand, an upper layer of anthracite with larger grain size than the underlying silica sand is provided, which will increase the void space to increase the floc penetration depth. This will increase the maximum filter run time before backwashing is required as well as increase the storage capacity of impurities removed from the incoming water.

Whereas different suppliers have their proprietary designs, we recommend and have based the conceptual design for the uMkhomazi WTW RGSFs on the following considerations:

- Number of filters = N+1, where N is the number of filters required to accommodate the full hydraulic capacity. This ensures that when a filter is taken offline for maintenance, the remaining filters are capable of handling the full plant flow. The N+1 principle will apply to 250 ML/d extensions; Phase 1 with 500 Ml/d capacity will have two extra filters that can be taken off line for maintenance purposes, whereas five off extra filters will be provided for the final demand of 1 250 ML/d, without affecting filtration performance;
- RGSF technology should be chosen that does not require the filter to be taken off-line during backwashing. This will allow constant filtration rates during a filter run. Numerous RGSF designs are available on the market where the normal hydraulic inflow to a given filter is maintained during backwashing to create a surface sweep that enhances floc removal from the surface;
- The filters will operate at 8.7 m/h linear filtration rates, regardless of whether backwashing is in progress on one of the filters in a filter bank;
- Filter run-times of at least 24 h must be achieved, but up to 48 hours should be possible, especially during times of low raw water inflow turbidity;
- A 1 500 mm deep bed with dual-media (anthracite and silica sand) will be provided in order to provide sufficient floc storage to achieve the required filter run times also during times of increased turbidity. The anthracite layer will be 700 mm deep (ES = 1.3 mm) while the sand layer will be 800 mm deep (ES = 0.8 mm).

For the uMkhomazi WTW we have based our conceptual design on 50 off double-bed RGSF to treat full plant capacity of 1 250 ML/d and **Table 11** summarises the double bed sand filter design (refer to P&ID 1301.X01.UW-001 Sheet 2 of 5):

Table 11: Double-bed rapid gravity sand filter design specifications

Quantity (for 1 250 ML/d)	50 off
Normal operating capacity (each)	1 045 m^3/h @ 8.7 m/h
Max theoretical design capacity (each)	1 200 m^3/h @ 10 m/h
Type	Double bed
Size	15 m x 4 m per bed – double bed
Filter Area	120 m^2 per double-bed filter
Media: • Silica Sand (0.8 ES) • Anthracite (1.3 ES)	 800 mm deep (bottom) 700 mm deep (top)
Backwash	Air + water combined
Control philosophy	Fully automated

RGSF Backwash. The filters will be backwashed automatically, using both water and air, one double-bed filter at a time on a constant level-based backwash control system. The normal inflow to the filter will be used for surface washing, to also save on filtered water required during backwashing. The backwash sequence will be as follows:

- Airscour - 5 minutes of combined water (16 m/h) and air (55 m/h) scour to semi-fluidise the media bed, obtain rigorous scouring and release trapped material from the sand particles;
- Final Rinse - 5 to 7 minutes of rinse with water (24 m/h) only from the bottom of the filter to wash out entrapped floc;
- The normal inflow to the filter will be available for backwashing purposes. This water contributes a linear cross-flow rate with inflow still at the normal filtration rate (of 8.7 m/h) and thus only 7.3 m/h of washwater for the combined air and water scour and 15.3 m/h for the water only rinse are required from the bottom of the bed, respectively. This water is supplied *via* gravity flow from a washwater reservoir;
- An estimated 20 minutes (to be determined during commissioning) of "first-filtrate-to-waste" after backwashing in order for the filter bed to mature;
- Backwash water will be transferred to the sludge handling facility at the plant, for removing solid waste and recovering as much water as possible (see Section 1.4.3.7).

The total water demand for backwashing for Phase 1 and 2 is given in **Table 12** below. The worst case condition, *viz* backwashing every filter once every 24 hours, was considered.

Table 12: Washwater required and wastewater produced per RGSF backwash

	Time (max)	Total water velocity* required	Normal inflow velocity*	Additional washwater velocity required	Washwater flow rate required	Washwater volume required (max)	Total backwash water volume out
Air + water scour	5 min	16 m/h	8.7 m/h	7.3 m/h	880 m^3/h	74 m^3	160 m^3
Rinse (max)	7 min	24 m/h	8.7 m/h	15.3 m/h	1 840 m^3/h	215 m^3	336 m^3
Total backwash water per filter backwash						289 m^3	496 m^3
First filtrate to waste per filter							90 m^3
TOTAL PER DAY PHASE 1 (500 ML/d PLANT)						**5.78 ML/d**	**11.72 ML/d**
TOTAL PER DAY PHASE 2 (1 250 ML/d PLANT)						**14.45 ML/d**	**29.3 ML/d**

* "velocity" refers to linear filtration downflow and/or backwash water upflow rates – $m^3/m^2/h$

Control system. The backwash control system will be based on keeping the level of water above the filter media constant. The inflow to the plant is distributed evenly between the filters using an overflow weir to each filter. Each filter is fitted with a level sensor which is used to control the filtered water outlet control valve. The level in each filter is kept constant by opening the outlet valve as the filter bed becomes clogged. Since the level is kept constant in the filter, the outflow is equal to the inflow and each filter therefore operates at a constant throughput rate. Once the outlet valve is fully open and the level above the media starts to rise, the backwash procedure is automatically initiated. Should a filter not become clogged enough for this to occur over a period of 48 hours, a backwash will be initiated through timer control. The entire system is envisaged to operate fully automated and only minimal operator input will be required, for example for maintenance purposes only.

6.3.5 Granular Activated Carbon Filtration

Granular activated carbon (GAC) is used in drinking water plants for removing organic constituents, which can cause aesthetic problems such as unpleasant smell, taste and color. GAC works mainly *via* the process of adsorption, by which soluble material in the water physically attaches to the surface of carbon particles. It also removes chlorine and chlorine-based disinfection byproducts. Chlorine-based compounds are usually adsorbed more easily and strongly than organic constituents and chlorination for final disinfection is thus performed after carbon filtration.

GAC is used in either gravity or pressure filters so that the carbon is immobile and water passes over the carbon for adsorption to occur. As soon as the GAC is near saturation, the filter bed is replaced with new carbon. GAC is not generally used in South Africa for potable water production, but is widely used in European plants. In Europe, the GAC is typically exhausted and replaced every 24 to 36 months where GAC is used in potable water treatment plants using river water. The spent carbon needs to be disposed of at a suitable waste disposal facility.

For the new uMkhomazi WTW the organic load of the water from the impoundment dam is expected to be relatively low, assuming proper dam operation and water draw-off procedures. However, the envisaged impoundment (Smithfield Dam) is anticipated to be mesotrophic and occasional blooms of nuisance algal species can be expected in future (see **Section 5.1**). This will initially be manageable with proper dam operation such as spilling, scouring and abstracting raw water from the aerobic zone for treatment in the WTW. It is also envisaged that the raw water quality in the impoundment will deteriorate in future due to increased human settlement along the river banks and agricultural activities that will result in an increased nutrient discharge into the catchment area of the dam. This may result in the dam becoming eutrophic and the raw water will require treatment in the WTW to reduce mainly organic carbon and microbial by-products. If this situation occurs, incorporating a GAC filtration unit into the process at a later stage is recommended in order to deal with the increased organic load.

Although initially not required, it is our opinion that GAC filtration/polishing will be necessary at this plant in the foreseeable future due to nutrient enrichment of the impoundment dam. Therefore, our conceptual design allows for GAC filtration to be incorporated at a later stage. The initial construction will exclude this unit process until it is found to be necessary. All hydraulics and plant layout designs have catered for easy addition of GAC filters at a later stage. Should this be required, we recommend and have based the conceptual design on the following considerations:

- Number of filters = N+2, where N is the number of filters required to accommodate the full hydraulic capacity. Contrary to a rapid gravity sand filter, a GAC filter needs to be taken out of service during backwashing. For the GAC filters, the inflow to a specific filter is stopped during backwashing and only water from the washwater reservoir is used. Also, the GAC bed is replaced from time-to-time, which also requires a complete filter to be taken out of service. Therefore, N+2 filters are required to ensure that when a filter is taken offline for backwashing, maintenance or carbon recharge, the remaining filters are capable of handling the full plant flow. The N+2 principle was again applied to a 250 ML/d train; thus for the final demand of 1 250 ML/d, it would be possible to take 10 off filters off-line for backwashing and/or maintenance purposes at any given time without affecting filtration performance;

- The maximum linear filtration rates must be less than 20 m/h, even if 2 off filters are taken off-line;
- The Empty Bed Contact Time (EBCT) must be at least 10 minutes, even if 2 off filters are taken off-line;
- A 2-stage system should be employed, with an upflow-downflow configuration during filtration in which the water flows upflow through the first bed/stage and downflow through the second bed/stage. Carbon depth in each bed (up and down) must be at least 1.5 m;
- The GAC filters will be backwashed with water only (35 m/h) using water from the washwater reservoir;
- The GAC filters are designed to have a 3 – 4 week filter run-time before backwashing will be required;
- Backwash water will be transferred to the sludge handling facility at the plant, for removing solids and recovering as much water as possible (see Section 1.4.3.7).

Table 13 summarises the double bed GAC filter design (Refer to P&ID 1301.X01.UW-001 Sheet 3 of 5):

Table 13: Double-bed GAC filter design specifications

Quantity (for 1 250 ML/d)	60
Normal operating capacity (each)	870 m^3/h @ 14.51 m/h
Max design capacity (each)	1 045 m^3/h @ 17.5 m/h
Type	Double bed, upflow-downflow configuration
Size	15 m x 4 m per bed – double bed
Filter Area	120 m^2 per double-bed filter
Media: • Granular Activated Carbon	 1 500 mm deep per bed, ES = 1.3 mm
Backwash	Water only
Control philosophy	Fully automated

<u>GAC Filter Backwash</u>. The filters will be backwashed automatically, using only water from the washwater reservoir, one double-bed filter at a time. The backwash procedure will be controlled *via* a timer so that each filter has a filter-run of 3 – 4 weeks before backwashing occurs. Both beds of a GAC filter will be backwashed one after the other, each for 20 minutes. Only water from the washwater reservoir will be used for this at a linear backwash rate of 35 m/h.

The total water demand for GAC filter backwashing for Phase 1 and 2 is given in **Table 14** below. The conditions of 20 minutes backwashing every filter once every 24 days (3.5 weeks) are considered.

Table 14: Washwater required and backwash water produced per double-bed GAC filter

	Time (for both beds)	Washwater linear velocity	Washwater flow rate	Washwater volume (max)	Total backwash water
Backwash	40 min	35 m/h	2 100 m^3/h	1 400 m^3	1 400 m^3
			First filtrate to waste per filter		100 m^3
			TOTAL PER DAY PHASE 1 (500 ML/d PLANT)	**1.4 ML/d**	**1.5 ML/d**
			TOTAL PER DAY PHASE 2 (1 250 ML/d PLANT)	**3.5 ML/d**	**3.75 ML/d**

Control System. GAC filter backwashing will be timer controlled, with each filter being backwashed once every 3 – 4 weeks. The backwash water will be transferred to the sludge handling facility at the plant. The carbon in each filter will need to be replaced as soon as it gets saturated with organic constituents, which cannot be removed with filter backwashing. This is done by measuring the TOC concentration of the water after it has passed through the first stage of the double-bed GAC filter. Once a predetermined maximum TOC concentration is reached, the filter is taken offline and the GAC of the first bed is replaced with new GAC. The flow direction of the filter is then reversed so that the bed with virgin GAC now becomes the second bed in the upflow-downflow sequence, thereby ensuring that virgin GAC is always contained in the second bed of the 2-stage filter system. This ensures that the first bed with the partly exhausted GAC always receives the highest organic load and the second bed with the "newer" GAC is the final, polishing step. If TOC levels between the two filters exceed the maximum, typically set at 1 mg/l TOC, this will always be as a result of the first bed being saturated and no longer performing satisfactorily.

6.3.6 Disinfection and Final Water Storage

Disinfection is the final treatment step in any drinking water treatment process. This is to ensure that the final water conforms to local Drinking Water Quality Standards and that the water is safe for human consumption at all times. Disinfection kills mainly bacteria and viruses but also inactivates protozoan cysts and other pathogens that can be harmful if ingested by humans. There are many disinfection technologies available, with some being more effective than others for certain applications. It is widely accepted that there is no single disinfection technology that can achieve all the treatment objectives and the following technologies were considered for the new uMkhomazi WTW:

- Chlorination using chlorine gas. Although other chlorine-based chemicals such as sodium hypochlorite (liquid) and calcium hypochlorite (solid) are also available, these are much more expensive than pure chlorine gas and are thus not considered for a plant of this size. Chlorination, also in conjunction with ammonia, is the only disinfection method that provides residual disinfection, which means that the water will remain disinfected in the distribution network downstream of the chlorination plant. Since the final water of the uMkhomazi WTW will be stored in a retention tank with 6 h storage time and then discharged to the Umlaas Road pipeline, having residual disinfection capacity will be important for the final water produced at this plant. With high concentrations of organic matter in the water, chlorination by-products such as trihalomethanes (THMs) and haloacetic acids, which are carcinogenic, can be formed.

Therefore, should the organic content in the raw water from the dam increase it will be necessary to provide GAC filters at the plant, as discussed under the previous heading, if chlorination is further used as preferred disinfectant;

- Ozonation, which uses the powerful oxidising agent ozone (O_3) to destroy pathogenic organisms. Ozonation is a very powerful disinfection method, requires lower concentrations and similar contact times than chlorine-based disinfection chemicals. However, ozonation cannot provide residual disinfection and can also produce harmful by-products such as bromate.
- Ultraviolet (UV) radiation, is a disinfection method that does not use any chemicals and thus does not alter the aesthetic quality of the water. UV radiation causes lesion of cell walls and damages the microorganisms' DNA, thereby inactivating them from performing their pathogenic functions and does not form harmful disinfection by-products if used in waters with high organic content. UV radiation also cannot provide residual disinfection and distribution systems downstream of the disinfection unit are susceptible to re-contamination.

Chlorination is by far the most commonly used disinfection method in South Africa and is also used at all of UW's bigger water treatment works (Thompson, 2013). Also, substantial storage (12 h) capacity on site is provided necessitating residual disinfection. We thus recommend and have based our design on a chlorine gas disinfection system using 1 ton cylinders, as typically reflected in **Figure 16**.

Figure 16: Typical large-scale chlorine gas disinfection system (Waterwise)

It is generally understood that the active constituent in chlorine disinfection is hypochlorous acid, which forms when chlorine dissolves in water:

$$Cl_2 + H_2O \rightarrow HCl + HOCl \qquad \text{..............5}$$

For efficient disinfection at the uMkhomazi WTW we have based our design on the following considerations (refer to P&ID 1301.X01.UW-001 Sheet 4 of 5):

- Chlorine gas will be used for disinfection. This is a well-known technology and operators of UW plants will be familiar with operating procedures. Chlorination will also provide residual disinfection;
- A minimum contact time of 20 minutes will be provided for effective disinfection (Barnes & Wilson, 1983);
- Water for backwashing of the RGSF and GAC filters will be abstracted from the chlorine contact tank after disinfection and pumped to the backwash water storage tanks. From there it will flow via gravity to the filters to be backwashed. Water from these storage tanks will be fully treated and disinfected and will also be used for domestic purposes such as drinking water, toilet flushing and showers at the water treatment works itself;
- Two equally sized chlorine contact tanks will be provided for Phase 1 (500 ML/d) and Phase 2 (1 250 ML/d) for plant symmetry and ease of construction. The tanks will be constructed below ground, with chemical storage and make-up station buildings directly above the tanks to reduce the footprint of the overall plant;
- Each chlorine contact tank will have dimensions of 100 x 20 x 4.5 m height for an effective volume of 9 000 m^3, which will allows for 20 minutes contact time at full flow;
- An 80 000 m^3 intermediate water storage tank, integrated with the chlorine contact tank, will be provided underneath the chemical storage and make-up building. This will serve as emergency storage to have sufficient backwash water for backwashing the sand and carbon filters for 48 h, should the plant be out of operation. In addition, should maintenance be required downstream of the intermediate storage tank, a 3 h buffer capacity will be available;
- A final water reservoir will be provided below the sand and carbon filters, sludge treatment plant and chlorination facilities, again to reduce the overall footprint of the plant;
- The chlorination system will include chlorinators, injectors, vacuum regulators, booster pumps, flow regulators, chlorine drum handling gear and safety equipment. The expected chlorine consumption is shown in **Table 3**;
- The chlorination room will be fitted with a chlorine gas scrubber, to ensure that chlorine leakages can be taken care of immediately.

6.3.7 Sludge Thickening and Dewatering

The waste sludge from the high-rate clarifiers and backwash water from the RGSF and GAC filters gravity flows to a sludge thickening and dewatering facility. This wastewater consists of various streams with different sludge consistency, from ca 3% to 8% DS from the clarifiers to less than 0.2% DS from the filters. The objective of the dewatering and thickening facility is to first obtain a blended sludge with more or less uniform consistency. This sludge will then be thickened and dewatered as far as possible to give a waste product high in solids for disposal off-site, while recovering as much wastewater as possible, which will be recycled back to the plant, at the inlet works of the WTW. The sludge handling facility consists of two unit treatment processes, *viz.* new generation sludge thickeners and belt presses.

Blending/homogenisation. The sludge entering the sludge handling facility consists of various streams with different sludge consistency. In order for the sludge thickeners to operate optimally, a uniform or homogeneous sludge first needs to be produced from the various sludge streams. This is done in a sludge holding tank, where blowers are used to mix the sludge and to obtain a uniform sludge concentration throughout the tank. Without blowers, the sludge would settle to the bottom of the tank and a uniform concentration for downstream processing in the sludge thickeners would not be achieved.

Sludge Thickening. The process for sludge thickening is almost identical to the high rate clarification process with sludge recycling in a sludge contact clarifier as described in Section 1.4.3.3. For clarification, the desired result is to get the liquid component as pure and free of solids as possible, while the solid component is discharged for further treatment. For sludge thickening, this is reversed. The aim is to get the solids component as concentrated (dewatered) as possible and discharging the liquid component back to the inlet of the plant. Advanced coagulation and flocculation methods used for clarification are also used for sludge thickening, with solid and liquid components eventually being separated in a lamella clarifier. The clarifier underflow draw-off is the thickened sludge, which is then further dewatered typically in centrifuges or belt presses, while the clarifier overflow is returned to the inlet works to be treated in the WTW. As with high-rate clarifiers, various water treatment companies have successfully developed their own specialised proprietary sludge thickening technologies. Amongst these, the Veolia, Dégremont and Siemens systems have proven to be very successful.

The envisaged total wastewater discharge at the uMkhomazi WTW, stemming from the clarifiers, RGS and GAC filters is shown in **Table 15** below. Whereas filter washwater is discharged non-continuously, the below table gives the average hourly wastewater flows as generated throughout the plant and further to be treated in a continuous sludge treatment facility.

Table 15: Sludge handling facility inflow sources and quantities

Source	PHASE 1 Flow rate (max)	PHASE 2 Flow rate (max)
High-rate clarifier sludge	232 m^3/h	580 m^3/h
RGSF backwash water	480 m^3/h	1 200 m^3/h
GAC filter backwash water	61 m^3/h	153 m^3/h
ACTUAL TOTAL	**773 m^3/h**	**1 933 m^3/h**
Safety margin	27 m^3/h	67 m^3/h
DESIGN CAPACITY	**800 m^3/h**	**2 000 m^3/h**

We recommend and have based the conceptual design on the following design considerations (refer to P&ID 1301.X01.UW-001 Sheet 5 of 5):

- Each 250 ML/d train will require a 400 m^3/h sludge thickening unit. This translates to 2 units with a total capacity of 800 m^3/h for Phase 1 (500 ML/d) and 5 units with total throughput capacity of 2 000 m^3/h for Phase 2 (1 250 ML/d);
- The influent solids concentration to the thickening plant varies from 5 to 10 g/l DS (w/w), depending on the turbidity of the raw water feed to the WTW;

- New generation sludge thickeners will be used, which make use of enhanced coagulation using organic polymers and flocculation techniques as well as lamella clarifiers with sludge recirculation in order to increase the settling velocity of particles to ultimately reduce the footprint of the sludge thickening plant compared to conventional sedimentation/ thickening techniques. As mentioned, a very strong focus of the design is to keep the WTW footprint to a minimum and thus high-rate technology is recommended.
- As for high-rate clarification an organic polymer will be dosed at the thickeners in order to aid the flocculation process;
- Water recovered during the dewatering process is returned to the inlet works of the WTW in order to reduce water wastage;
- The thickened sludge must have a solids concentration of approximately 5 % - 8 % DS (w/w), which is the optimum feed concentration for further sludge dewatering, after sludge thickening.

Sludge Dewatering. After thickening, the sludge from the sludge thickeners needs to be further dewatered to reduce the total volume of waste sludge and to recover as much water as possible. This can be done using various technologies, typically incineration, centrifuges and belt presses are used. Incineration produces a final ash as waste product while belt presses and centrifuges produce a final dewatered sludge that can be finally disposed of in a landfill site, reused for agricultural purposes or as a base material in the brickmaking process. For a plant of this size, sludge management is of crucial importance as reuse and disposal options are very limited for the large quantities of sludge that will be produced daily. The aim is therefore to reuse the waste sludge as far as possible. Sludge that has been dried to 50 % DS (w/w) can feasibly be used in the manufacturing of bricks and will be addressed as another possible option in more detail later in this section. This will alleviate two problems: Firstly, the problem of what to do with the high quantities of sludge and secondly, this disposal option will actually generate income through the sale of bricks, as opposed to other disposal options that will be cost-negative. **Table 16** compares centrifuge and belt press technologies for the final dewatering of sludge. Capital and O&M Costs have been calculated for unit production costs taking Net Present Value (NPV) into account as discussed in **Section 9** of this report. The cost comparison is based on the full plant capacity of 1 250 Ml/d.

Table 16: Comparison of sludge drying technologies

Operational Parameter	Centrifuges	Belt presses
Operational control	High	High
Adaptability to changing operation	Reasonable	High
Skilled maintenance required	Very high	High
Sophistication of operation	Very high	High
Dry solids composition in final sludge	25%	50%
Feasibility of dried sludge for brickmaking	Low	High
Power Installed [kW] for 1 250 ML/d plant	820	30
Capital cost		
• Civil	9 800 000	9 800 000
• Mechanical	36 000 000	41 600 000
TOTAL	**45 800 000**	**51 400 000**

Annual cost • Civil redemption • Mechanical redemption • Power • Maintenance **TOTAL**	566 735 3 237 880 2 588 697 1 489 000 **7 882 312**	566 735 3 741 550 116 461 1 713 000 **6 137 746**

Belt presses nowadays achieve a dried sludge with 50% DS content. Not only is the final volume of waste sludge halved and the operation cost of using belt presses less than that of centrifuges, but the final product from the belt presses can be used in a brickmaking process. The potential to use dewatered sludge for brick manufacture, together with the fact that belt presses only use ca 4% of the total power that centrifuges require, makes this technology a very attractive option. Using belt presses for dewatering is therefore the most feasible option, from both an economic and an environmental point of view, and should be seriously considered by UW not only for this plant, but also when upgrading the sludge dewatering technology of existing plants. However, this technology as applied to drinking water sludge is fairly new despite being implemented at many new WTWs globally. It has also not been tested at any of UW's plants yet. The benefits of belt presses over centrifuges are substantial and it is therefore recommended that a pilot study using this technology is conducted as soon as possible to verify if UW could also benefit from such technology.

An alternative technology to belt presses is centrifugation. Centrifuges are only capable of achieving approximately 25 % DS (w/w), which results in ca. double the volume of waste sludge produced at a specific WTW, compared to belt presses .

Another alternative would be spray irrigation of only slightly thickened sludge for land application and agricultural reuse. This option was also considered for the uMkhomazi WTW (see later). However, for large plants this alternative is not viable due to the high volumes of sludge produced that require very large land areas for spray irrigation and also due to significant water losses experienced when only slightly thickened sludge is produced. For a plant with the size of the new envisaged uMkhomazi WTW, we therefore recommend belt press technology and have based the conceptual design on using belt press technology for sludge dewatering and drying.

For the EIA and estimates of sludge volumes envisaged to be produced in the new uMkhomazi WTW, the median SS value as per UW's estimate (Appendix A10) for the raw water as measured at Lundy's weir was used, as instructed by UW (Subramanian, 2015). The author is of the opinion that these volumes are too low and should at least be doubled, because:

- The envisaged retention time of the Smithfield Dam of 100 days is very short, which will mainly result in silt removal from the inflow, but not turbidity removal due to colloidal particles, in the final outflow;
- The catchment area shows that large tracts of soil with high clay content occur, which will result in fine colloidal matter that will remain in the impoundment (Hodgson, 2013).

Table 17: Estimated average sludge quantities based on UW estimates (Subramanian, 2015)

	Units	500 ML/d Phase 1		1 250 ML/d Phase 2	
DS in final sludge	ton/day	8.95		22.38	
Dewatered to % DS	% (m/m)	25	50	25	50
Wet Sludge Volume	m^3/d	31.83	13.92	79.58	34.80
Wet Sludge Mass	ton/d	35.81	17.90	89.53	44.75

The conceptual design was based on the following design considerations:

- Belt press technology will be used for sludge drying purposes as this is the most effective technology currently available for a large plant such as the new envisaged uMkhomazi WTW, taking into account that the dried sludge could be used for brick-making;
- Final sludge when using belt presses will have a solids concentration of ca 50% DS (m/m);
- The liquid component (eluate) will be recovered by returning it to the inlet works of the plant;
- Sludge dewatering is a vital component of the plant and ample standby capacity will be required to ensure that sludge can be treated at all times. Therefore, 6 duty and 2 standby belt presses will be provided for the full plant capacity of 1 250 ML/d, so that maintenance can be performed without interrupting operation.

6.3.8 Final Sludge Disposal

Three options considered for final disposal of the sludge produced at the uMkhomazi WTW were: Disposal to a suitable landfill site; agricultural land application; incorporating and reusing the dried sludge for brickmaking. For landfill disposal and also for brick making, it was assumed that a 50% (m/m) DS sludge will need to be disposed of; for agricultural land application, it was assumed that only 25% (m/m) DS sludge will be produced in order to have it in a more dilute form for mechanical application (spraying).

6.3.8.1 Option 1: Landfill

Dewatered sludge may be disposed of at a landfill site that has been designed with specific consideration for volume and characteristics of sludge, design life of the WTW and leachate generation and management. An analysis for design requirements specifically for the uMkhomazi WTW revealed that a G:L:B+ type landfill would be required (see Appendix A for details). This three-letter classification is based on type of sludge, size of landfill site and leachate management requirement, respectively.

For comparison, it was assumed that this sludge will consist of 50% (m/m) DS, be non-hazardous and thus a General (G) landfill design can be adopted. Approximately 45 tons wet sludge (at 50% DS) per day will need to be disposed of at the landfill site (see Appendix A.10). The leachate management requirements were determined by taking moisture content of sludge and historical evaporation data into account (Appendix A.2), which determined that significant leachate will be produced (classified as B^+) and an appropriate leachate management system will be required. Co-disposal of waste with solid and liquid components such as sludge is allowed at a G:L:B$^+$ site as long as proper leachate management is performed. The co-disposal ratio is affected by various factors and needs to be calculated after a specific landfill site is selected. A detailed example calculation is presented in Appendix A.3 for a lift of 5 m for sludge with a field capacity of 50%. Detailed requirements for the design of the G:L:B$^+$ landfill with regard to lining, leachate collection system, capping and final cover are presented in Appendix A.4-A.9. It was assumed that a suitable landfill site can be established within 30 km from the plant and transport to the site, 30 km away, was allowed for.

6.3.8.2 Option 2: Agricultural land application

Umgeni Water presently disposes of the sludge generated at Midmar WTW by a process called land application. Knight Piésold carried out a brief case study on the Brookdale Farm operation, approx. 3.5 kilometres from Midmar WTW (see **Figure 17**), with the intention of assessing its relevance to the proposed uMkhomazi project. To this end, the superintendent of Midmar WTW, Mr. T. Mdlalose was interviewed and a site visit was conducted to the Midmar WTW sludge handling facility as well as Brookdale Farm.

Figure 17: Aerial image of Brookdale Farm

- Details of the Brookdale Farm operation. Brookdale Farm was purchased by UW for the purpose of land application of the Midmar WTW sludge. UW as the owner leases the property to a farmer. The lease agreement gives Umgeni Water the right to dispose of the WTW sludge on areas of the farm that are not in productive use over the period of time that sludge is applied to that portion of the land. Under the present lease agreement, it is the responsibility of the farmer to collect sludge at an agreed frequency from Midmar WTW.

  Sludge generated at the Midmar WTW is dewatered by means of a centrifuge to a 25% DS content. The farm currently receives approximately 6 loads of sludge per day, i.e. 18 m^3/day or 21.6 t/day. The sludge is transported by road in a 'muck spreader' pulled by a tractor (see **Figure 18**).

Figure 18: Typical Rotor-spreader “Muck” spreader used on the Brookdale Farm

A muck spreader is an agricultural machine typically used to distribute manure over a field. A typical muck spreader consists of a tractor which tows a trailer with a rotating mechanism driven by the tractor's power take off (PTO). The muck spreader currently in use at Brookdale Farm has a capacity of three cubic metres.

A typical application rate of 76 t/ha of wet sludge is presently achieved. The sludge is allowed to air dry after application for two months before the next application cycle (Moodley, 2001).

The farm is divided into 4.5 Ha blocks of land, each containing 65 strips approximately 690 m² in size. The strip size has been calculated to roughly match the area covered in a single run when the tractor pulls the muck spreader in 1st gear at 2 000 r.p.m. Once the 4.5 Ha block has received the equivalent of 128 t/Ha of dry sludge it is returned to its former land-use and another 4.5 Ha block is identified for further sludge disposal. The case study determined that it takes approximately 2 years of continuous sludge disposal with the 2 month drying period per strip for the 4.5 Ha block to achieve the 128 t/Ha maximum advisable coverage.

- <u>Extension of the Brookdale Farm sludge handling concept to the proposed uMkhomazi Project.</u>
Although Brookdale Farm was purchased by Umgeni Water to provide a ‘guaranteed’ disposal area for the Midmar WTW sludge, this may not necessarily be the case for the uMkhomazi scheme. Phase 1 (for 500 Ml/d) of the uMkhomazi Project will generate an estimated 35.8 t/day of wet sludge with a total solids content of 25%, which is comparable with the 21.6 t/day of sludge presently generated at Midmar WTW (for 350 Ml/d).

  For landfill application, the sludge needs to be relatively thin. Only sludge with 25% dry solids content was considered, as is presently the case with the Midmar WTW sludge. At this stage, it has been assumed that no land would have to be purchased by UW for this purpose. The sludge would be given to farmers in the region free of charge for them to utilize on their land. Delivery may be in the form of large capacity tip trucks or even by pumping of the sludge as slurry. For the purposes of this study, road transport has been assumed for discarding ca 40 t of sludge per day for Phase 1 and 90 t per day when Phase 2 has also been added.

By applying the techniques used at Brookdale Farm to the proposed Umkhomazi WTW, it was possible to estimate the total area that would be required for the disposal of sludge generated from the proposed treatment process.

Applying the present application rate at Brookdale, it has been calculated that a total area of ca. 1.5 Ha would be required per day.

If the same methodology and drying period that is currently used at the Brookdale farm is applied to these proposed sites, land parcels of ca. 4.5 Ha each would need to be identified. Each land parcel would then be further divided into ca. 82 strips ca. 55 m^2 in size. The area of each strip is sized to match the volume of sludge that can be distributed in a single run by a 4.2 m^3 muck spreader, which is the largest capacity muck spreader commercially available in South Africa.

Once the 4.5 Ha block receives the recommended load for each rotation cycle, i.e. 12.8 t/ha over 2 years as is the case at Brookdale Farm, it would be returned to its former use and another 4.5 Ha block would have to be identified for further sludge disposal. The rotation cycle would be dependent on the soil characteristics as well as the levels of phosphorus present in the sludge.

The rotation cycle however, is also dependent on the commercial need to develop the portion of the farm receiving the sludge, i.e. the timing of planting crops on that piece of land may not coincide with the time required to complete the land application process to the optimal coverage.

Table 18 compares the Midmar WWTW and proposed uMkhomazi WTW land application requirements.

Table 18: Comparison of Midmar WTW and proposed uMkhomazi WTW land application

Midmar Water Treatment Works - Brookdale Farm Land Application Initial Tests	
Volume of sludge produced per day	6.5 tonnes/day
Percentage Solids contained in Sludge to be spread	24 - 28 %
Capacity of Spreader	5.25 m^3
Application Rate of Spreader	7.6 kg/m^2 of wet sludge in 1st gear at 2000 r.p.m
Coverage area	690 m^2 (3 m wide x 230 m long)
Drying Period	2 months
Total Farm Area	126 hectares
Total Usable Area	9.32 hectares
Rotation Cycle	2 years
Maximum Loading per Cycle	128 tonnes/hectare
Umkhomazi Water Treatment Works - Land Application Estimated Quantities for Sludge Containing 25% solids – PHASE 1 + 2 (1 250 Ml/d production)	
Volume of sludge produced per day	90 tonnes/day
Percentage Solids contained in Sludge to be spread	25 %
Capacity of Spreader	4.2 m^3
Estimated Application Rate of Spreader	6.08 kg/m^2 of wet sludge in 1st gear at 2000 r.p.m
Estimated Coverage area	552 m^2 (3 m wide x 184 m long)
Estimated Drying Period	2 months
Estimated Total Daily Usable Area Required	1.5 hectares/day
Estimated Rotation Cycle	dependent on levels of Phosphorus present in sludge and soil characteristics
Estimated Maximum Loading per Cycle	dependent on soil characteristics and intended crops

- Conclusions. The total area of farmland required to make land application viable over each two year cycle is ca 90 Ha. If it is assumed that the sludge will be disposed of by land application on farms within a 15 km radius of the WTW, less than 0.2% of the available farmland within this radius will be required at any given time for the purposes of land application.

  After this two-year period, the land will be released for cultivation and new portions identified for land application. It is possible that the land application cycles could also be timed to coincide with existing crop rotation cycles.

  The option to dispose of the uMkhomazi WTW sludge by land application therefore appears to be viable. When the WTW is operational and once the volumes of sludge are known more accurately, Umgeni Water would need to take a decision on whether to purchase farmland for the purpose of land application or to sign agreements with farmers to accept the sludge onto their land.

6.3.8.3 Option 3: Brickmaking

It was assumed that the final, dewatered sludge will be handed over to an existing brick maker in the closer vicinity of the new plant (within 15 km), who will be able to use the sludge instead of base material (see discussion later). Thus, no new land will need to be acquired in the vicinity of the plant and the brickmaking process with subsequent sales will be viable to carry all costs associated with final disposal of the sludge.

A comprehensive study into the feasibility of using the uMkhomazi WTW's waste sludge for brickmaking was undertaken especially taking South African conditions into account. For this, assistance from a specialist brickmaker, *viz* Paarl Brickfields (Mr A Esterhuizen, 2014) was obtained and the following section is based largely on this study.

WTP waste sludge that mainly utilised alum as primary flocculant has a similar composition to that of natural clay. Substitution of natural clay with this waste sludge has been done successfully with up to 50% sludge:clay mixes (Esterhuizen, 2014). A key consideration is the dry solids content of the sludge used in the brickmaking process. The lower the dry solids content, the more water needs to be removed during the brick firing (baking) process, which requires more energy input. Thus, a higher solids content sludge is preferred.

- Location. The intended uMkhomazi WTP is situated 45km west of Pietermaritzburg, meaning that it is surrounded by the Lower Ecca Group or Pietermaritzburg Formation of shale and sandstone (**Figure 19**). This is the main source of clay for the larger clay brick manufacturers in the area – there are 3 particularly large manufacturers in the area – and the abundance of iron oxides in the clay provides the rich red colouring on some of their products.

  Very little additives are required in this type of clay due to the natural plasticity and green strength during extrusion as well as compressive strength after firing. It must be noted that the green strength (the strength of the green extruded brick that needs to be dried then fired/burnt) is assisted by a relatively large amount of quartz present in the clay. The location of the WTW is thus logistically suitable with regard to the supply of raw materials and the physical access to the required markets. The type of clay abundant in the area lends itself well to addition of wet substance since it has a superior green strength. In addition to the above, the relatively close sources of coal in KZN as main energy driver for the process, provides an operational advantage in that the delivered price of coal is not as inhibitive as it is, for example, in the Western Cape.

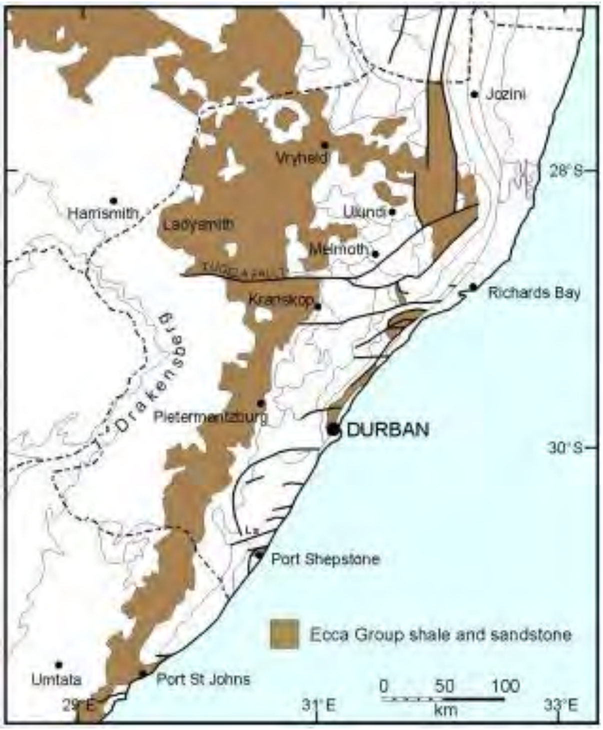


Figure 19: Ecca Group shale and sandstone formation in the Pietermaritzburg area (UKZN)

- Process. Smaller constituents in the waste sludge, such as organic content and other waste components, do not play a significant role in the actual quality of the final brick (due to the type of manufacturing process employed) but will influence the look/colour of the final product. The dryness of the waste sludge, on the other hand, has a major impact on the amount of bricks that needs to be manufactured to capture all the sludge. To ensure the final brick product has qualities equivalent to that minimum required in SANS 227, particularly fired compressive strength and water absorption, the cumulative amount of clay and waste sludge in the green brick (on a dry solids basis) should not be below 50% to 55% otherwise the green strength would be below the minimum threshold to ensure the handling of the wet brick without high % of wastage. The aim is to get to 70% to ensure the minimum wastage during extrusion and wet brick handling. It is fortunate that the high percentage of quartz in the natural clay of the Ecca Formation will impart a good portion of green strength during manufacturing.

  In order to use the sludge generated at the uMkhomazi WTW for brickmaking will require a dewatering process that can ensure 50% DS is achieved. Thus, centrifuges can not be used for this purpose because they only achieve a 25% to 35% DS content, whereas belt presses can produce sludge with 50% to 55% DS content.

- Conclusions. It is very difficult to determine, at this early stage, if there will be a brick maker in the vicinity prepared to accept sludge from the proposed uMkhomazi WTW for brick making as brick makers would need to sample the sludge first to assess if this can be used without detriment to their manufacturing process. However, since this would be the most environmentally friendly option and would result in no additional costs to UW other than transporting costs of the sludge to the brick making plant, this option should be further pursued during the detail design phase.

- For Option 1, if the landfill area is rated at a zero cost item because it is shared with other UW plants, the only routine operating additional costs for UW would be the transport costs to the landfill site itself. If such a site is developed within a 30 km radius from the plant as discussed, the average annual transport costs would be approximately R4 359 408 for the full 1 250 Ml/d plant and this figure was used for all further costs calculations.
- The above cost figure would also apply if the sludge is transported to farmers in the near vicinity and applied to the land as discussed in our Option 2.

7 Operation and Maintenance

Drinking water treatment plants require skilled personnel for successful operation and maintenance. The more complex the treatment processes and technologies employed at the plant, the more skilled the process controller and operator(s) need to be. Even though the proposed uMkhomazi WTW consists of conventional treatment processes with technologies that UW operators will be mostly familiar with, the high-rate clarification process will be new to them and additional skills will have to be developed for personnel operating at this plant to ensure optimum plant performance and the safe supply of drinking water at all times. However, sufficient control will be incorporated to ensure that, if the water after clarification goes out of specification in a particular train, this train will be shut off and a warning given to the operator.

The on-site sludge treatment facility could be treated as a standalone operation, with specially trained operators and technicians. The sludge treatment facility manager would report to the WTW plant manager, but from an organizational point of view, the two facilities would be independent.

Personnel.

Figure 20 shows the proposed organogram for the WTW and the sludge treatment facility. Operations, chemicals and security personnel will have shift teams for continuous, 24-hours a day operation of the treatment works. Plant operators and chemical handlers will have four teams that operate in 8 hour shifts while security will have three teams that operate in 12 hour shifts. Due to the plant not situated in or close to a town, UW will most probably have to permanently employ security staff, instead of employing a specialist contractor for this function. Critical equipment will be provided with standby units in case of failure, but maintenance teams will also be on stand-by for after-hours emergency breakdowns.

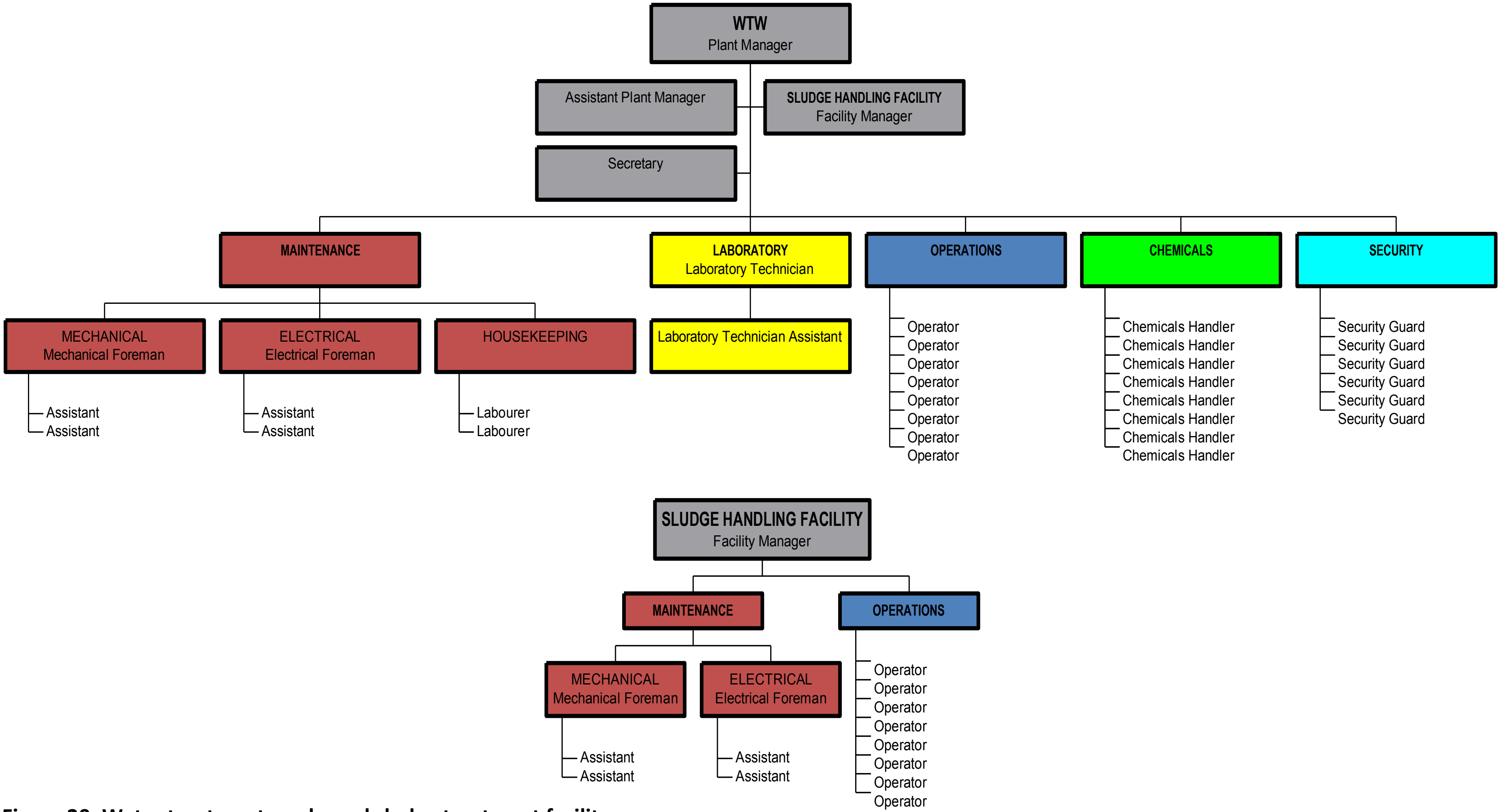


Figure 20: Water treatment works and sludge treatment facility organograms

Operation and Maintenance. The WTW must have a proper operating manual containing all the details necessary to successfully operate and understand processes and procedures of the plant. The manual should be properly bound and be available in the English language. The following information will be included in the manual, as a minimum:

- The commissioning procedure and plant settings after successful commissioning;
- All plant-related drawings and diagrams. This includes layout, mechanical, and piping and instrumentation drawings as well as electrical wiring diagrams and any other drawings which may be useful for plant operation and maintenance;
- Complete functional description of the process including the control philosophy;
- Illustrated operating instructions including start-up, shut-down, backwashing, regeneration and/or cleaning procedures and emergency actions to be taken in the case of possible equipment failures;
- Maintenance instructions to include the descriptions and required frequency of all maintenance tasks;
- Equipment data sheets and manufacturer's operation and maintenance instructions;
- Procedures for chemicals preparation with cautionary notes and clearly visible signage for hazardous chemicals. Clear instructions for emergency procedures to be followed in case of an accident involving chemicals must be easily visible and available;
- Chemicals suppliers contact details;
- Trouble shooting notes with contact details for emergency action;
- Suggested typical plant operating parameters, such as chemical dosing, flow rates and head losses. After commissioning, such values that are fine-tuned during the commissioning process should be included in the commissioning report and included in the operation and maintenance manual;
- Sample calculations where applicable.

Spares and Consumables. In addition to the regular checks and procedures to be followed, it is very important to keep stock of critical spares and consumables on the plant. In the event of failure of equipment that is crucial to the successful operation of the plant, a technician should be able to replace or repair such equipment with minimal or no plant down time. Stock levels of consumables and chemicals should also be managed carefully in order to ensure that sufficient time is allowed for re-ordering and delivering new supplies. Typical spares to be kept on site include pumps, valves, pipes and fittings, instrumentation and service kits for major equipment.

Asset inventory. An asset inventory helps water services providers to identify what assets they own, where these assets are located or stored and what their condition and service history is. This data needs to be catalogued in a logical, readable format such as a handwritten list, spreadsheet software, database software or even commercially available asset management database software for very large plants. These lists can be drawn up for installed equipment, chemical supplies as well as for general stock available at the plant.

A simplified example of an asset management table for plant equipment is illustrated in **Table 19** below.

Table 19: Simplified example of an equipment asset management list

Equipment Tag No.	Description	Date Installed	Condition	Expected Year of Replacement	Comments
PS-201 A	Submersible Drainage Pump	2004	Fair	2019	Coil repaired Aug 2007
MV-708	Actuator	2009	Good	2024	
LE-1305	Ultrasonic Level Sensor	2013	Bad	Needs replacement	Electrical damage
TK-212	HCl Tank	2002	Fair	2022	Corrosion on bolts

Safe Operation. The main health and safety concern is that the WTW produces a final water that complies with the prescribed water quality standards at all times. Failure to comply with the required final water quality must result in immediate plant shutdown until remedial action is performed to rectify the factor(s) causing the non-compliance. The WTW needs to monitor its final water quality in order to ensure that the consumer receives a safe drinking water supply at all times. This is done with regular sampling and testing in either the on-site or an independent laboratory.

The WTW will need to keep records of plant performance regarding final water quality achieved, with minimum sampling frequencies as set out in the applicable water quality standards. The results of these analyses must be available at any time for auditing by the Department of Water Affairs in order to ensure the safe supply of drinking water. The following records need to be available and may be requested by DWA during a plant audit:

- Logs of final water quality, with minimum sampling frequencies as prescribed;
- Proof of valid license. This license needs to be renewed as necessary;
- Proof of operator and process controller qualifications and attendance registers, as proof that the minimum operator and process control qualification requirements have been met.

8 Environmental, Safety and Health Aspects

The uMkhomazi WTW needs to adhere to all relevant local Acts regarding the operation and environmental impacts of the plant. The plant must provide a safe drinking water at all times while having a minimum impact on the environment.

Both the construction and the operation of the uMkhomazi WTW need to be compliant with the National Environmental Management Act (NEMA) (Act no. 107 of 1998). A full Environmental Impact Assessment (EIA) will need to be performed prior to approval of the project to determine its environmental implications. The following aspects are especially important for the environmental analysis:

- Quantities and nature of chemicals used at the WTW. Emergency preparedness plans, safety equipment and emergency clean-up procedures need to be in place in case of a spillage. Chlorine gas that is used for disinfection is a particular concern, as this is a highly toxic gas and can have severe health and environmental impacts if leakages occur. The chlorination equipment including the chlorine drums will be contained within a separate building, away from any other chemicals. All relevant safety notices and safety equipment will be available at this building. The chlorination system will also be fitted with automated leakage detection systems and a chlorine scrubber, which will ensure that any leakages are automatically and safely cleaned.
- Waste material disposal. All waste produced by the plant, including waste sludge from the process, domestic waste and sewage needs to be disposed of or treated in compliance with NEMA. Large quantities of sludge will be produced by the plant and a full assessment needs to be performed in order to determine the most suitable disposal or reuse method for this waste product. Recommended options include the use of sludge for brickmaking as this is a disposal method that can generate income. Other disposal options include land application or disposal at land fill sites. Whichever option is finally chosen needs to be fully investigated with an EIA.
- Construction. The construction process for such a large water treatment works is likely to last 2 – 3 years. During this time, care must be taken to ensure minimal impact on the environment and to ensure that all construction works comply with the relevant Acts regarding health and safety.

The uMkhomazi WTW will be operated by UW, which currently operates numerous other WTW's in the area. All current Health and Safety procedures and directives applicable to UW's other treatment plants will need to be revised for application at the new uMkhomazi WTW. UW will be responsible to obtain all applicable permits and licenses required for the operation of a drinking water plant and disposal of associated waste products.

9 Cost Estimates

Cost estimates were developed for the WTW infrastructure recommended in this report. The following costs are presented in this section.

- Capital costs
- Operation and maintenance costs, consisting of:
    - Chemical consumption;
    - Personnel;
    - Power;
    - Maintenance costs.
- Total annual costs

All the above costs were evaluated in order to develop a balanced perspective of future financial commitments. Annual costs are based on Net Present Value (NPV) for comparative purposes. All costs are based on 2014 rates and exclude VAT.

9.1 Capital Costs

For the purpose of estimating the WTW capital costs, a bill of quantities was drawn up for estimating material and equipment quantities for civil works, mechanical equipment and electrical and control items. These bills were then priced by contractors that would typically be involved in the construction of a plant of the size of the proposed uMkhomazi WTW.

Buildings at the WTW site will include a 450 m^2 control room with offices and boardroom, laboratory, operator change rooms and ablutions, 1 350 m^2 chemical make-up and dosing area, 1 800 m^2 chemicals and general storage areas and a security check-point building. Site services will include security fencing with access control, flood lighting, access road to the plant, sanitation, safety equipment and adequate drainage.

In addition to the WTW infrastructure costs, the bulk earthworks required at each site was priced. A summary of the earthworks calculations is included in **Appendix B**. Drawings showing preliminary earthworks designs for each site are included in **Appendix D**.

Specialist civil contractors and mechanical and electrical (M&E) contractors were requested to submit budget prices and

Table 20 is a summary of project costs as given by these contractors (only).

it should be noted that costs for Phase 2 are not in addition to Phase 1, but are cumulative for the full 1250 Ml/d plant. Capital costs for rapid gravity sand filtration and granular activated sludge filtration include the first charge of media.

Table 20: Estimate of capital costs for uMkhomazi WTW

Item	Phase 1 (500 ML/d)	Phase 1 & 2 (1 250 ML/d)
1. Preliminary & general	33 804 477	60 927 208
2. Site clearance and preliminary work	18 079 990	79 633 138
3. Inlet works:		
Civil works	1 714 371	1 714 371
M&E	1 781 880	1 781 880
4. Clarification:		
Civil works	117 527 020	146 908 775
M&E	113 406 298	146 460 928
5. Rapid gravity sand filters:		
Civil works	32 508 875	81 272 188
M&E	56 028 485	135 667 523
6. GAC filters:		
Civil works	36 269 051	83 850 105
M&E	196 889 495	486 059 785
7. Tanks - chlorine contact, washwater and final water storage:		
Civil works	104 774 459	209 548 917
M&E	2 230 616	4 461 236
8. Sludge treatment:		
Civil works	16 935 472	36 116 565
M&E	47 293 441	96 923 615
9. Chemical dosing equipment	11 215 858	22 431 738
10. Instrumentation and laboratory	5 820 576	11 641 163
11. Electrical and control	75 478 788	116 121 328
12. Facilities and buildings	79 822 720	150 481 577
13. Finishes and miscellaneous civil works	62 870 492	122 532 654
14. Drainage and stormwater	2 104 355	3 142 455
15. Access roads and paving	6 809 880	11 683 518
16. Fencing	3 504 420	3 504 420
SUB-TOTAL	**R 1 026 871 018**	**R 1 956 413 144**

Capital costs from

Table 20 have been summarized into the broader disciplines of civil, buildings and mechanical and electrical (M&E) works and 10% contingencies and professional fees were added to assess final project cost, which are reflected in **Table 21**.

The power requirements for the WTW were provided to Eskom who in turn determined the electrical infrastructure requirements and associated budget costs. Of the three pricing options received from Eskom, the option that provided the most stable power supply to the WTW was selected. This option is also the most expensive and therefore caters for the worst case scenario in relation to power supply.

The budget power supply costs provided by Eskom are included in **Appendix C**.

Table 21: Summary of total capital costs

	Cost for Phase 1 (500 Ml/d) [R]	**Cost for Phase 2 (1 250Ml/d) [R]**
Civil works (excl buildings)	R 436 902 862.00	R 780 834 315.00
Buildings	R 79 822 720.00	R 150 481 577.00
M&E equipment	R 510 145 436.00	R 1 025 097 251.00
Bulk Earthworks	R 118 498 432.37	R 236 996 864.75
Power Supply Cost	R 47 873 205.80	R 95 746 411.60
TOTAL	**R 1 193 242 656.17**	**R 2 289 156 419.35**

9.2 Operational and Maintenance Costs

The operational and maintenance costs will include the individual costs contributed by chemical consumptions, power consumption, personnel and maintenance expenses.

9.2.1 Chemical costs

The estimates of the annual chemical costs are based on average dose rates, which were determined either with flocculation tests or estimated from other drinking water plants operated by UW in similar circumstances. Initially, the plant will be built without a Granular Activated Carbon (GAC) filtration treatment step, but this may become necessary at a later stage, as described in Section 1.4.3.5. Provision has therefore been made in the design to incorporate GAC filtration into the treatment plant. GAC needs to be replaced ca every 2 years and these recharge costs contribute significantly to the overall chemical costs. Due to the fact that the initial plant will be built without GAC filtration, the chemical consumption costs have been indicated for two options, namely with and without GAC recharge. **Table 22** summarises the total annual chemical costs.

Table 22: Estimate of total annual chemical costs

Chemical	Average Dosage [mg/l]	Unit Cost [R/kg]	Annual Cost [Mil R/annum] Phase 1 (500 Ml/d)	Phase 2 (1 250 Ml/d)
Lime (Stabilisation)	10	2.674	4.88	12.20
Lime (Sludge thickening), based on sludge volume	150	2.674	0.34	0.85
Potassium permanganate	1	38.468	7.02	17.55
Alum	15	3.450	9.44	23.61
Polymer (for Clarification)	1	9.994	1.82	4.56
Polymer (Sludge thickening), based on sludge volume	Based on DS sludge	9.994	3.02	7.55
Bentonite	3	4.589	2.51	6.28
Chlorine (pre- and post-chlorination)	3.5 (1.5 mg/l pre-chlorination, 2 mg/l post-chlorination)	14.722	9.40	23.51
GAC replacement	Once every 2 years	40	43.2	108
TOTAL excluding GAC replacement [Mil R/annum]			**38.44**	**96.11**
TOTAL including GAC replacement [Mil R/annum]			**81.64**	**204.11**

9.2.2 Power Costs

The total estimated power consumption is shown in **Table 23** below, with corresponding power costs shown in **Table 24**. Some components are operated for less than 24 hours a day to keep the consumption to a minimum. Installed power is based only on duty units, even if additional standby units are given. Absorbed power was calculated for each component individually according to the operating hours and component's efficiency.

Table 23: Estimate of total annual power consumption

Process/Activity	Phase 1 (500 ML/d)		Phase 2 (1 250 ML/d)	
	Duty inst. power [kW]	Total abs. power [kWh/day]	Duty inst. power [kW]	Total abs. Power [kWh/day]
Inlet works and chemical dosing:				
Lime feeder	3 x 5.5	317	6 x 5.5	634
$KMnO_4$ feeder	3 x 5.5	317	6 x 5.5	634
Flocculation and clarification:				
Bentonite feeder	2 x 5.5	211	2 x 5.5	422
Alum feeder	3 x 5.5	317	6 x 5.5	634
Coagulation mixers	16 x 45	14 688	20 x 45	18 360
Fast flocculation mixers	16 x 3.2	983	20 x 3.2	1 229
Slow flocculation mixers	16 x 2.2	676	20 x 2.2	845
Scraper bridge motor	16 x 7.5	2 448	20 x 7.5	3 060
Sludge recycle pumps	16 x 5.5	1 584	20 x 5.5	1 980
Rapid gravity sand filtration:				
Backwash air blowers	4 x 75	510	8 x 75	1 020
Valve actuators (sum)	20 x 3.35	54	50 x 3.35	134
GAC filtration:				
Valve actuators (sum)	24 x 3	72	4 x 75	600
Chlorine contact tank & final water storage:				
W/W reservoir pumps				
Chlorine booster pumps	2 x 30	765	4 x 30	1 530
Control valve	2 x 15	612	4 x 15	134
	1 x 2.2	42	2 x 2.2	84
Sludge treatment plant:				
W/W return pumps	2 x 150	6 120	4 x 150	12 240
Scraper bridge motor	3 x 1.1	63	6 x 1.1	127
Sludge mixing blowers	1 x 110	1 980	2 x 110	3 960
Sludge holding tank mixer	4 x 4	288	8 x 4	576
Belt presses	3 x 5	306	6 x 5	612
Small, non-continuous power including drainage pumps, hoists and cranes (sum)	1 x 26.4	25.5	2 x 26.4	51
Ancillaries including instrumentation, flood lighting of site, and workshop, office and laboratory (sum)	1 x 125	1 368	2 x 125	2 736
TOTAL	**2 202 kW**	**33 732 kWh/day**	**3 712 kW**	**52 235 kWh/day**

The estimated annual power costs consist of a passive power cost and an active usage cost. Time of use periods were taken as per Eskom's "Megaflex" pricing structure. **Table 24** summarizes the estimated total annual power costs for the new uMkhomazi WTW.

Table 24: Estimate of total annual power costs

	Phase 1 (500 ML/d)	Phase 2 (1 250 ML/d)
Total duty installed power [kW]	2 202	3 712
Total absorbed power demand [kWh/day]	33 732	52 235
Passive power costs [Mil R/annum]	1.652	2.806
Active power costs [Mil R/annum]	6.204	9.496
TOTAL [Mil R/annum]	**7.856**	**12.302**

9.2.3 Personnel Costs

The estimated costs for salaries of personnel directly associated with the operation of the plant have been allowed for. Any financial benefits, travel allowances and other indirect personnel costs are not included in the estimate. The water treatment works (WTW) and sludge treatment plant personnel have been considered separately, as a specialised full-time team will be necessary for the sludge handling facility. The total number of employees per category includes provision for shift work for continuous, 24 hour per day plant operation. The total personnel costs are summarised in **Table 25**.

Table 25: Estimate of total annual personnel costs

		Phase 1 (500 ML/d)		Phase 2 (1 250 ML/d)	
Job Description	**Monthly Salary [R]**	**Total employed**	**Total Annual Salary [R]**	**Total employed**	**Total Annual Salary**
WATER TREATMENT WORKS					
Plant Manager	50 000	1	600 000	1	600 000
Assistant Plant Manager	30 000	1	360 000	1	360 000
Operator	18 000	8	1 728 000	16	3 456 000
Labourer/Cleaner	8 000	2	192 000	4	384 000
Mechanical Technician Foreman	25 000	1	300 000	1	300 000
Mechanical Technician Assistant	10 000	2	240 000	2	240 000
Electrical Technician Foreman	25 000	1	300 000	1	300 000
Electrical Technician Assistant	10 000	2	240 000	2	240 000
Laboratory Technician	25 000	1	300 000	1	300 000
Laboratory Assistant	10 000	1	120 000	1	120 000
Security	8 000	8	768 000	16	1 536 000
Chemicals Handling Staff	8 000	8	768 000	16	1 536 000
Secretary	15 000	1	180 000	1	180 000
SUB-TOTAL		**37**	**R 6 096 000**	**63**	**R 9 552 000**
SLUDGE HANDLING FACILITY					
Facility Manager	50 000	1	50 000	1	50 000
Mechanical Technician Foreman	25 000	1	25 000	1	25 000
Mechanical technician Assistant	10 000	2	20 000	2	20 000
Operators	18 000	8	144 000	8	144 000
SUB-TOTAL		**12**	**239 000**	**12**	**239 000**
TOTAL		**49**	**R 6 335 000**	**75**	**R 9 791 000**

9.2.4 Maintenance Costs

Routine general and preventative maintenance costs were taken as a percentage of the capital costs (see **Table 21**) and are shown in **Table 26** below:

Table 26: Estimate of total annual maintenance costs

Component	Percentage of capital costs incurred per annum	Phase 1 (500 ML/d) [Mil R]	Phase 2 (1 250 ML/d) [Mil R]
Civil works	0.5	2.18	5.13
Buildings	1.5	1.20	2.26
M & E	4	20 41	41.00
TOTAL [Mil R/annum]		**23.79**	**48.39**

9.3 Total Annual Costs

The total annual cost associated with the construction, operation and maintenance of the uMkhomazi WTW is summarised in **Table 27** below, with GAC replacement costs also shown as an option (see section 2.2.1). Capital redemption was calculated for:

- Civil works - 4% over 45 years;
- Buildings - 4% over 45 years;
- Mechanical/electrical installation - 4% over 15 years.

Table 27: Estimate of total annual costs

Component	Phase 1 (500 Ml/d)	Phase 2 (1 250 Ml/d)
1. Capital redemption		
Civil works	21 086 005	37 684 982
Buildings	3 852 440	7 262 610
Mechanical & electrical	48 598 500	92 198 375
2. Chemical costs:		
Excluding GAC recharge	38 441 053	96 110 451
OR	OR	OR
Including GAC recharge	81 641 053	204 110 451
3. Sludge disposal costs (as per Section 1.4.3.7)	2 179 704	4 359 408
4. Power costs	7 855 532	12 301 953
5. Personnel costs	6 335 000	9 791 000
6. Maintenance	23 787 673	48 386 600
TOTAL Annual Cost excluding GAC replacement [R]	**149 420 449**	**308 095 380**
TOTAL Annual Cost including GAC replacement [R]	**192 620 449**	**416 095 380**
Production Cost excluding GAC replacement [c/m^3]	**81.87**	**67.53**
Production Cost including GAC replacement [c/m^3]	**105.55**	**91.20**

10 Summary

This conceptual design focused on raw water quality, selection of individual water treatment processes, plant layout and the economic implications of the construction and operation of the proposed uMkhomazi WTW.

For the Smithfield Dam water to be purified, a continuous treatment plant is recommended to treat the water according to conventional processes including coagulation, flocculation, sedimentation, filtration and disinfection. Emphasis is placed on high-rate processes that require as little footprint area as possible, in order to limit the amount of land that has to be expropriated from local farmers. In addition, processes that result in minimal headloss are preferred as only approximately 10 m of hydraulic headloss is available for the treatment plant in order to allow for gravity discharge from the dam to the WTW and from the WTW to the distribution system. The following water treatment processes and auxiliary facilities have been proposed:

1) Pre-chlorination, water stabilisation with lime and iron and manganese oxidation with potassium permanganate will be performed at or upstream of the inlet works of the plant. Mixing will occur inherently while water is transferred through the distribution tower, which distributes the raw water to the separate treatment plant trains;
2) Coagulation with alum will be done using mechanical mixing to achieve the desired mixing intensity. Hydraulic or static mixers are not recommended as they increase the total head required between the start and end of the WTW process. Hydraulic mixing should be reconsidered at detailed design stage if additional the WTW can be sufficiently elevated to create an opportunity for this type of mixing;
3) Flocculation with an organic polyelectrolyte will be done using mechanical mixing to achieve the desired mixing intensity. To reduce hydraulic losses, hydraulic mixing methods for flocculation have not been considered. It is however recommended that hydraulic mixing technology be reassessed at detailed design stage;
4) Clarification/sedimentation will be performed using high-rate clarifiers that may employ bentonite as ballasting agent and will include micro-flocculation or sludge recirculation for the rapid formation of heavy flocs. These high rate clarifiers significantly reduce the overall plant footprint;
5) For Phase 1, these high-rate clarifiers will operate at relatively low linear upflow rates of 4.5 m/h. It is recommended that UW gradually increase the throughput in order to meet the actual design upflow rates of 9.0 m/h by the time Phase 2 is being implemented, which will be 20 years after implementation of Phase 1;
6) Rapid gravity sand filters with a dual-media bed of anthracite and silica sand are recommended to ensure maximum floc penetration and filter run times. Double bed filters will be used with a normal filtration rate of 8.75 to 10 m/h. Backwashing will be done using both air and water;
7) Granular activated carbon (GAC) filtration is recommended as polishing step because the raw water presently shows DOC values above 1 mg/l, which need to be reduced.
8) GAC filtration has been allowed for in the plant design, but for Phase 1 this may not be implemented. The source water must be closely monitored with regards to TOC/DOC and if it becomes more enriched with nutrients the GAC polishing step will be necessary for the removal of organic material. GAC filters will have a double-bed configuration so that an upflow-downflow operation sequence can be achieved;

9) Chlorination using chlorine gas has been allowed for to also give residual disinfection capability to prevent contamination of the final water in the water distribution system;

10) Final water will be stored on site in an 80 000 m³ intermediate tank to serve the plant's final water demand, with a retention time of 3 hours. This will provide sufficient storage capacity to provide for emergency backwash water for sand and carbon filter backwashing for 2 days;

11) An additional potable water reservoir serving the distribution system downstream, has been provided for balancing storage;

12) Various auxiliary facilities have also been included in the WTW design. These will be vital in the successful operation of the plant:

    - Chemical storage and dosing of all chemicals coagulants and flocculants, including alum, potassium permanganate, lime, polyelectrolyte, bentonite and chlorine. Dry feeding of alum, lime, potassium permanganate and bentonite is suggested, while provision will be made for the preparation and dosing of dry as well as liquid polyelectrolyte;
    - Chlorination installation will allow for the application of chlorine to the raw water (pre-chlorination) as well as the final water (post-chlorination). The chlorination equipment will be housed in a separate building from all other chemicals for safety reasons. All necessary safety equipment as well as a chlorine neutralisation scrubber system need to be provided;
    - Clarifier underflow, sand filter backwash and GAC filter backwash water will be 46.4 ML/d (at 1% (m/m) DS content), which will be collected and treated in a dedicated sludge handling facility on site. The water recovered by this facility will be returned to the inlet works of the plant while the thickened and dried final sludge will be disposed of off-site;
    - The final, waste sludge produced will be 45 t/day at 50% (m/m) DS content and can only economically be produced when using belt press technology. If centrifuges are used, only 25% (m/m) DS will be achieved, resulting in double the volume of sludge that has to be disposed of;
    - Final sludge should be used for brick manufacturing. This will be the most environmentally friendly way to dispose of sludge and will reduce the overall carbon footprint of the plant;
    - Water for backwashing of the sand and GAC filters will be stored in a washwater reservoir on site. The reservoir is filled with chlorinated water from the chlorine contact tank;
    - Facilities at the plant will include a control room, laboratory, operator change rooms and ablutions, chemical make-up and storage area, general storage areas;
    - Site services will include security fencing with access control, flood lighting, access road to the plant, sanitation, safety equipment and adequate drainage.

13) The proposed plant layout was constrained by the delivery of final treated water to the Umlaas Road distribution pipeline system *via* gravity discharge, as well as by the necessity to keep the total WTW footprint to a minimum. The proposed plant layout combines features of accessible and compact unit process configuration, minimum length of interconnecting pipework, minimum amount of required excavations and ease of future extension.

14) The estimated plant capital and unit production costs of the proposed uMkhomazi WTW are summarised in **Table 28** below.

    Phase 1 consists of an initial plant capacity of 500 ML/d while Phase 2 consist of a plant extension to a total capacity of 1 250 ML/d. These summary costs include GAC filtration and GAC recharge.

Unit production cost estimates include capital redemption, chemical costs, power consumption, personnel and maintenance cost estimates.

Table 28: Summary of proposed uMkhomazi WTW costs (all excl. V.A.T.)

	Cost for Phase 1 [R]	Cost for Phase 2 [R]
Plant Capital Costs [R]	R 1 193 242 656.17	R 2 289 156 419.35
Unit Capital Redemption [c/m^3]	38.81	30.06
Unit Operating Costs [c/m^3]	66.74	61.14
Total Unit Production Costs [c/m^3]	105.55	91.20

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12 Appendix A: Classification & Design of a Landfill Site for WTW Sludge Disposal

Appendix A.1: Maximum Rate of Deposition

$$MRD = (IRD)(1+d)^t$$

Where:

IRD =	initial rate of deposition of refuse on site in T/day
d =	expected annual development rate, based on expected population growth rate in the area served by the landfill (Annual growth rate)
t =	years since deposition started at IRD (Design life)
MRD =	maximum rate of deposition after t years

IRD	=	920	T/d	@ 184 T/d for every 125 Mℓ/d with capacity of 625 Mℓ/d by 2053
d	=	0	%	
t	=	30	years	

$$MRD = (920)(1+0)^{30}$$

= 920 T/d > 500 T/d Therefore a **Large** Landfill Site will be used

Landfill Size Classes

Landfill Size Class		Maximum Rate of Deposition (MRD) (Tonnes per day)	
Communal	C		<25
Small	S	>25	<150
Medium	M	>150	<500
Large	L	>500	

Appendix A.2: Climatic Water Balance

B = R - E

Where:

B = the Climatic Water Balance in mm of water

R = the rainfall in mm of water

E = is the evaporation from a soil surface, taken as 0,70 x A-pan evaporation in mm or 0,88 x Span evaporation in mm

Rainfall (mm)														
YEAR	MONTHS												AVERAGE	Avgs for wettest 6 months
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec		
1950		68.6	144	54.6	16.5	0.8	28.5	49	1.8	40.4	54.4	86.9	49.6	65.7
1951	174.5	59.4	87.4	33	2.8	6.6	0	95.3	29.5	64.5	49.3	80.5	56.9	85.9
1952	146.6	103.1	94	49	25.7	4.3	3.3	28.7	18.5	45.2	59.4	130.3	59.0	96.4
1953	135.6	144.3	47.5	23.6	10.2	15	0	86.1	27.7	70.6	36.3	116.6	59.5	91.8
1954	138.2	61	45.5	26.4	60	4.1	4.6	3.3	56.4	111	79.3	33	51.9	78.0
1955	166.1	141.2	145	41.2	9.1	18.3	0	11.4	35.6	71.9	73.9	104.2	68.2	117.1
1956	12.5	193.1	131.1	31.5	22.1	4.1	2	14.5	21.6	51.3	136.9	214.1	69.6	123.2
1957	**146.1**	**68.3**	**69.3**	119.6	8.9	2.3	10.9	25.7	143	**132.4**	**170.7**	**90.9**	82.3	**113.0**
1958	61.5	142	69.3	127.5	1.5	2.8	1	6.1	45	27.2	112.8	131.1	60.7	90.7
1959	132.3	91.5	20.8	27.2	207.8	0	8.9	44.2	12.2	70.6	63.5	84.3	63.6	77.2
1960	32.3	79.5	168.9	51.8	2	4.8	3.1	5.3	33.3	47.5	106.7	274.3	67.5	118.2
1961	151.6	97	127.8	129.3	10.2	3.8	3.3	22.9	63	27.4	87.1	135.1	71.5	104.3
1962	101.6	128.8	95.8	43.9	0	0	0	34.5	1.3	42.9	105.7	56.7	50.9	88.6
1963	115.1	90.9	162.3	58.2	0.3	12.5	87.6	5.3	7.9	41.7	59.4	21.8	55.3	81.9
1964	170.2	67.6	109.5	40.6	3.3	36.1	14.7	0.3	65.3	97.6	76.7	97.5	65.0	103.2
1965	56.1	83.1	8.6	11.2	27.7	65.5	11.7	45.7	37.9	51.6	102.9	144.8	53.9	74.5
1966	160.5	69.6	10.7	58.2	23.6	2.3	0	28.5	17.8	60.7			43.2	50.3
													60.5	

S Class Pan Evaporation (mm)														
YEAR	MONTHS												AVERAGE	Avgs for wettest 6 months
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec		
1950		143.3	136.2	103.1	94.5	84.6	98.8	94	146.8	192.6	162.6	151.7	128.0	131.1
1951	140.5	152.4	128.3	119.1	107.7	107.7	113.3	111.8	123.7	113	191.3	126.8	128.0	142.1
1952	132.1	122.7	121.7	107.7	104.6	80.8	94.7	111.8	135.6	186.7	128.5	126.5	121.1	136.4
1953	174.3	115.8	108	70.6	109.7	91.2	103.1	101.3	120.6	125.2	107.4	126.5	112.8	126.2
1954	100.6	92.2	92	31.2	97.3	102.4	66.6	124.5	110.3	104.6	89.9	124.5	94.7	100.6
1955	126	103.9	153.2	82.3	77	76.5	100.9	125.8	152.7	117.3	108.7	112.3	111.4	120.2
1956	203.5	126.5	113	101.9	88.1	79.3	94.5	119.1	133.6	102.6	101.3	109.5	114.4	126.1
1957	**133.1**	**127.5**	**120.2**	88.4	92.7	90.9	67	113	89.2	**103.6**	**113.5**	**130.6**	105.8	**121.4**
1958	104.9	97.8	133.9	81.8	104.1	92.7	106.9	84.3	91.7	129.8	107.4	128.6	105.3	117.1
1959	139.5	113.3	126	125.2	55.9	88.9	92	120.1	128.8	143	117.9	135.1	115.5	129.1
1960	152.2	120.4	135.9	84.6	96.5	85.1	102.9	99.6	127	130.1	138.4	131.6	117.0	134.8
1961	162.1	137.4	122.2	88.9	88.6	70.4	81.8	117.4	121.4	129.8	112.8	157.5	115.9	137.0
1962	134.9	117.1	123.4	111	96.3	95.5	113.8	125.7	135.9	135.1	119.9	157.7	122.2	131.4
1963	137.1	154.9	110	101.8	104.4	69.1	73.9	118.4	135.6	121.7	149.4	175.8	121.0	141.5
1964	155.7	140.9	137.1	97.1	103.7	88.4	93	107.2	103.4	76	129.6	144.3	114.7	130.6
1965	133.6	149.3	150.6	106.7	110.5	77.2	75.9	102.9	101.9	114.8	93.2	150.9	114.0	132.1
1966	143.5	113.8	157.2	94.2	83.6	75.9	93.2	107.5	112	106.2			108.7	86.8
													114.7	

1955 B = 117.1 – (0.88 x 120.2) = + 11.324

1956 B = 123.2 – (0.88 x 126.1) = + 12.232

1957 B = 113.0 – (0.88 x 121.4) = + 6.168

1958 B = 90.7 – (0.88 x 117.1) = - 12.348

1959 B = 77.2 – (0.88 x 129.1) = - 36.408

Appendix A.3: Co-Disposal

The operational value of CR is selected, so that on average no more than 200mm of leachate per year will be produced.

$$CR = \frac{\bullet \times H}{L \times (1 + w) + \bullet \times H \times (f\text{-}w) - P_n \times (1 + w)\, \bullet_w}$$

Where

$\bullet$	=	The wet density of the "dry" waste (kg/m^3)
H	=	The height of lift of the landfill above the landfill base
L	=	average of 200mm/y (0,20m/y) of leachate
w	=	water content of the incoming 'dry' waste on a dry mass basis
f	=	field capacity of waste on a dry mass basis
P_n	=	nett precipitation per year per m^2
R	=	rainfall (m/y)
eS	=	evaporation (S Pan x 0.88) (m/y)
$\bullet_w$	=	density of water (kg/m^3)

$\bullet$	=	1.1	T/m^3	
H	=	5	Assumed…	Dependant on geotechnical investigation
L	=	0.2	m/y	
w	=	50	%	
f	=	0.5	Assumed…	Dependant on final sludge composition
P_n	=	-2.02	m/y …	(R - eS) x (w)
R	=	0.060	m/y	
eS	=	0.101	m/y	
$\bullet_w$	=	1	T/m^3	

$$CR = \frac{\bullet \times H}{L \times (1 + w) + \bullet \times H \times (f\text{-}w) - P_n \times (1 + w)\, \bullet_w}$$

$$= \frac{(1.1 \times 5)}{[0.2 \times (1 + 0.5)] + [1.1 \times 5 \times (0.5 - 0.5)] - \{[-2.02 \times (1 + 0.5)] \times (1)\}}$$

$$= 0.860556$$

$$y = \frac{1}{CR}$$

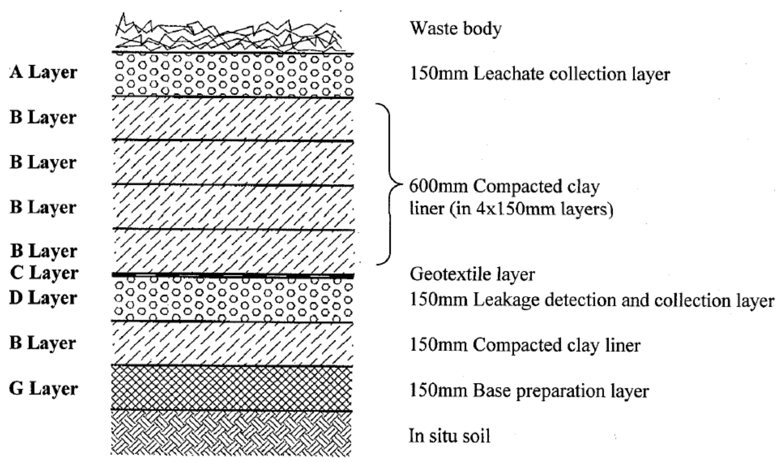
$$= \frac{1}{0.86}$$

$= 1.16$ The co-disposed liquids, as a proportion of the dry solids

Appendix A.4: Design of Lining

The number and sequence of liner components will vary with the class of landfill and leachate generated. ***Figure A.8.5*** below shows the typical layers for the liner for a G:L:B^+ class of site.

FIGURE A.8.5
G:M:B^+ and G:L:B^+ Landfills



A layer: A leachate collection layer comprising a 150mm thick layer of single-sized gravel or crushed stone having a size of between 38mm and 50mm.

B layer: A 150mm thick compacted clay liner layer. This must be compacted to a minimum density of 95% Standard Proctor (0,945 ℓ cylindrical mould, 2,5 kg hammer dropped 300 mm. Compaction in 3 layers each compacted with 25 blows (compactive effort = 595 kNm/m3)) maximum dry density at a water content of Proctor optimum to optimum +2%. Permeabilities must be such that the outflow rate of 0,3 m/y (1 x 10-6 cm/s) is not exceeded. Interfaces between B layers must be lightly scarified to assist in bonding the layers together. The surface of every clay liner layer must be graded towards the leachate collection drain or sumps at a minimum gradient of 2% for general waste disposal sites. At the discretion of the Department, B layers may be replaced by a geomembrane, a GCL, or a composite liner.

C layer: This is a layer of geotextile laid on top of any D layer to protect it from contamination by fine material from above.

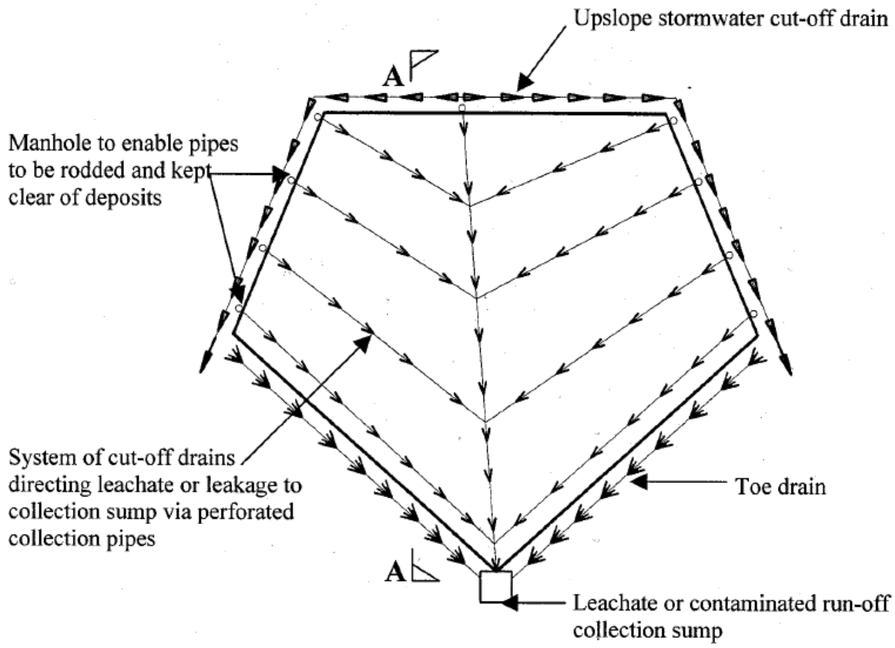
D layer: A leakage detection and collection layer. This is always below a C layer and above a B layer in B+ and hazardous waste landfills. In lagoons it is underlain by an E layer which protects the second FML or geomembrane. It has a minimum thickness of 150mm and will consist of single-sized gravel or crushed stone having a size of between 38mm and 50mm.

G layer: This is a base preparation layer consisting of a compacted layer of reworked in-situ soil with a minimum thickness of 150mm and constructed to the same compaction standards as a B layer. Where the permeability of a G layer can be proven to be of the same standard as a B layer it may replace the lowest B layer. The surface of every G layer must be graded towards a leachate collection drain or sump in the case of B^+ landfill. The minimum gradient must be 2% for G sites.

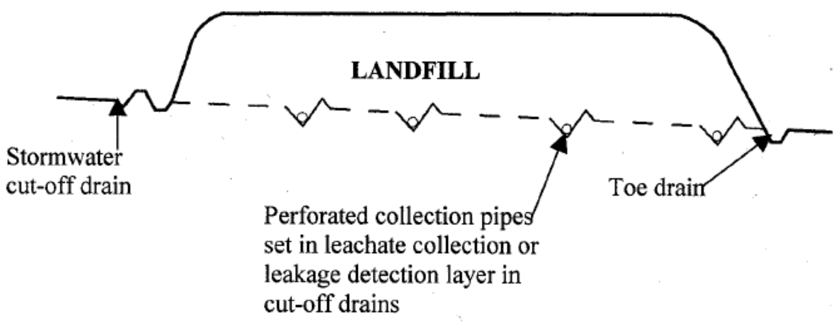
Appendix A.5: Design of Leachate Collection System

FIGURE A.8.9

Typical leachate collection system



Plan of landfill showing typical drainage systems



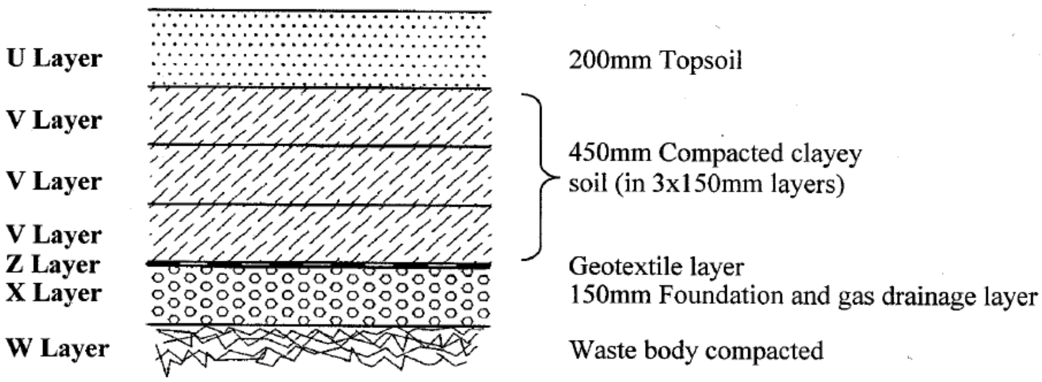
Section A-A through landfill

Appendix A.6: Design of Capping and Final Cover

The capping system is designed to maximise run-off of precipitation, while minimising infiltration and preventing ponding of water on the landfill. Cover requirements, and hence the number and sequence of components, will vary with the class of landfill under consideration. ***Figure A.8.12*** below shows the typical layers for the capping for a G:M:B$^+$ or G:L:B$^+$ class of site.

FIGURE A.8.12

G:M:B$^+$, G:L:B$^+$ and Hazardous Landfills



U layer: A 200mm thick layer of topsoil planted with local grasses and shrubs. The layer must be lightly compacted after spreading. In arid regions, this can be substituted with a layer of natural gravel.

V layer: A compacted 150mm soil cap layer. Any soil used in a V layer must have a Plasticity Index of between 5 and 15 and a maximum particle size of 25mm. This will be compacted to the maximum density reasonably attainable under the circumstances to ensure the required impermeability. This must not be less than 85% of Proctor maximum dry density at a water content of Proctor optimum to Proctor optimum +2%. The saturated steady state infiltration rate into a compacted soil V layer should not exceed 0,5m/y, as measured by means of an in situ double ring infiltrometer test. The surface of every V layer must be graded initially at a minimum of 3% to shed precipitation. At the discretion of the Department, V layers may be replaced by a geomembrane, a GCL, or a composite liner.

W layer: Shaped and compacted upper surface of waste body. (If available, it may prove useful to cover the waste surface with builders' rubble before compacting).

X layer: A gas venting layer having a minimum thickness of 150mm and consisting of single sized stone or gravel of between 25mm and 50mm in size. The X layer must be connected to a gas management system.

Z layer: This is a layer of geotextile laid on top of any X layer to protect the X layer from contamination.

Appendix A.7: Minimum Requirements for Landfill Design

LEGEND

- B^- = No significant leachate produced
- B^+ = Significant leachate produced
- R = Requirement
- N = Not a requirement
- F = Flag: special consideration to be given by expert or Departmental representative

CLASSIFICATION SYSTEM										
	G General Waste								H Hazardous Waste	
	C Communal Landfill		S Small Landfill		M Medium Landfill		L Large Landfill		h Hazard Rating 3 & 4	H Hazard Rating 1-4
MINIMUM REQUIREMENTS	B^-	B^+	B^-	B^+	B^-	B^+	B^-	B^+		
Appoint a Responsible Person	R	R	R	R	R	R	R	R	R	R
Conceptual Design Confirm site classification	R	R	R	R	R	R	R	R	R	R
Assess cover volume	N	N	R	R	R	R	R	R	R	R
Indicate unsaturated zone after cover excavation	N	N	R	R	R	R	R	R	R	R
Determine available airspace	N	N	R	R	R	R	R	R	R	R
Estimate airspace utilisation	N	N	R	R	R	R	R	R	R	R
Estimate site life	N	N	R	R	R	R	R	R	R	R
Address any impacts identified by investigation and/or by the IAPs	R	R	R	R	R	R	R	R	R	R
Site layout design	N	N	R	R	R	R	R	R	R	R
Surface drainage design	R	R	R	R	R	R	R	R	R	R
Development Plan	R	R	R	R	R	R	R	R	R	R
Closure/Rehabilitation Plan	R	R	R	R	R	R	R	R	R	R
Design of leachate management system	N	N	N	R	N	R	N	R	R	R
Design of the toe drains	N	R	N	R	R	R	R	R	R	R
Monitoring system design	N	N	F	R	R	R	R	R	R	R
End-use Plan	N	N	R	R	R	R	R	R	R	R
Testing of soils and materials	N	N	N	F	F	F	F	F	F	F

LEGEND B^- = No significant leachate produced B^+ = Significant leachate produced R = Requirement N = Not a requirement F = Flag: special consideration to be given by expert or Departmental representative	CLASSIFICATION SYSTEM									
	G General Waste								H Hazardous Waste	
	C Communal Landfill		S Small Landfill		M Medium Landfill		L Large Landfill		h Hazard Rating 3 & 4	H Hazard Rating 1-4
MINIMUM REQUIREMENTS	B^-	B^+	B^-	B^+	B^-	B^+	B^-	B^+		
Technical Design Surface hydrology and drainage design	N	N	N	F	R	R	R	R	R	R
Consult lining requirements in *Table 8.1* /Appendix 8.2	R	R	R	R	R	R	R	R	R	R
Water quality monitoring system	N	F	N	R	R	R	R	R	R	R
Leachate detection system	N	F	F	N	R	N	R	N	N	N
Leachate treatment system	N	N	N	F	N	R	N	R	R	R
Leachate management and monitoring system	N	F	N	R	N	R	N	R	R	R
Gas management and monitoring system	N	N	N	N	F	F	F	F	F	F
Consult cover requirements in *Table 8.2*/Appendix 8.2	R	R	R	R	R	R	R	R	R	R
Stability of slopes	N	N	F	F	F	F	F	R	R	R
Erosion control design	N	N	F	F	R	R	R	R	R	R
Design drawings and specifications	N	N	N	N	R	R	R	R	R	R
Approval of Technical Design	N	N	N	R	R	R	R	R	R	R

Appendix A.8: Minimum Requirements for Liner Components

LEGEND

B⁻ = No significant leachate produced
B⁺ = Significant leachate produced
R = Requirement
N = Not a requirement

LINER COMPONENTS	CLASSIFICATION SYSTEM										
	G General Waste								H Hazardous Waste		
	C Communal Landfill		S Small Landfill		M Medium Landfill		L Large Landfill		H:h Hazard Rating 3 & 4	H:H Hazard Rating 1-4	Lagoons
	B⁻	B⁺	B⁻	B⁺	B⁻	B⁺	B⁻	B⁺			
12 Waste body	R	R	R	R	R	R	R	R	R	R	R
11 Dessication protection	N	N	N	N	R	N	R	N	N	N	N
10 Leachate collection layer	N	N	N	R	N	R	N	R	R	R	N
9 Cushion layer	N	N	N	N	N	N	N	N	R	R	R
8 1,5mm or 2mm geomembrane	N	N	N	N	N	N	N	N	R	R	R
7 Compacted clay liner	N	N	N	N	N	R	N	R	R	R	R
6 Geotextile layer	N	N	N	N	N	R	N	R	R	R	R
5 Leakage detection layer	N	N	N	N	N	R	N	R	R	R	R
4 Cushion layer	N	N	N	N	N	N	N	N	N	N	R
3 1mm geo-membrane liner	N	N	N	N	N	N	N	N	N	N	R
2 Compacted clay liner	N	N	N	R	R	R	R	R	R	R	R
1 Base preparation layer	N	N	R	R	R	R	R	R	R	R	R

Note: Numbers 1 - 12 indicate order of construction.

Appendix A.9: Minimum Requirement for Capping Components

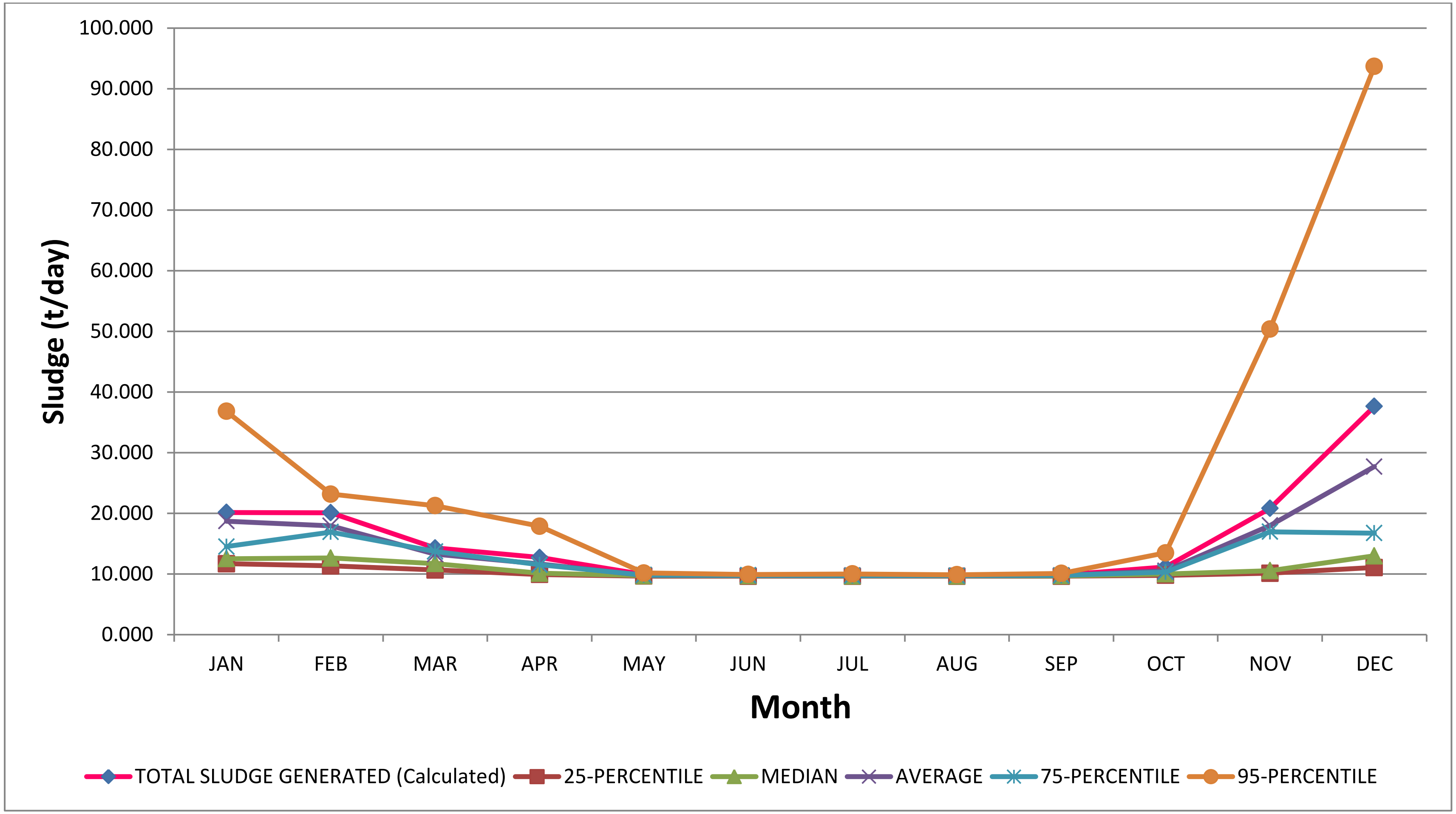
LEGEND	CLASSIFICATION SYSTEM									
B^- = No significant leachate produced B^+ = Significant leachate produced R = Requirement N = Not a requirement	G General Waste								H Hazardous Waste	
	C Communal Landfill		S Small Landfill		M Medium Landfill		L Large Landfill		H:h Hazard Rating 3 & 4	H:H Hazard Rating 1-4
CAPPING COMPONENTS	B^-	B^+	B^-	B^+	B^-	B^+	B^-	B^+		
5 Layer of Topsoil	R	R	R	R	R	R	R	R	R	R
4 Compacted Clay Layer	N	N	R	R	R	R	R	R	R	R
3 Geotextile Layer	N	N	N	N	N	R	N	R	R	R
2 Gas Drainage Layer	N	N	N	N	N	R	N	R	R	R
1 Shaped and Compacted Waste Surface	R	R	R	R	R	R	R	R	R	R

Note: Numbers 1 - 5 indicate order of construction.

Appendix A.10: Estimation of Sludge Generation at the new uMkhomazi WTW based on median values of SS at Lundy's

		JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
Calculation procedure based on Dr. Günter Lempert's presentation dated 1 April 2015													
Using calculated SS													
Sludge due to Turbidity (t/day)													
Average Raw Water Turbity	NTU	117.644	110.626	55.299	59.404	9.874	6.689	12.483	8.142	9.594	52.734	242.473	396.621
SS from Turbidity (t/day)	(1.35*NTU) & 600Ml/day production	95.292	89.607	44.792	48.117	7.998	5.418	10.111	6.595	7.771	42.715	196.403	321.263
SS in suspension	(refer to "Trap Efficiency Worksheet")	11%	12%	10%	6%	4%	2%	2%	2%	3%	4%	6%	9%
Sludge due to Turbidity (t/day)		**10.529**	**10.496**	**4.683**	**3.115**	**0.292**	**0.134**	**0.210**	**0.127**	**0.197**	**1.555**	**11.210**	**28.026**
Sludge due to Coagg &Flocc													
Alum		9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000	9.000
Poly		0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Sludge due to Coagg &Flocc (t/day)		**9.600**	**9.600**	**9.600**	**9.600**	**9.600**	**9.600**	**9.600**	**9.600**	**9.600**	**9.600**	**9.600**	**9.600**
TOTAL SLUDGE GENERATED (Calculated)		**20.129**	**20.096**	**14.283**	**12.715**	**9.892**	**9.734**	**9.810**	**9.727**	**9.797**	**11.155**	**20.810**	**37.626**
Calculations based on actual SS at Lundys													
SS (mg/L)													
25-PERCENTILE		31.400	24.500	16.900	7.850	2.000	2.000	2.000	2.000	2.000	8.000	15.200	28.000
MEDIAN		44.050	42.900	33.600	13.450	5.050	5.200	5.200	6.400	6.000	18.400	27.300	64.700
AVERAGE		137.162	118.772	58.169	53.084	8.093	9.247	11.469	9.447	10.485	41.728	244.522	345.386
75-PERCENTILE		74.100	104.100	65.400	49.200	9.600	11.275	9.070	10.000	10.400	32.925	214.400	136.000
95-PERCENTILE		410.350	192.950	185.700	212.700	24.980	20.780	32.080	22.300	31.800	177.700	1190.200	1606.120
Sludge after WW													
25-PERCENTILE		2.08165	1.721929	1.060145	0.304933	0.043809	0.029614	0.024882	0.023043	0.030479	0.174772	0.520547	1.465579
MEDIAN		2.920277	3.015133	2.107745	0.522466	0.110619	0.076997	0.064693	0.073737	0.091438	0.401977	0.93493	3.386534
AVERAGE		9.093083	8.347648	3.648973	2.062064	0.177282	0.136919	0.142683	0.108847	0.159785	0.911602	8.374017	18.07824
75-PERCENTILE		4.912429	7.316443	4.102574	1.911175	0.210285	0.16695	0.11284	0.115214	0.158492	0.719298	7.342456	7.118525
95-PERCENTILE		27.20398	13.56107	11.64905	8.262336	0.54718	0.307691	0.399108	0.256928	0.484621	3.882133	40.76022	84.06769
Sludge due to Coagg &Flocc		9.600	9.600	9.600	9.600	9.600	9.600	9.600	9.600	9.600	9.600	9.600	9.600
TOTAL SLUDGE GENERATED (t/day)													
25-PERCENTILE		**11.682**	**11.322**	**10.660**	**9.905**	**9.644**	**9.630**	**9.625**	**9.623**	**9.630**	**9.775**	**10.121**	**11.066**
MEDIAN		**12.520**	**12.615**	**11.708**	**10.122**	**9.711**	**9.677**	**9.665**	**9.674**	**9.691**	**10.002**	**10.535**	**12.987**
AVERAGE		**18.693**	**17.948**	**13.249**	**11.662**	**9.777**	**9.737**	**9.743**	**9.709**	**9.760**	**10.512**	**17.974**	**27.678**
75-PERCENTILE		**14.512**	**16.916**	**13.703**	**11.511**	**9.810**	**9.767**	**9.713**	**9.715**	**9.758**	**10.319**	**16.942**	**16.719**
95-PERCENTILE		**36.804**	**23.161**	**21.249**	**17.862**	**10.147**	**9.908**	**9.999**	**9.857**	**10.085**	**13.482**	**50.360**	**93.668**
	Correlation Check with Durban Heights												
	Durban Heights sludge disposal		7 m3	average per day									
	Assuming Density of		1800	kg/m3									
	Tonnage		13	t/day									

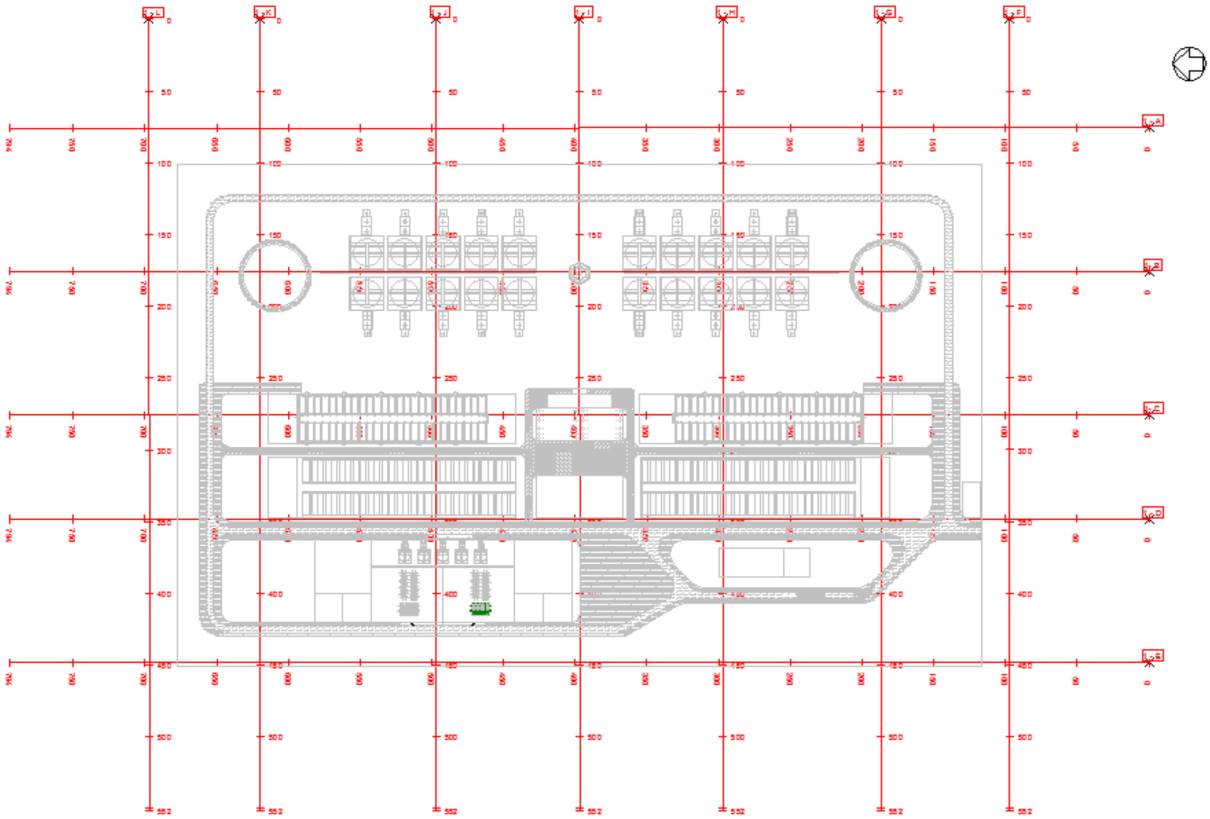
Sludge Generation Chart (Subramanian, 2015)



13 Appendix B: Earthworks Cost Report

This report presents the earthworks costs for the first phase of the water treatment works. Bulk earthworks to create the terraces for the second phase of the site were taken into consideration.

The earthwork volumes were calculated using the average end area method. The cut and fill end areas were averaged over the entire length between them. These areas were measured along the cross sections of the proposed site at specific grid intervals as shown on the image below.



Bulk earthworks calculations for sites B2, B3 and M1 are attached.

UMKHOMAZI TREATMENT WORKS SITE OPTION B2 - BULK EARTHWORKS
AVERAGE END AREA CALCULATION

GRID	DISTANCE (m)	FILL AREA (m^2)	CUT AREA (m^2)	FILL VOLUME (m^3)	CUT VOLUME (m^3)
2A		1 987.00	2 145.00		
	100.00			541 100.00	244 400.00
2B		3 424.00	299.00		
	100.00			357 500.00	422 800.00
2C		151.00	3 929.00		
	72.50			47 197.50	444 135.00
2D		500.00	2 197.00		
	100.00			458 400.00	219 700.00
2E		4 084.00	0.00		
TOTAL				**1 404 197.50**	**1 331 035.00**

GRID	DISTANCE (m)	FILL AREA (m^2)	CUT AREA (m^2)	FILL VOLUME (m^3)	CUT VOLUME (m^3)
2F		3 630.00	0.00		
	90.00			448 290.00	18 990.00
2G		1 351.00	211.00		
	110.00			198 220.00	133 980.00
2H		451.00	1 007.00		
	100.00			58 000.00	252 700.00
2I		129.00	1 520.00		
	100.00			12 900.00	344 500.00
2J		0.00	1 925.00		
	122.52			86 744.80	357 883.55
2K		708.00	996.00		
	77.48			201 833.06	77 169.18
2L		1 897.00			
TOTAL				**1 005 987.85**	**1 185 222.73**

STRUCTURE	VOLUME (m^3)	No. Off
CLARIFIER	5 910.00	12.00
INTAKE AND DISTRIBUTION	3 534.29	1.00
CUT VOLUME OF STRUCTURES	74 454.29	

TOTAL FILL	**2 410 185.35**	**m3**	48%
TOTAL CUT	**2 516 257.73**	**m3**	52%
SPOIL VOLUME	180 526.67		

UMKHOMAZI TREATMENT WORKS SITE OPTION B3 - BULK EARTHWORKS
AVERAGE END AREA CALCULATION

GRID	DISTANCE (m)	FILL AREA (m^2)	CUT AREA (m^2)	FILL VOLUME (m^3)	CUT VOLUME (m^3)
3A		104.00	4 783.00		
	100.00			72 300.00	710 400.00
3B		619.00	2 321.00		
	100.00			61 900.00	902 000.00
3C		0.00	6 699.00		
	72.50			120 930.00	587 902.50
3D		1 668.00	1 410.00		
	100.00			900 600.00	141 000.00
3E		7 338.00	0.00		
TOTAL				**1 155 730.00**	**2 341 302.50**

GRID	DISTANCE (m)	FILL AREA (m^2)	CUT AREA (m^2)	FILL VOLUME (m^3)	CUT VOLUME (m^3)
3F		1 917.00	0.00		
	90.00			221 850.00	117 810.00
3G		548.00	1 309.00		
	110.00			60 280.00	422 290.00
3H		0.00	2 530.00		
	100.00			0.00	521 600.00
3I		0.00	2 686.00		
	100.00			0.00	455 000.00
3J		0.00	1 864.00		
	122.52			305 934.69	280 327.82
3K		2 497.00	424.00		
	77.48			481 377.65	32 851.14
3L		3 716.00	0.00		
TOTAL				**1 069 442.34**	**1 829 878.96**

STRUCTURE	VOLUME (m^3)	No. Off
CLARIFIER	5 910.00	12.00
INTAKE AND DISTRIBUTION	3 534.29	1.00
CUT VOLUME OF STRUCTURES	74 454.29	

TOTAL FILL	**2 225 172.34**	**m3**	34%
TOTAL CUT	**4 171 181.46**	**m3**	66%
SPOIL VOLUME	2 020 463.41		

UMKHOMAZI TREATMENT WORKS SITE OPTION M1 - BULK EARTHWORKS
AVERAGE END AREA CALCULATION

GRID	DISTANCE (m)	FILL AREA (m^2)	CUT AREA (m^2)	FILL VOLUME (m^3)	CUT VOLUME (m^3)
4A		155.00	4 971.00		
	100.00			86 500.00	630 400.00
4B		710.00	1 333.00		
	100.00			89 400.00	390 400.00
4C		184.00	2 571.00		
	72.50			227 360.00	232 580.00
4D		2 952.00	637.00		
	100.00			1 179 300.00	63 700.00
4E		8 841.00	0.00		
TOTAL				**1 582 560.00**	**1 317 080.00**

GRID	DISTANCE (m)	Column1	Column2	FILL VOLUME (m^3)	CUT VOLUME (m^3)
4F		0.00	2 197.00		
	90.00			12 240.00	372 960.00
4G		136.00	1 947.00		
	110.00			83 270.00	353 870.00
4H		621.00	1 270.00		
	100.00			175 400.00	179 900.00
4I		1 133.00	529.00		
	100.00			249 700.00	85 300.00
4J		1 364.00	324.00		
	122.52			464 354.21	50 233.57
4K		2 426.00	86.00		
	77.48			398 629.97	6 663.20
4L		2 719.00	0.00		
TOTAL				**1 383 594.18**	**1 048 926.77**

STRUCTURE	VOLUME (m^3)	No. Off
CLARIFIER	5 910.00	12.00
INTAKE AND DISTRIBUTION	3 534.29	1.00
CUT VOLUME OF STRUCTURES	74 454.29	

TOTAL FILL	**2 966 154.18**	**m3**	55%
TOTAL CUT	**2 366 006.77**	**m3**	45%
IMPORT VOLUME	525 693.12		

14 Appendix C: Budget Power Supply Costs

From: Simphiwe Ngwenya <NgwenySC@eskom.co.za>
Sent: 13 April 2015 3:25 PM
To: Amal Doorgapershad; Gavin Subramanian
Cc: Oupa Makaleng
Subject: RE: 20140514 30300413 uMWP: WTW details for Eskom

Good day

As discussed earlier, below are the number of options we went through earlier today.

Option 1 (132kV bulk supply):

In order for Eskom to be able to avail a premium supply of 8MVA, a 132/11 kV substation is recommended:
- Construct approximately 2x200m of Kingbird 132kV line from Ariadne/Riverdale 132kV line to create a loop-in loop-out set up to the new proposed substation site.
- Install metering unit on the 132kV busbar.

Summary of Estimated Costs

	0.4km Kingbird Line
Material & Transformer	R 411 678
Labour & Transport	R 372 328
Engineering Fees	R 141 121
Dismantle	0
OH(7%)	R 69 082
IDC(10.14)	R 8 339
Material & LabourContingency(30%)	R 61 751.71
TOTAL PROJECT COST	R 1 064 299.82

Please note that the customer contribution was calculated as follows:
- 100% of 132kV Kingbird line
- The customer will have to construct and commission the entire substation according to Eskom standards

Option 2 (11kV bulk supply):

In order for Eskom to be able to avail a premium supply of 8MVA, a 132/11 kV substation is recommended:
- Establish a new 2x20MVA 132/11kV substation in the vicinity of 29 45 45.892 S & 30 21 52.700 E
- Construct approximately 2x200m of Kingbird 132kV line from Ariadne/Riverdale 132kV line to create a loop-in loop-out set up to the new proposed substation site.
- Install and equip 4x11kV feeder bays, two on either side of the bus section and install metering equipment on the 11kV busbar accordingly.

Summary of Estimated Costs

	2*20MVA 132/11 kV Sub	0.4km of Kingbird line
Material	R 19 730 255.66	R 411 678.04
Labour & Transport	R 4 731 384.69	R 372 328.27
Engineering Fees	R 5 724 023.84	R 141 121.14
Dismantle	R 0.00	R 0.00
OH(7%)	R 2 626 690.94	R 69 081.54
IDC(10.14)	R 6 658 058.74	R 8 339.13
Material & Labour Contingency(30%)	R 7 338 492.11	R 61 751.71
SUB-TOTAL	R 46 808 905.98	R 1 064 299.82
TOTAL PROJECT COST		R 47 873 205.80

Please note that the customer contribution was calculated as follows:
- 100% of 2x20MVA 132/11kV substation
- 100% of 132kV Kingbird line

Assumptions made in the Scope of Work
- DOORGAPERSHAD will have a dedicated, firm (n-1) HV/MV Substation.

- Eskom will provide the necessary metering infrastructure.
- Switchgear is available within required lead times.

Option 3 (11kV bulk supply from existing Thornville substation)

- Extend the substation yard and a control room to cater for the new equipment.
- Install a new 132/11kV 20MVA transformer at Thornville substation.
- Replace the existing 132/11kV 10MVA transformer with a 132/11kV 20MVA transformer.
- Install and equip 2x11kV feeder bays to supply the customer.
- Construct 2x 5km of 11kV Chicadee line and template at 80 degrees Celsius from Thornville substation to customer's point of supply.

Summary of Estimated Costs

	2*20MVA 132/11 kV Sub	2x5km of Chicadee line
Material	R 16 292 852.14	R 1 235 000.00
Labour & Transport	R 5 025 627.24	R 592 800.00
Engineering Fees	R 4 988 524.17	R 305 908.33
Dismantle	R 25 000.00	R 0.00
OH(7%)	R 2 289 178.32	R 161 768.64
IDC(10.14)	R 6 500 570.93	R 250 736.77
Material & Labour Contingency(30%)	R 6 395 543.81	R 518 700.00
SUB-TOTAL	R 41 517 296.6	R 3 064 913.74
TOTAL PROJECT COST		R 44 582 210.35

These costs for this option shall be shared with Eskom according to the load taken from the sub.
Disadvantages of this option:

MV lines require way-leave which may escalate costs.
Environmental studies will be required; this will delay the project further.
132kV line radial supply from Ariadne 400kV Substation.

Please note that the figures above are note 100% and are not final.
Amal, the approximate dimension for the 2*20MVA substation is 50mx50m at the worst case scenario.

Kind Regards

Simphiwe Ngwenya
Asset Creation
Network Planning Department
1st Floor Engineering Building,
1 Portland Road, Mkondeni
Eskom KZN OU
033 395 3772
071 946 9042
Fax-2_e-mail: 0862159236
E-mail: ngwenysc@eskom.co.za

From: Amal Doorgapershad [mailto:adoorgapershad@knightpiesold.com]
Sent: 02 April 2015 09:53 AM
To: Gavin Subramanian; Oupa Makaleng
Cc: Simphiwe Ngwenya
Subject: RE: 20140514 30300413 uMWP: WTW details for Eskom

Good morning.

Fine with me as well. I will confirm with a calendar invite.

Regards
Amal

Amal Doorgapershad, Pr Eng
Regional Manager : KwaZulu Natal
Knight Piésold (Pty) Ltd.

From: Gavin Subramanian [mailto:gavin.subramanian@umgeni.co.za]
Sent: 02 April 2015 08:20 AM
To: Oupa Makaleng; Amal Doorgapershad
Cc: Simphiwe Ngwenya
Subject: RE: 20140514 30300413 uMWP: WTW details for Eskom

Ok with me

Regards
Gavin Subramanian PrTech Eng

Planning Engineer

Umgeni Water | Head Office
Tel: 033-341 1271 | Fax: 033 341 1218 | Cell: 071 671 7764
Physical Address: 310 Burger Street, Pietermaritzburg, 3201, South Africa
E-Mail: gavin.subramanian@umgeni.co.za | **Website:**www.umgeni.co.za

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From: Oupa Makaleng [mailto:MakaleSO@eskom.co.za]
Sent: 02 April 2015 07:56 AM
To: Amal Doorgapershad
Cc: Gavin Subramanian; Simphiwe Ngwenya
Subject: RE: 20140514 30300413 uMWP: WTW details for Eskom

Morning,

Can we make it on the 13th at 11 am

From: Amal Doorgapershad [mailto:adoorgapershad@knightpiesold.com]
Sent: 01 April 2015 09:09 PM
To: Oupa Makaleng
Cc: Gavin Subramanian (gavin.subramanian@umgeni.co.za)
Subject: RE: 20140514 30300413 uMWP: WTW details for Eskom

Hi Oupa

We are available on 13th, 14th and 16th April 2015 from 10am. Please confirm a meeting time.

Regards
Amal

Amal Doorgapershad, Pr Eng
Regional Manager : KwaZulu Natal
Knight Piésold (Pty) Ltd.

From: Oupa Makaleng [mailto:MakaleSO@eskom.co.za]
Sent: 31 March 2015 08:54 AM

To: Amal Doorgapershad
Subject: RE: 20140514 30300413 uMWP: WTW details for Eskom

Morning Amal,

I was talking to the guys at Network Planning and they told me that feedback that will be shared in the proposed meeting will be regarding the availability of capacity on each site, the cost and the advantages and disadvantages of the said sites. If there is someone technical locally then you will not need to bring the team down from Gauteng.

Regards,

Oupa Makaleng
Customer Advisor
Group Customer Services
1 Portland Road Mkhondeni Wattle Crane Ground Floor
Tel +27 33 395 7088
Cell +27 60 608 8883
Fax +27 33 395 3486
E-mail MakaleSO@eskom.co.za

"We the willing, led by the unknown, are doing the impossible for the ungrateful. We have done so much, with so little, for so long. We are now qualified to do anything with nothing." – Mother Theresa of Calcutta

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From: Amal Doorgapershad [mailto:adoorgapershad@knightpiesold.com]
Sent: 14 May 2014 08:49 AM
To: Oupa Makaleng
Subject: 20140514 30300413 uMWP: WTW details for Eskom

Hi

As discussed we require a quotation for 7 Megawatts of power for a new water treatment works at three possible sites. Only one site will be developed. The estimated commissioning year will be 2023. The names and coordinates of these sites are as follows:

a) WTW Site No. 04 (Open Field): Latitude: 29°45'44.34"S, Longitude: 30°21'58.42"E
b) WTW Site No. 02 (NCT): Latitude: 29°46'26.74"S, Longitude: 30°20'34.07"E
c) WTW Site No. 06 (Hopewell): Latitude: 29°45'55.63"S, Longitude: 30°24'30.28"E

Google Earth kmz files for each site have been attached together with a map showing the three sites.

Please feel free to contact me if you have any further queries.

Regards
Amal

Amal Doorgapershad, Pr Eng
Regional Manager : KwaZulu Natal
Knight Piésold (Pty) Ltd.

2nd Floor, Engen House, 171 Rodger Sishi Road (Blair Atholl Drive), Westville North
Durban | KwaZulu-Natal | South Africa | 3629
phone: +27 31 276 4660 | **fax:** +27 31 262 2950
direct: +27 31 276 4667
email: adoorgapershad@knightpiesold.com
web:
ISO 9001:2008 Certificate No: 212061140/1
BS OHSAS 18001:2007 Certificate No: 271012056

I'm part of the 49Million initiative.
http://www.49Million.co.za

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I'm part of the 49Million initiative.
http://www.49Million.co.za

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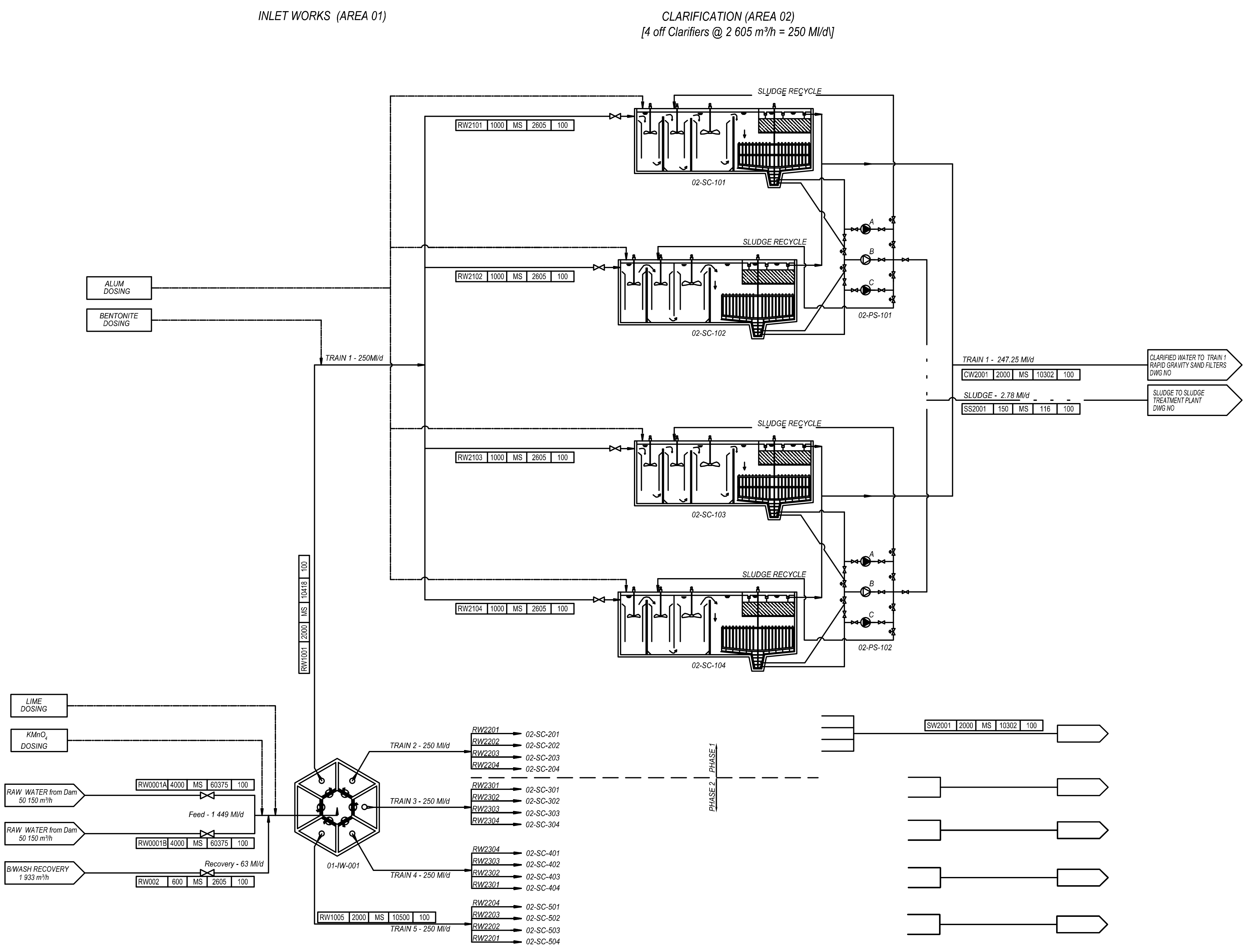
15 Appendix D: Drawings

- 1301.X01.UW-001 Sheet 1 of 5: P&ID – Inlet works & clarification
- 1301.X01.UW-001 Sheet 2 of 5: P&ID – Rapid gravity sand filters
- 1301.X01.UW-001 Sheet 3 of 5: P&ID – Granular activated carbon filters
- 1301.X01.UW-001 Sheet 4 of 5: P&ID – Chlorine contact tank and final water storage
- 1301.X01.UW-001 Sheet 5 of 5: P&ID – Sludge treatment plant
- 1301.X01.UW-002: Process flow diagram
- 1301.X01.UW-003: Hydraulic profile
- 1301.X01.UW-100: Site layout
- 1301.X01.UW-110: Site works
- 30300413/B07: WTW Earthworks Option B2 – Plan
- 30300413/G05: WTW Earthworks Option B2 - Cross Sections Sheet 1
- 30300413/G06: WTW Earthworks Option B2 - Cross Sections Sheet 2
- 30300413/B08: WTW Earthworks Option B3 – Plan
- 30300413/G07: WTW Earthworks Option B3 - Cross Sections Sheet 1
- 30300413/G08: WTW Earthworks Option B3 - Cross Sections Sheet 2
- 30300413/B09: WTW Earthworks Option M1 - Plan
- 30300413/G09: WTW Earthworks Option M1 - Cross Sections Sheet 1
- 30300413/G10: WTW Earthworks Option M1 - Cross Sections Sheet 2



01-IW-001

INLET DISTIBUTION
BOX - 1 OFF
CAPACITY - 52 100 m³/h

02-SC-101 TO 604

HIGH-RATE LAMELLA
CLARIFIERS - 20 OFF
CAPACITY - 2 605 m³/h

02-PS-101 TO 602

DESLUDGE/SLUDGE RECIRC.
PUMPS - 30 OFF
TYPE - PROGRESSIVE CAVITY
CAPACITY - 80 m³/h
HEAD - 6 mWC
MOTOR - 5.5 kW (Inst)

NOTES:

1. LINE NUMBER LEGEND

1001	25	PVC	2,5	27

LINE No
PIPE (O/DIA)
LINE MTL.
FLOW m³/hr – LIQUID
MASS kg/hr – GAS
OPP. PRESSURE kPa

Knight Piésold
CONSULTING

PRIMARY DISCIPLINE	DISCIPLINE	
DRAWN	A.S.	MAY2013
DRAWING CHECK	CHECKER	DATE
DESIGN	G.L.	MAY2013
DESIGN CHECK	CHECK	DATE
PROJECT ENGINEER	ENGIN	DATE

DRAWINGNO.

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
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UMGENI
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REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE
–	–	–

PROJECT: uMKHOMAZI WATER PROJECT
1 250ML/d WATER TREATMENT PLANT

TITLE: P & ID

SHEET: SHEET 1 OF 5

DRG. No. 1301.X01.UW - 001

REV.10

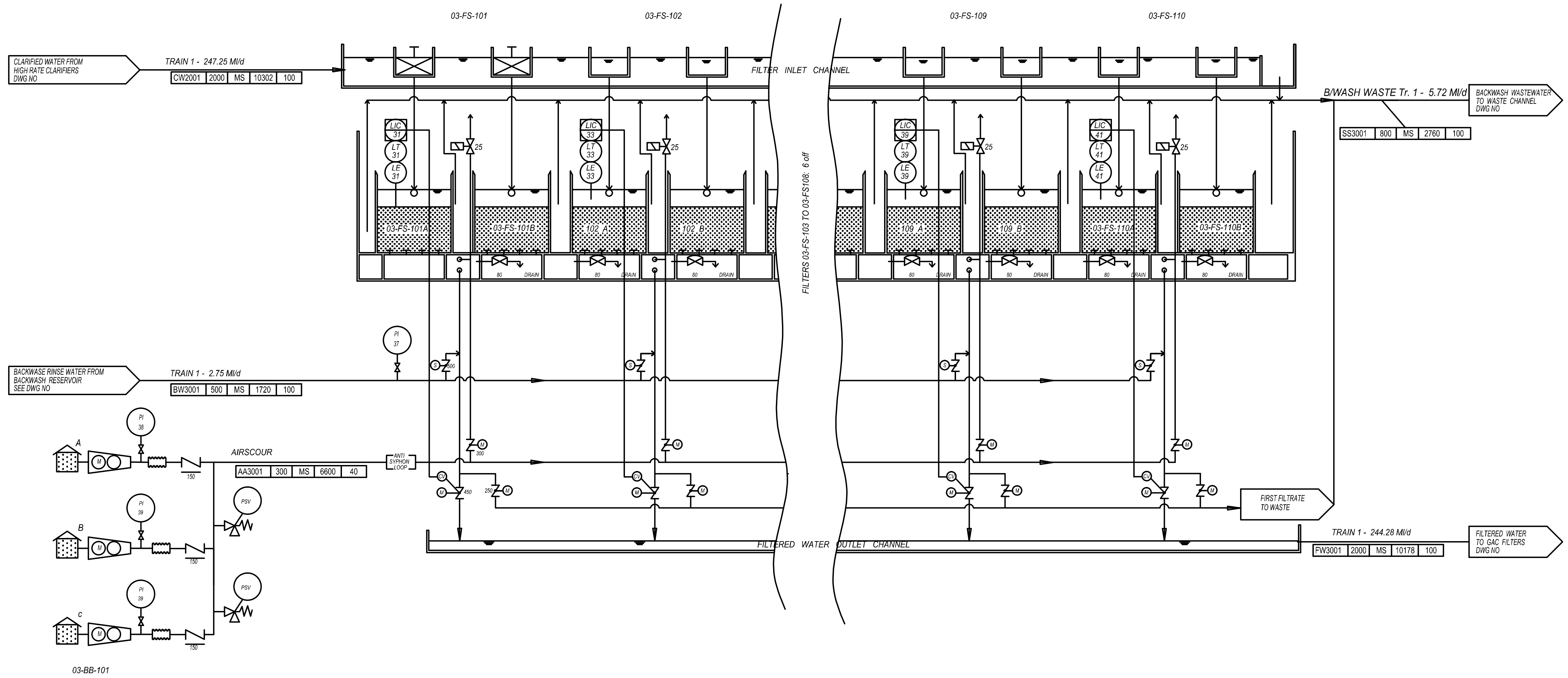
NOTES:

1. LINE NUMBER LEGEND

1001	25	PVC	2,5	27

LINE No
PIPE (O/DIA)
LINE MTL.
FLOW m³/hr – LIQUID
MASS kg/hr – GAS
OPP. PRESSURE kPa

RAPID GRAVITY SAND FILTERS (AREA 03)
[10 off (N+1) @ 1 030 m³/h = 247.25 Ml/d\]



03-BB-101A/B/C (2 duty/stdby)

BACKWASH BLOWERS - 3 OFF
TYPE - ROOTS
CAPACITY - 3300 m³/h FAD
dP - 40 kPa
MOTOR - 75 kW (inst)

03-FS-101 TO 03-FS-110

RAPID GRAVITY, DUAL MEDIA
SAND FILTERS - 10 OFF
CAPACITY - 1 030 m³/h normal
- 1 145 m³/h max
TYPE - DOUBLE BED
SIZE - 15 m x 4 m PER BED
Filter AREA - 120 m² PER FILTER
MEDIA:
Si Sand - 0,8 ES; 800mm DEEP
Anthracite - 1,3 ES; 700mm DEEP

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PRIMARY DISCIPLINE	DISCIPLINE	
DRAWN	A.S.	MAY2013
DRAWING CHECK	CHECKER	DATE
DESIGN	G.L.	MAY2013
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PROJECT ENGINEER	ENGIN	DATE

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UMGENI WATER • AMANZI

REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE
–	–	–

PROJECT: uMKHOMAZI WATER PROJECT
1 250ML/d WATER TREATMENT PLANT

TITLE: P & ID

SHEET: SHEET 2 OF 5

DRG. No. 1301.X01.UW - 001

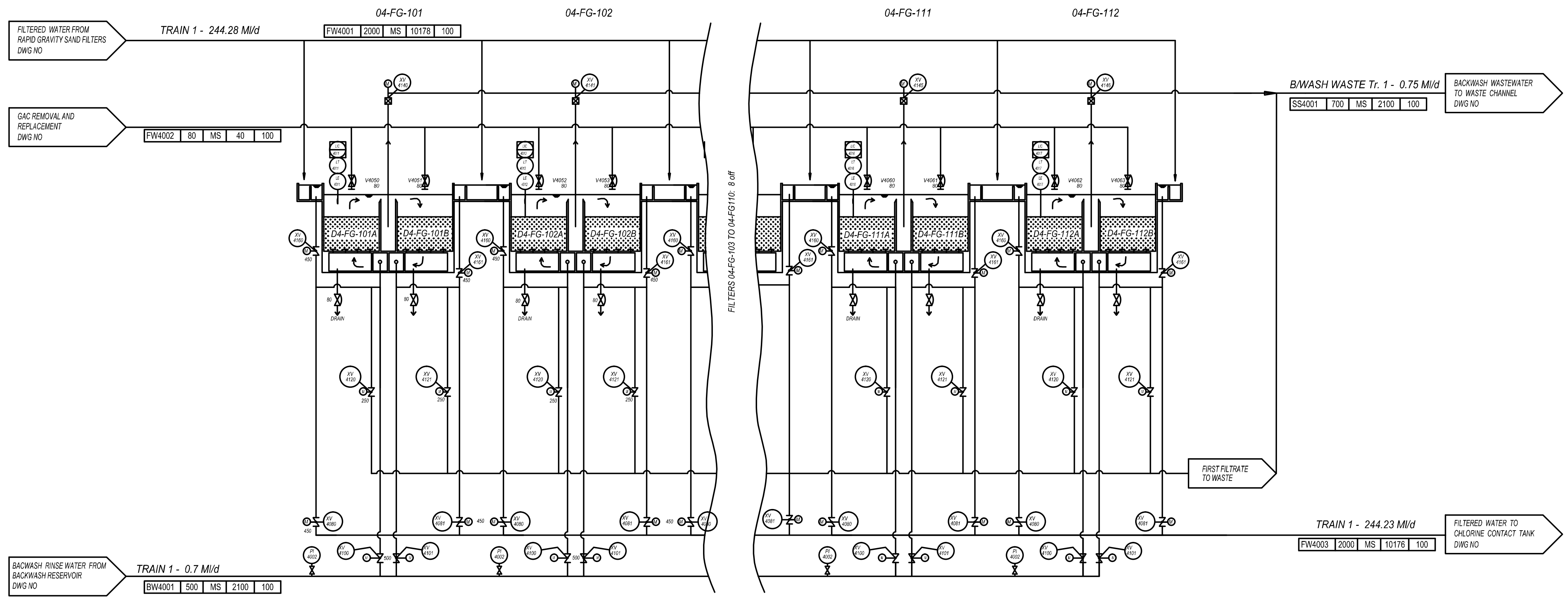
REV. 10

NOTES:

1. LINE NUMBER LEGEND

1001	25	PVC	2,5	27
LINE No	PIPE (O/DIA)	LINE MTL.	FLOW m³/hr – LIQUID MASS kg/hr – GAS	OPP. PRESSURE kPa

GRANULAR ACTIVATED CARBON FILTERS (AREA 04)
[12 off (N+2) @ 848 m³/h = 244.28 Ml/d]



04-FG-101 TO 04-FG-112

GRANULAR ACTIVATED CARBON
FILTERS - 12 OFF
CAPACITY - 848 m³/h normal
- 1 050 m³/h max
TYPE - 2-STAGE, DOUBLE BED
SIZE - 15 m x 4 m PER BED
Filter AREA - 120 m² PER FILTER
MEDIA:
GAC - 1.3 ES; 2 x 750 mm DEEP

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PRIMARY DISCIPLINE	DISCIPLINE	
DRAWN	A.S.	MAY2013
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DESIGN	G.L.	MAY2013
DESIGN CHECK	CHECK	DATE
PROJECT ENGINEER	ENGIN	DATE

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REVISION

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UMGENI WATER - AMANZI

REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE
–	–	–

PROJECT: uMKHOMAZI WATER PROJECT 1 250ML/d WATER TREATMENT PLANT

TITLE: P & ID

SHEET: SHEET 3 OF 5

DRG. No. 1301.X01.UW - 001 REV. 10

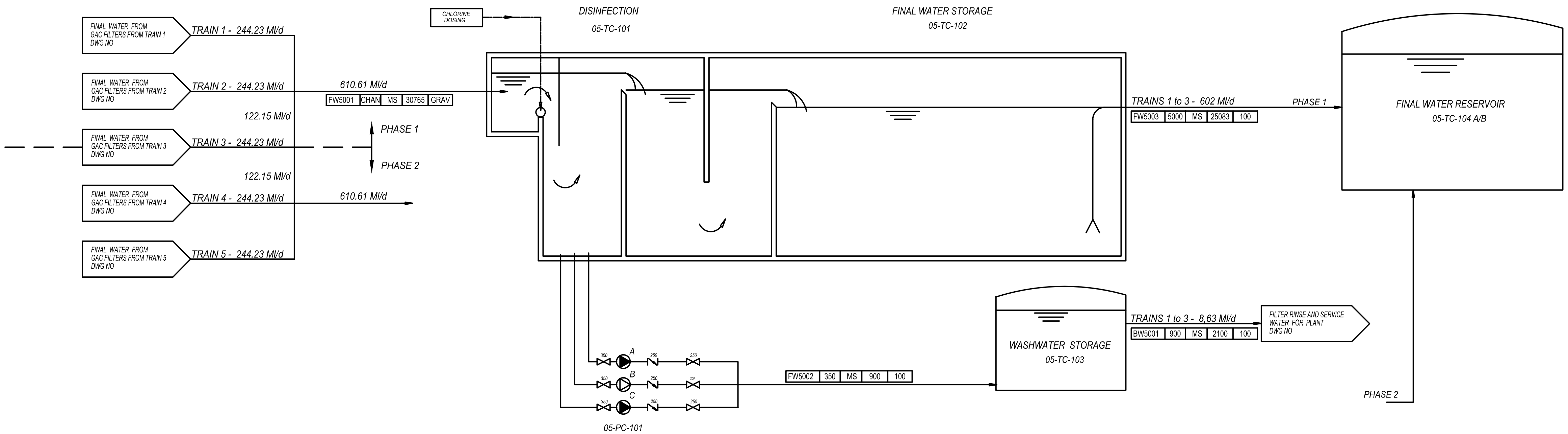
NOTES:

1. LINE NUMBER LEGEND

1001	25	PVC	2,5	27

- LINE No
- PIPE (O/DIA)
- LINE MTL.
- FLOW m^3/hr – LIQUID
- MASS kg/hr – GAS
- OPP. PRESSURE kPa

CHLORINE CONTACT TANK AND FINAL WATER STORAGE RESERVOIR (AREA 5)
(for 625 Ml/d)



05-PC-101 A/B/C (2 duty, 1 stdby)
WASHWATER RESERVOIR
FEED PUMPS - 3 OFF
TYPE - CENTRIFUGAL
CAPACITY - 300 m^3/h
HEAD - 20 mWC
MOTOR - 30 kW (Inst)

05-TC-101
CHLORINE CONTACT
TANK FOR 625 Ml/D - 1 OFF
EFFECTIVE VOLUME - 9 000 m^3
SIZE (lxbxht) - 100 m x 20 m x 4,5 m
CONTACT TIME - 20 min (minimum)

05-TC-102
FINAL INTERMEDIATE WATER STORAGE TANK
RESERVOIR FOR 625 Ml/D - 1 OFF
EFFECTIVE VOLUME - 80 000 m^3
SIZE (lxbxht) - 180 m x 60 m x 9,5 m ht
STORAGE TIME - 3 h

05-TC-103
WASHWATER STORAGE
RESERVOIR FOR 625 Ml/D - 1 OFF
EFFECTIVE VOLUME - 9 000 m^3
SIZE - 46 m dia, 5.5 m ht
STORAGE TIMES:
Average - 24 h
Minimum - 3.0 h

05-TC-104 A/B
FINAL WATER STORAGE
RESERVOIR FOR 625 Ml/D - 2 OFF
EFFECTIVE VOLUME - 160 000 m^3
SIZE - 160 x 100x 10 m
STORAGE TIMES: - 6h

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PRIMARY DISCIPLINE	DISCIPLINE	
DRAWN	A.S.	MAY2013
DRAWING CHECK	CHECKER	DATE
DESIGN	G.L.	MAY2013
DESIGN CHECK	CHECK	DATE
PROJECT ENGINEER	ENGIN	DATE

DRAWINGNO.

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
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UMGENI WATER · AMANZI

REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE
–	–	–

PROJECT: uMKHOMAZI WATER PROJECT
1 250ML/d WATER TREATMENT PLANT

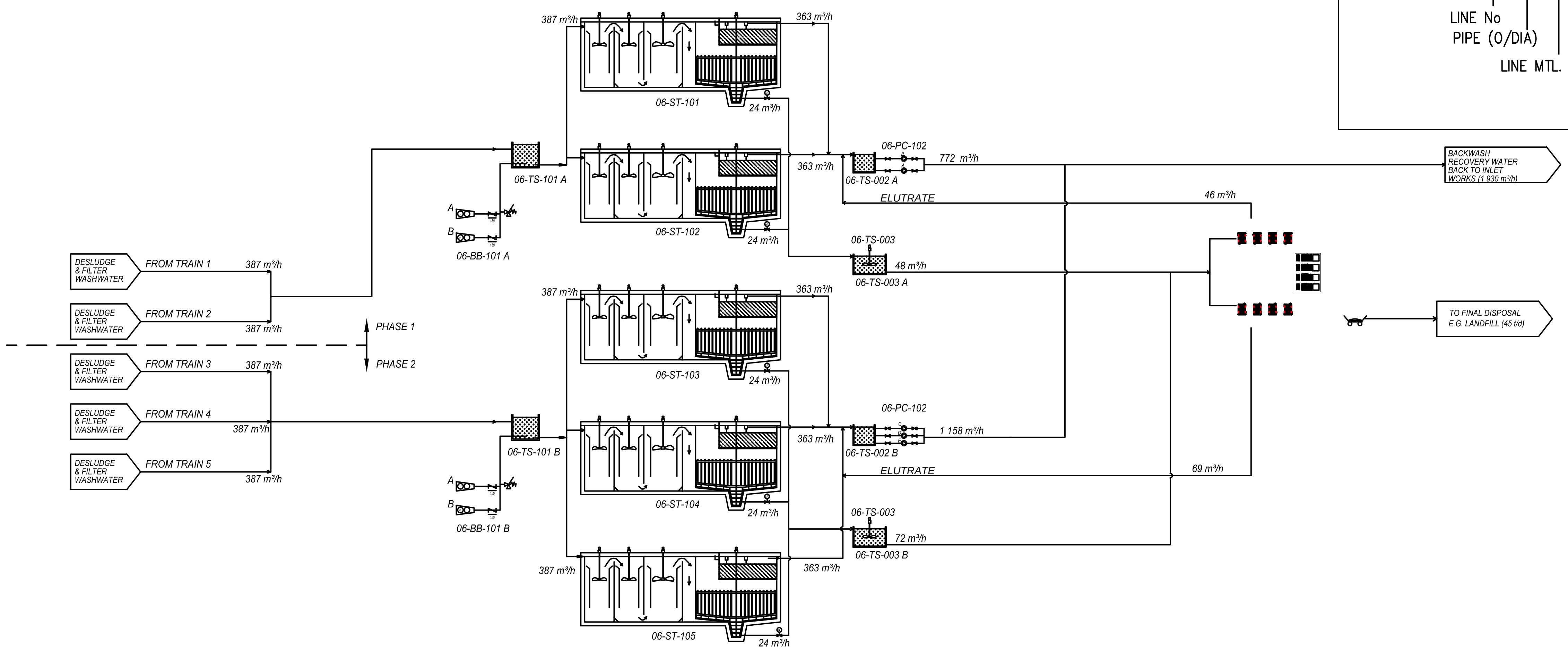
TITLE: P & ID

SHEET: SHEET 4 OF 5

DRG. No. 1301.X01.UW - 001 REV. 10

SLUDGE TREATMENT PLANT (AREA 06)

(for 1250 Ml/d)



NOTES:

1. LINE NUMBER LEGEND

1001	25	PVC	2,5	27

LINE No
PIPE (O/DIA)
LINE MTL.
FLOW m³/hr – LIQUID
MASS kg/hr – GAS
OPP. PRESSURE kPa

06-BB-101A/B
SLUDGE MIXING
BLOWERS : 2 OFF
TYPE: ROOTS
CAPACITY:
HEAD:
MOTOR:

06-TS-001 A/B
SLUDGE
HOLDING TANK - 2 OFF
EFFECTIVE VOLUME - 2 500 m³
SIZE (lxbxht) - 27 m x 27 m x 3.5 m

06-ST-101, 102, 103, 104 AND 105
HIGH-RATE LAMELLA
SLUDGE THICKENERS - 5 OFF
CAPACITY - 327 m³/h normal
- 387 m³/h max

06-TS-002 A/B
RECOVERED WASHWATER
HOLDING TANK - 2 OFF
EFFECTIVE VOLUME - 750 m³
SIZE (lxbxht) - 15 m x 15 m x 3.5 m

06-PC-102 A/B/C/D/E
RECOVERED WASHWATER
RETURN PUMPS - 3 OFF
TYPE - CENTRIFUGAL
CAPACITY - 775 m³/h
HEAD - 30 mWC
MOTOR - 110 kW (Inst)

06-TS-003 A/B
SLUDGE
HOLDING TANK - 1 OFF
EFFECTIVE VOLUME - 750 m³
SIZE (lxbxht) - 15 m x 15 m x 3.5 m

06-CT-101 TO 106
SLUDGE THICKENING
BELT PRESSES - 8 OFF
TYPE - XXXX
CAPACITY - m³/h
MOTOR - 5.5 kW (Inst)

Knight Piésold CONSULTING

PRIMARY DISCIPLINE	DISCIPLINE	
DRAWN	A.S.	MAY2013
DRAWING CHECK	CHECKER	DATE
DESIGN	G.L.	MAY2013
DESIGN CHECK	CHECK	DATE
PROJECT ENGINEER	ENGIN	DATE

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REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
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UMGENI WATER · AMANZI

REFERENCE DRAWINGS

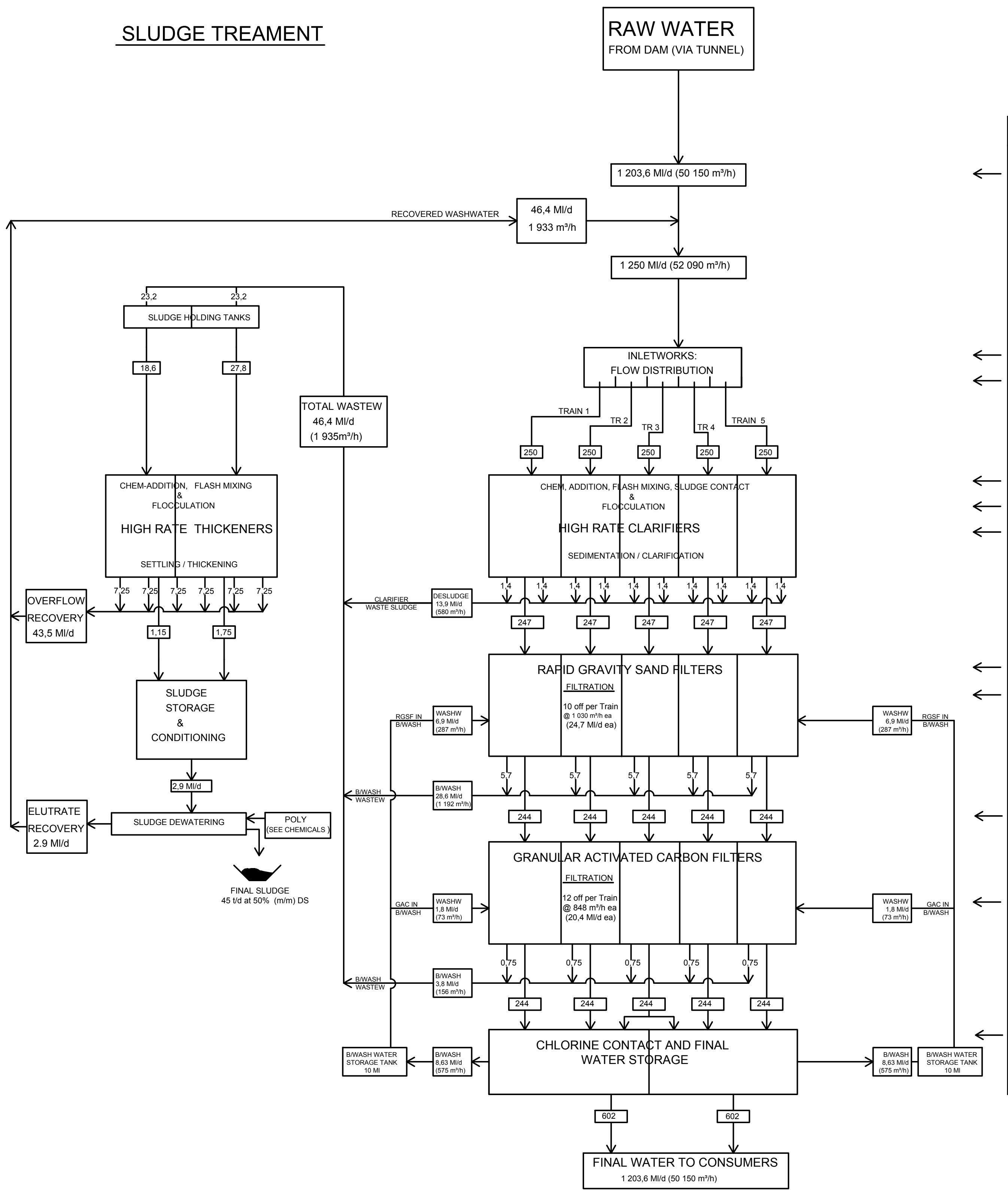
DRAWING No.	MAKERS No.	TITLE
–	–	–

PROJECT: uMKHOMAZI WATER PROJECT
1 250ML/d WATER TREATMENT PLANT

TITLE: P & ID

SHEET: SHEET 5 OF 5

DRG. No. 1301.X01.UW - 001 REV. 10



CHEMICAL REQUIREMENTS

USAGE / PURPOSE	CHEMICAL	DOSAGE RANGE Min mg/l	DOSAGE RANGE AV mg/l	DOSAGE RANGE Max mg/l	AVERAGE CONSUMPTION AT 500 MLD kg/d	AVERAGE CONSUMPTION AT 500 MLD Ton/mnth	AVERAGE CONSUMPTION AT 1 250 MLD kg/d	AVERAGE CONSUMPTION AT 1 250 MLD Ton/mnth
PRE - CHLORINATION	CHLORINE (GAS)	1	1,5	3	750	22,5	1 875	56,25
STABILIZATION	LIME (BROWN) (as commercially deliverd)	8	10	16	5 000	150	12 500	375
IRON OXIDATION	POTASSIUM PERMANGANATE	0,6	1,0	1,6	500	15	1 250	37,5
COAGULATION	ALUM (powder/chrystal) (as commercially deliverd)	10	15	25	7 500	225	18 750	562,5
BALLAST	BENTONITE	0	3	5	1 500	45	3 750	112,5
FLOCC AID	POLY (eg. U3 500)	0	1	2	500	15	1 250	37,5
AUGMENT LOSSES 5%/yr	ANTHRACITE 700mm deep	3%	5%	8%		5,48		14,6
AUGMENT LOSSES 5%/yr	FILTER SAND 800mm deep	3%	5%	8%		12,8		32,1
SLUDGE DEWATER. (sludge treatment)	POLY (e.g.U3500)	kg/TDS 4.5	kg/TDS 9	kg/TDS 13.5	828	24,9	2063	63,3
	LIME (BROWN) (as commercially deliverd)	mg/l 120	mg/l 150	mg/l 180	346	10,37	871	26,14
REPLACEMENT/ REGENERATION	GAC (e.g. AQ 2000) 1 500 mm deep	1x per 30 mnth	1x per 24 mnth	1x per 18 mnth		90		225
POST - CHLORINATION	CHLORINE (gas)	mg/l 2,0	mg/l 2,0	mg/l 2,0	1 000	30	2 500	75

Knight Piésold
CONSULTING

PRIMARY DISCIPLINE	DISCIPLINE	
DRAWN	A.S.	MAY2013
DRAWING CHECK	CHECKER	DATE
DESIGN	G.L.	MAY2013
DESIGN CHECK	CHECK	DATE
PROJECT ENGINEER	ENGIN	DATE

DRAWINGNO.

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
6	JUL2015	FINAL-WITH_UW_COMMENTS	GL	GL	

UMGENI
WATER • AMANZI

REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE
–	–	–

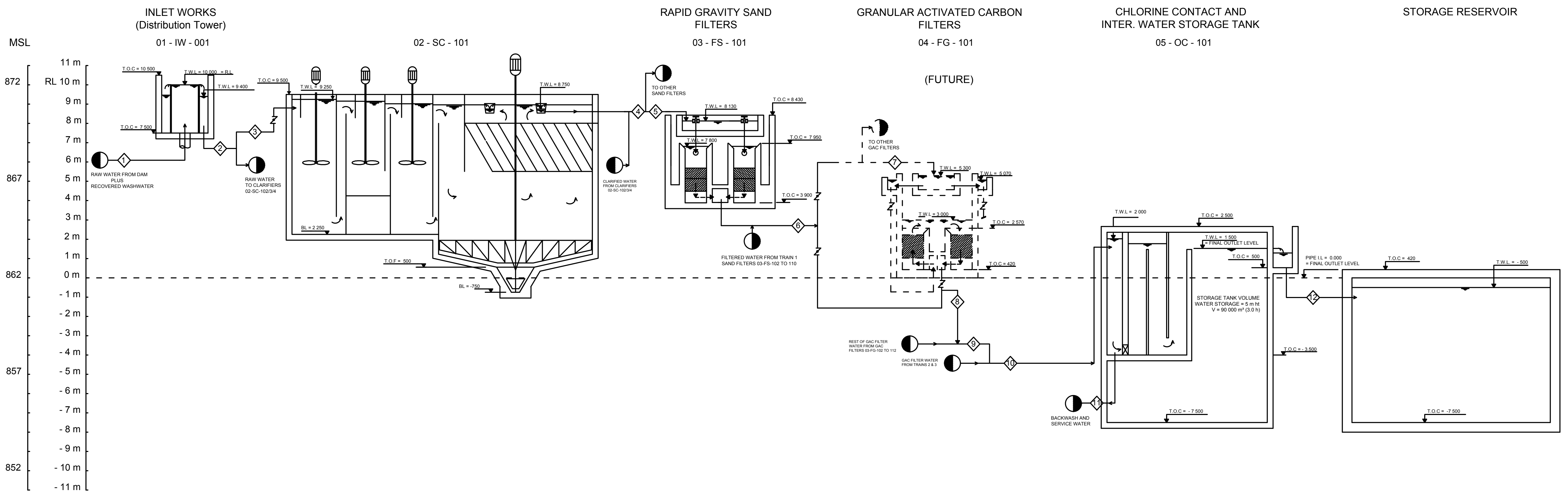
PROJECT: uMKHOMAZI WATER PROJECT
1 250ML/d WATER TREATMENT PLANT

TITLE: PROCESS FLOW DIAGRAM

SHEET:

DRG. No. 1301.X01.UW - 002 REV. 6

UMKHOMAZI WTW (PHASE 1 = 500 ML/d; PHASE 2 = 1 250 Ml/d) - HYDRAULIC PROFILE



PHASE 1: 500 ML/d	1	2	3	4	5	6	7	8	9	10	11	12
FLOW RATE (m^3/h)	52 090	10 420	2 605	10 300	1 030	10 180	848	848	10 180	20 360	900	50 150

Knight Piésold CONSULTING

PRIMARY DISCIPLINE	DISCIPLINE	
DRAWN	A.S.	MAY2013
DRAWING CHECK	CHECKER	DATE
DESIGN	G.L.	MAY2013
DESIGN CHECK	CHECK	DATE
PROJECT ENGINEER	ENGIN	DATE

DRAWINGNO.

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
8	JUL2015	FINAL-WITH_UW_COMMENTS	GL	GL	

UMGENI WATER · AMANZI

REFERENCE DRAWINGS

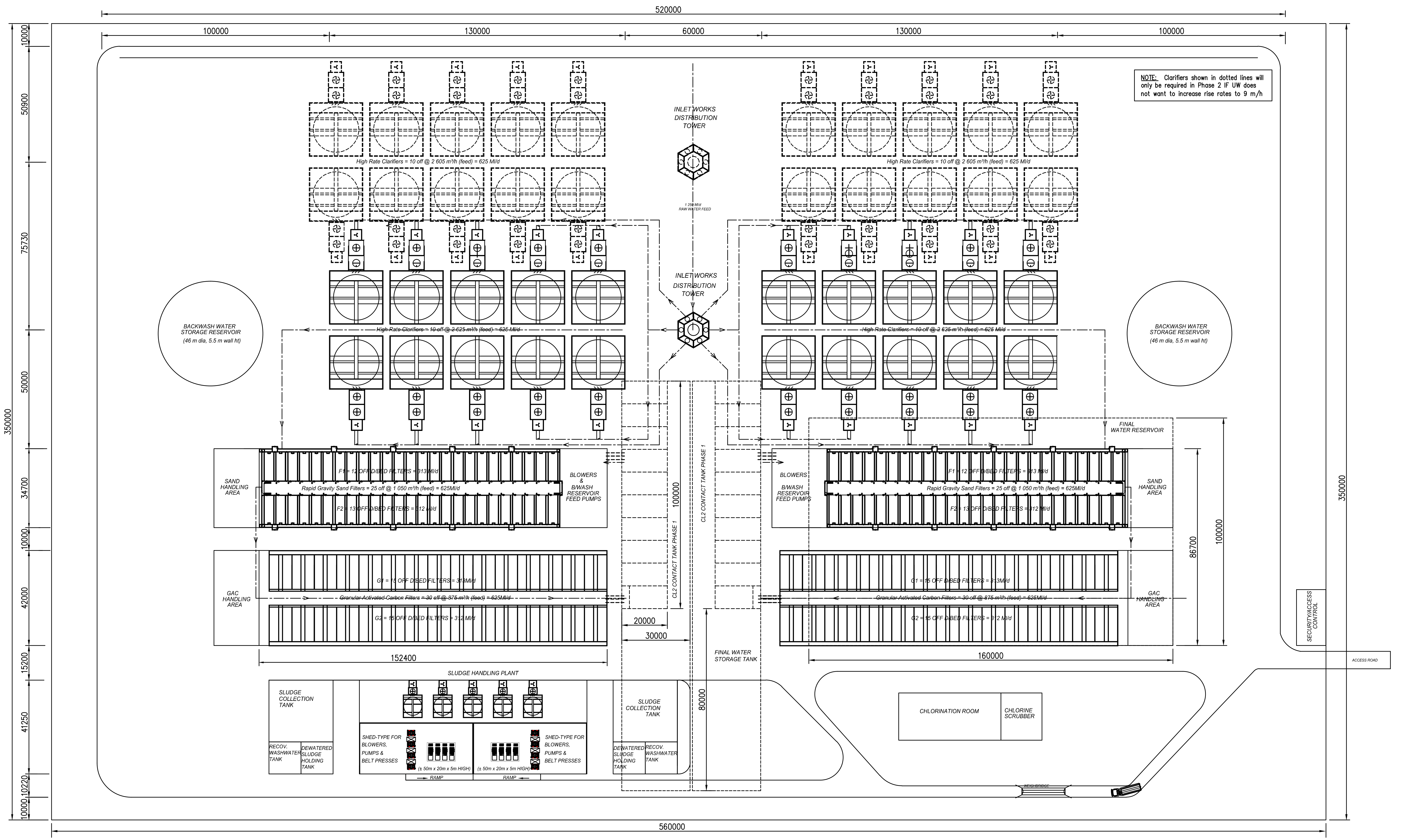
DRAWING No.	MAKERS No.	TITLE
–	–	–

PROJECT: uMKHOMAZI WATER PROJECT 1 250ML/d WATER TREATMENT PLANT

TITLE: HYDRAULIC PROFILE

SHEET:

DRG. No. 1301.X01.UW - 003 REV. 8



PRIMARY DISCIPLINE	DISCIPLINE			
DRAWN	A.S.	APRIL_13		
DRAWING CHECK	–	–		
DESIGN	G.L.	–		
DESIGN CHECK	–	–		
PROJECT ENGINEER	–	–		

Knight Piésold CONSULTING

DRAWINGNO.

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
10	JUL2015	FINAL–WITH_UW_COMMENTS REV	–	–	
–	–	REV REV	–	–	
–	–	REV REV	–	–	
–	–	REV REV	–	–	
–	–	REV REV	–	–	

UMGENI WATER • AMANZI

REFERENCE DRAWINGS

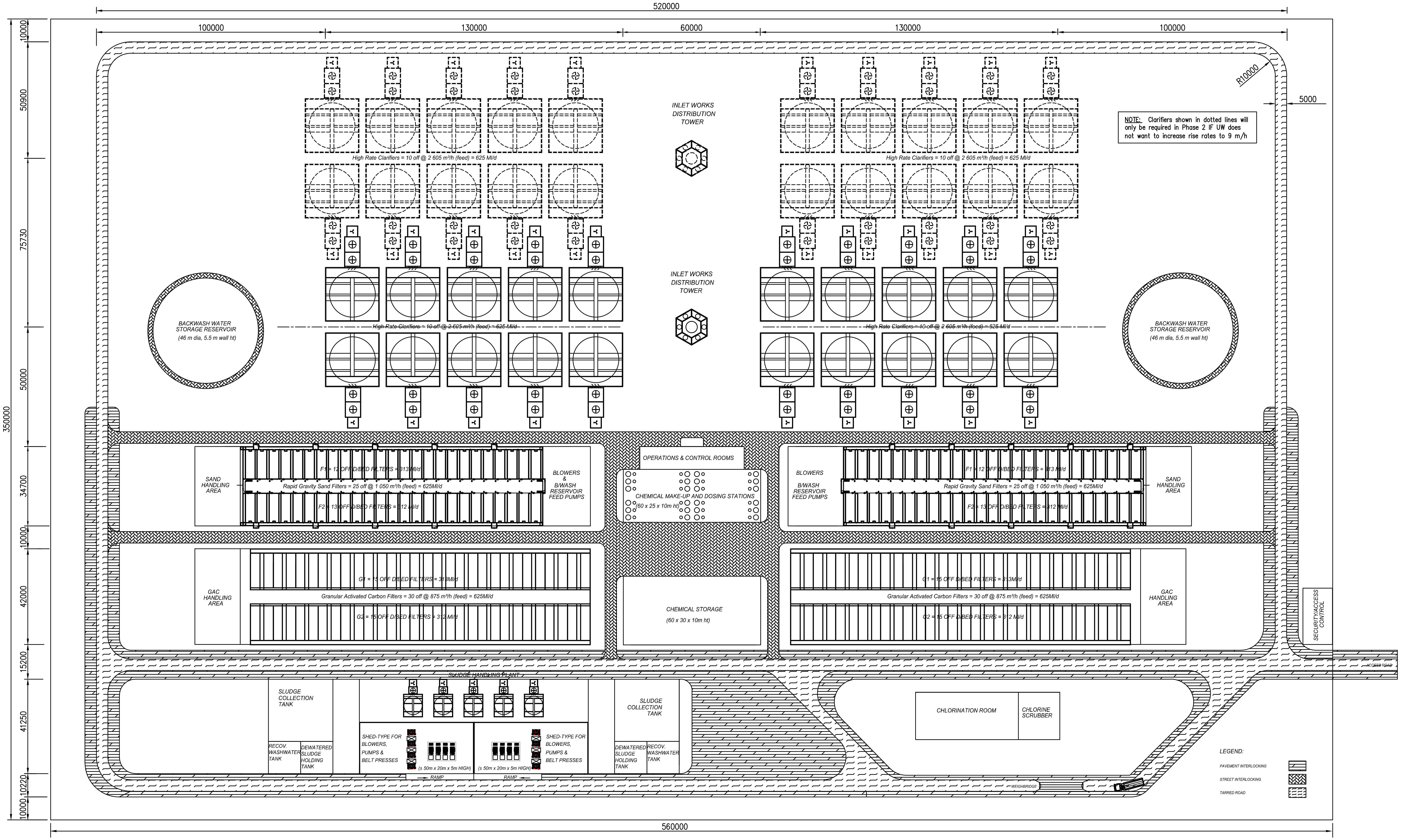
DRAWING No.	MAKERS No.	TITLE
–	–	–
–	–	–
–	–	–
–	–	–

PROJECT: **uMKHOMAZI PROJECT 1 250 ML/d WATER TREATMENT WORKS**

TITLE: **SITE LAYOUT**

SHEET: SHEET 1 OF 1

DRG. No. **1301.X01.UW-100** REV. 10



Knight Piésold CONSULTING

PRIMARY DISCIPLINE	DISCIPLINE	
DRAWN	A.S.	APRIL_13
DRAWING CHECK	–	–
DESIGN	G.L.	–
DESIGN CHECK	–	–
PROJECT ENGINEER	–	–

DRAWINGNO.

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
7	JUL2015	FINAL–WITH_UW_COMMENTS REV	–	–	
–	–	REV REV	–	–	
–	–	REV REV	–	–	
–	–	REV REV	–	–	
–	–	REV REV	–	–	

UMGENI WATER · AMANZI

REFERENCE DRAWINGS

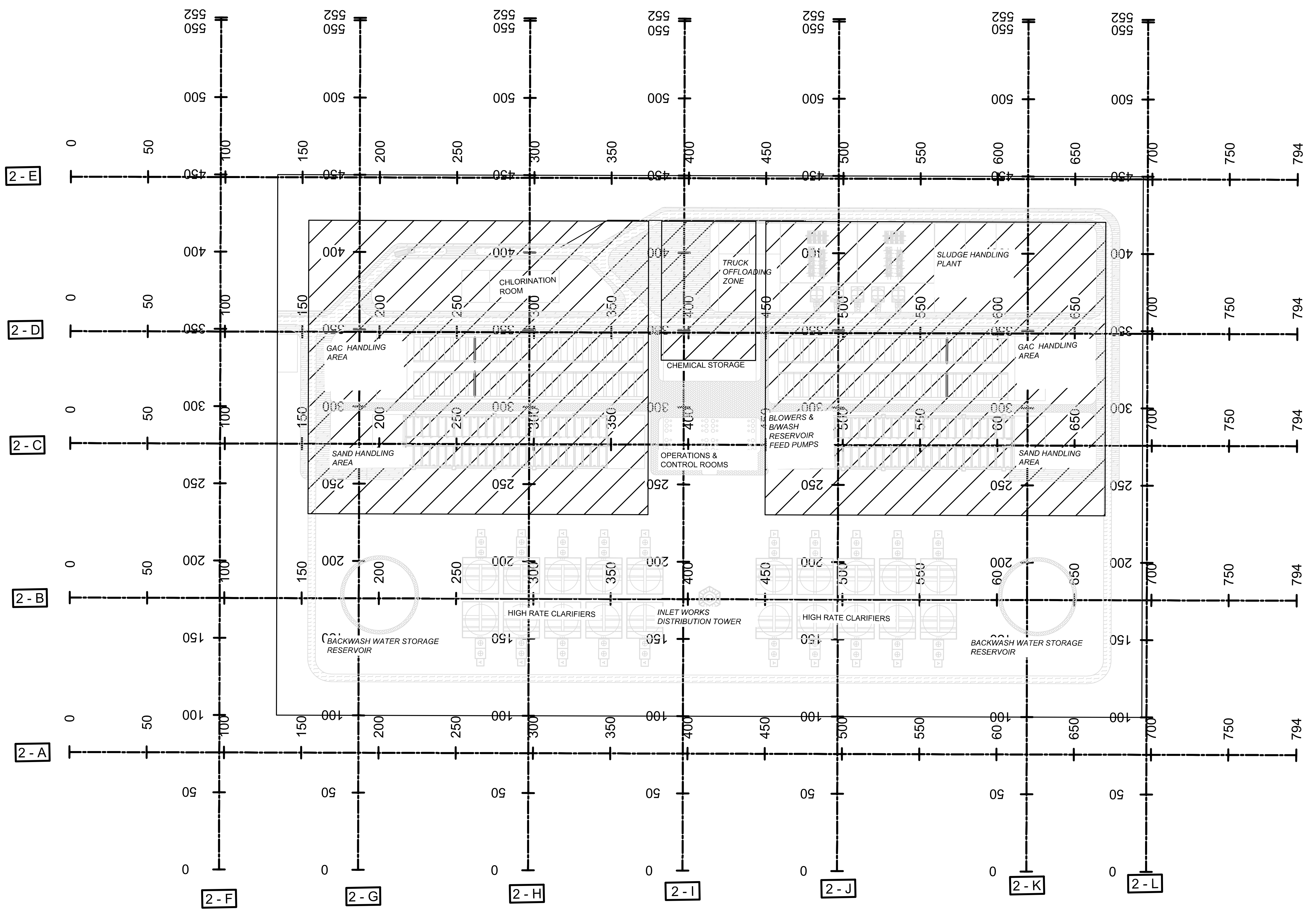
DRAWING No.	MAKERS No.	TITLE
–	–	–
–	–	–
–	–	–
–	–	–

PROJECT: **uMKHOMAZI PROJECT 1 250 ML/d WATER TREATMENT WORKS**

TITLE: **SITE WORKS**

SHEET: **SHEET 1 OF 1**

DRG. No. **1301.X01.UW-110** REV. 7



PLAN
SCALE 1: 2500

Knight Piésold
CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	K.G	OCT 2014
DRAWING CHECK	P.G	OCT 2014
DESIGN	P.G	OCT 2014
DESIGN CHECK	A.D	OCT 2014
PROJECT ENGINEER	A.D	OCT 2014

30300413/B07

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	10/2014	WATER TREATMENT WORKS – –OPTION 2 : PLAN	K.G	A.D	

UMGENI
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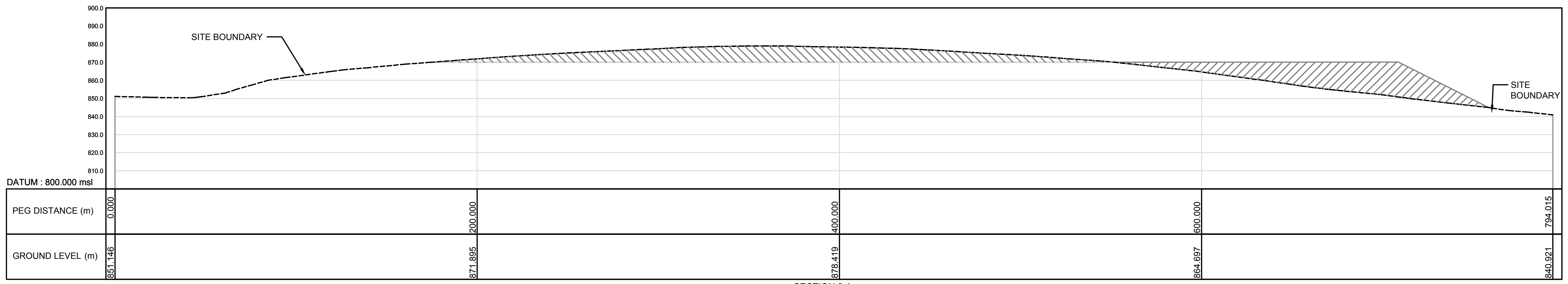
REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE

UMKHOMAZI WATER PROJECT

WATER TREATMENT WORKS
OPTION 2 SITE B2 PLAN

DRG. No. 30300413/B07 REV. A



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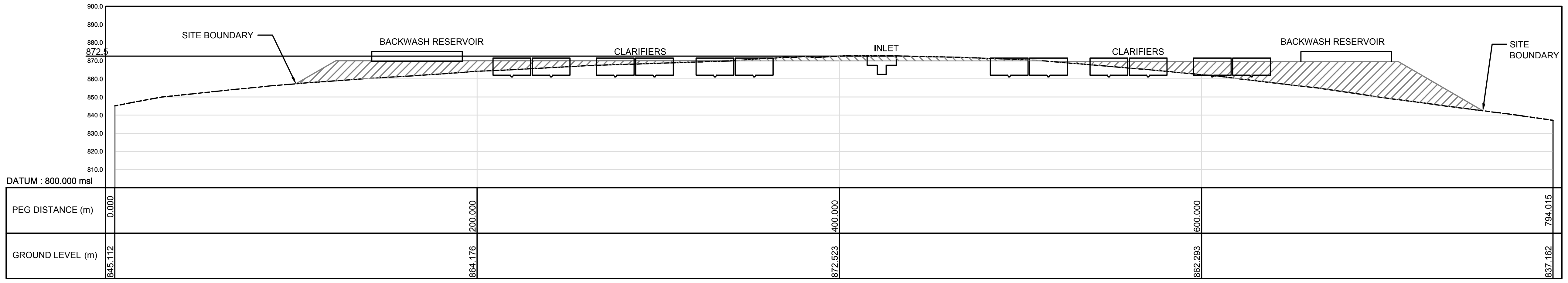

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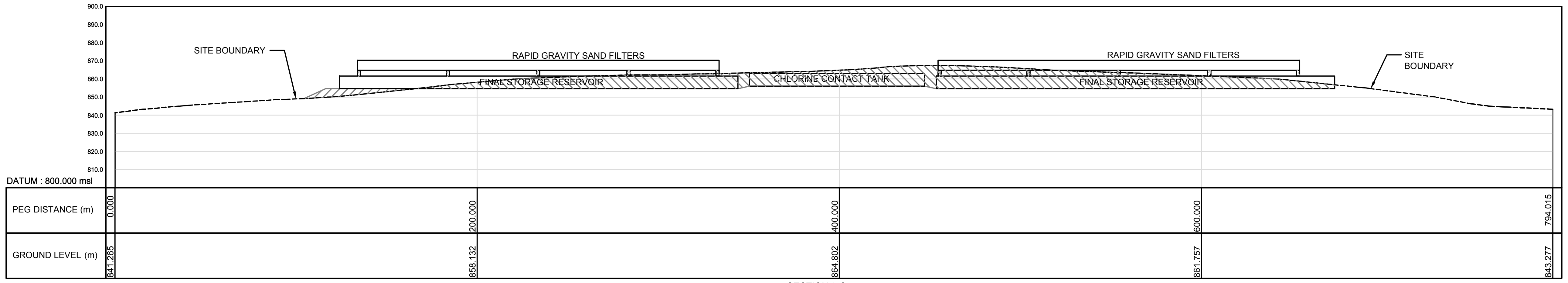

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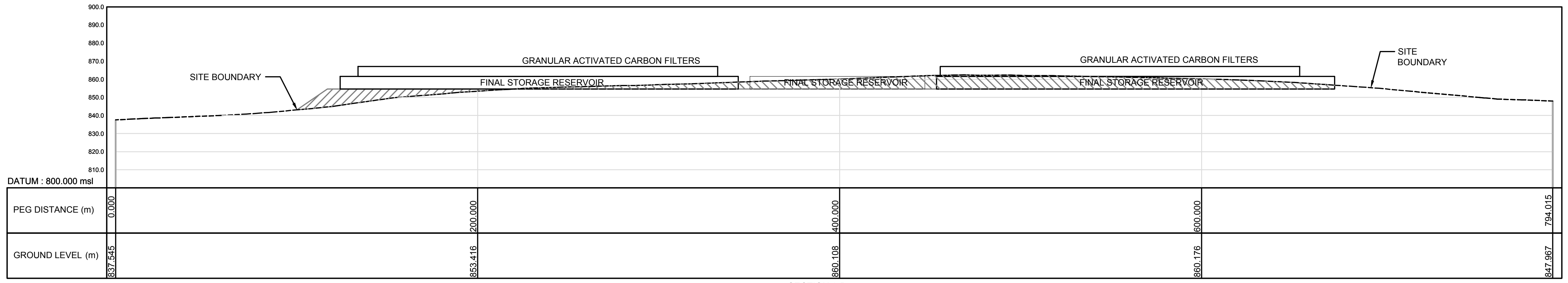

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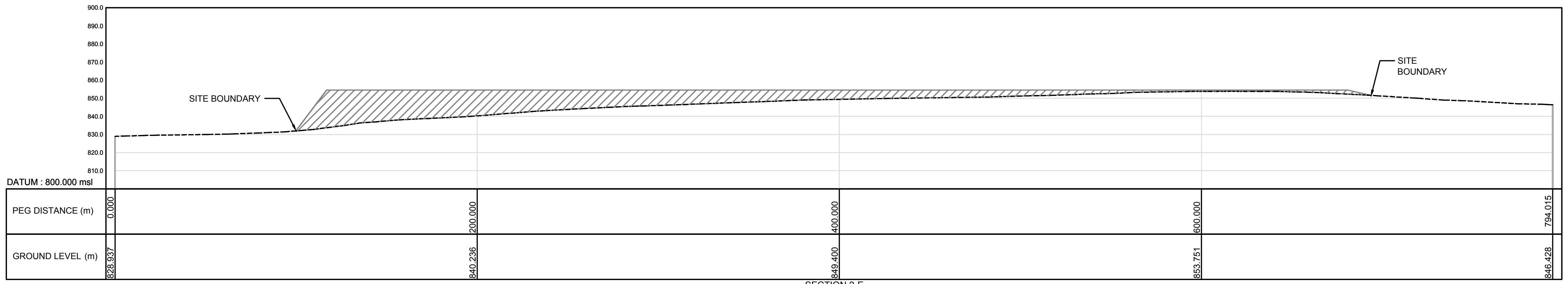


SECTION 2-E
FROM 0.000 TO 794.015
SCALE 1: 2500

Knight Piésold CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	K.G	OCT 2014
DRAWING CHECK	P.G	OCT 2014
DESIGN	P.G	OCT 2014
DESIGN CHECK	A.D	OCT 2014
PROJECT ENGINEER	A.D	OCT 2014

30300413/G05

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	10/2014	WATER TREATMENT WORKS – –OPTION 2 : CROSS SECTIONS	K.G	A.D	

UMGENI WATER · AMANZI

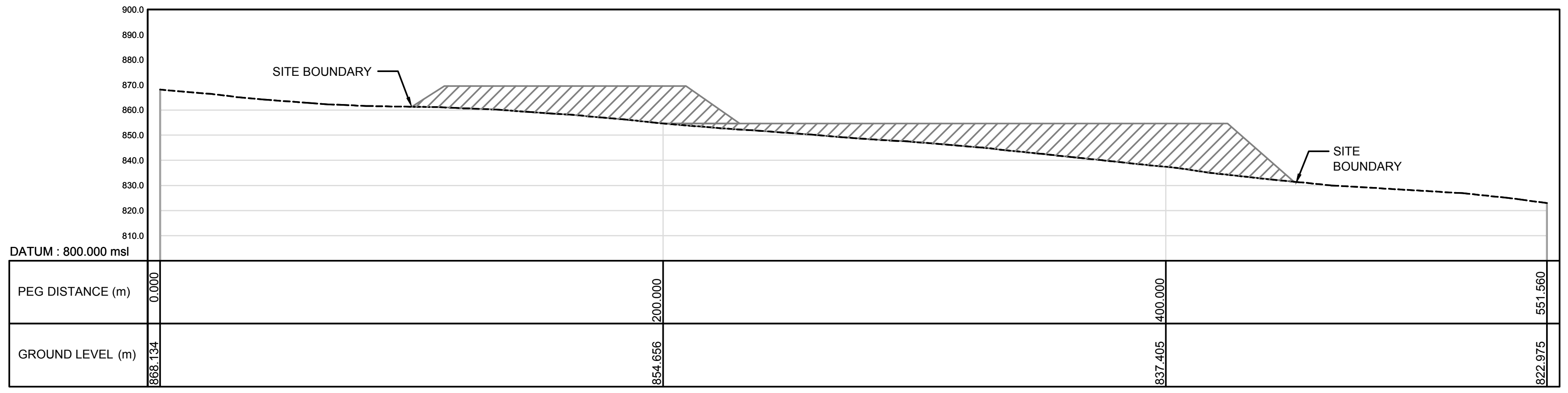
REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE

UMKHOMAZI WATER PROJECT

WATER TREATMENT WORKS OPTION 2
SITE B2 CROSS SECTIONS SHEET 1 OF 2

DRG. No. 30300413/G05 REV. A



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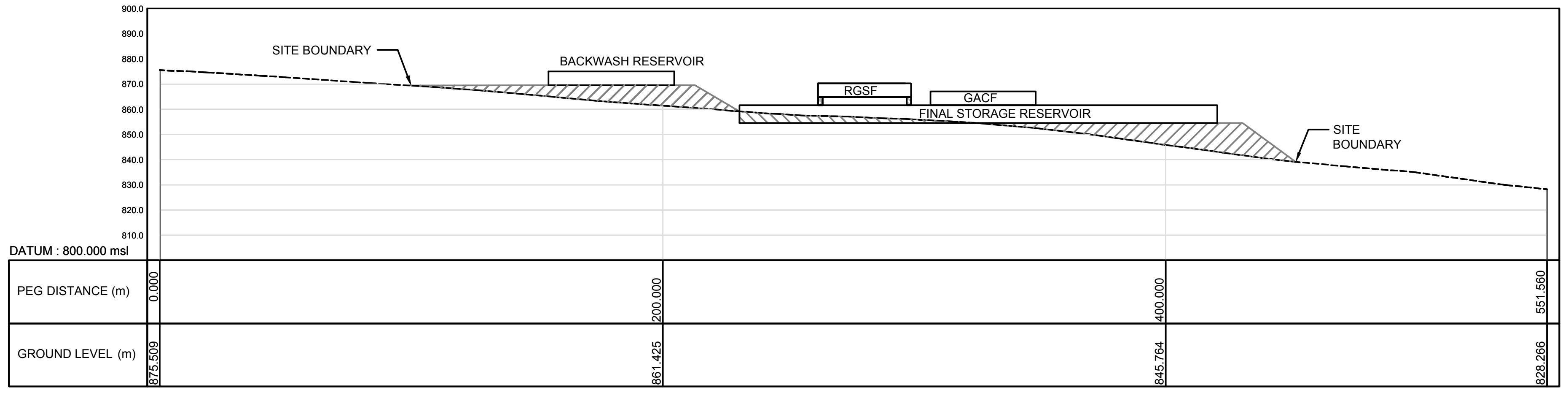
SCALE 1: 2500



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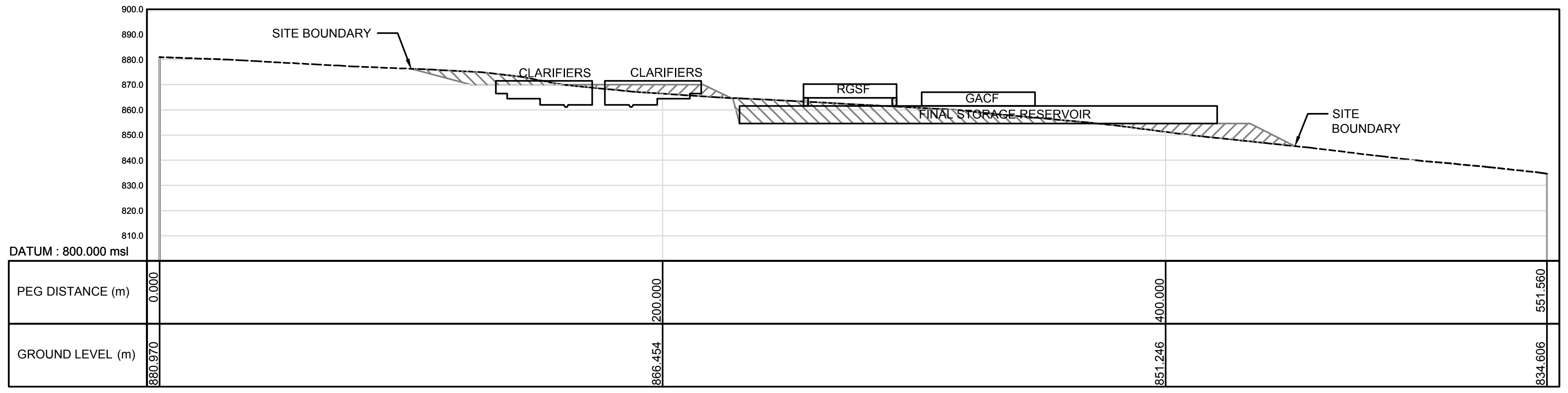

SECTION 2-H
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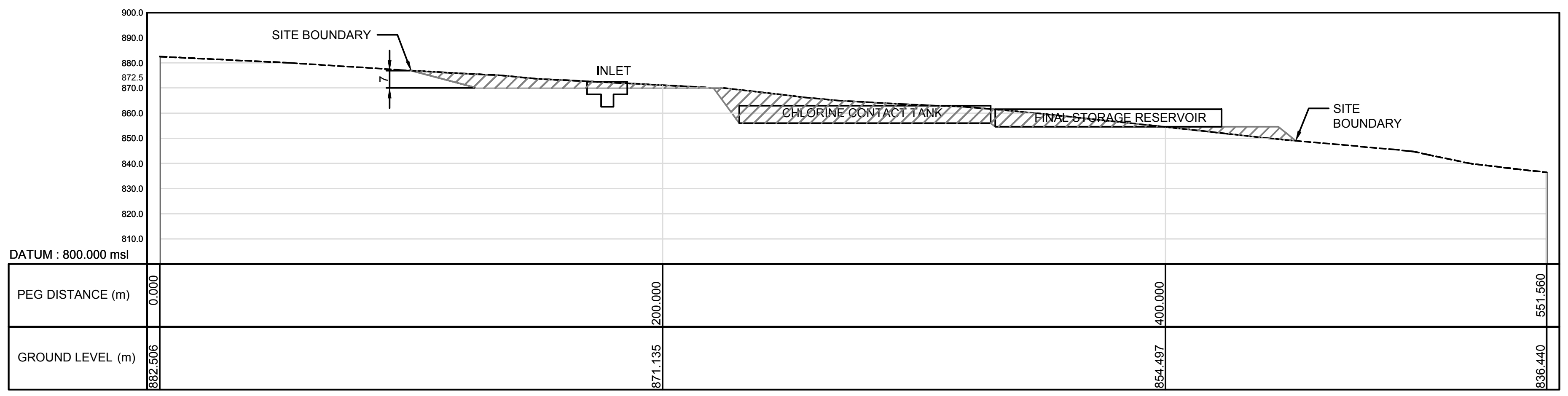

SECTION 2-I
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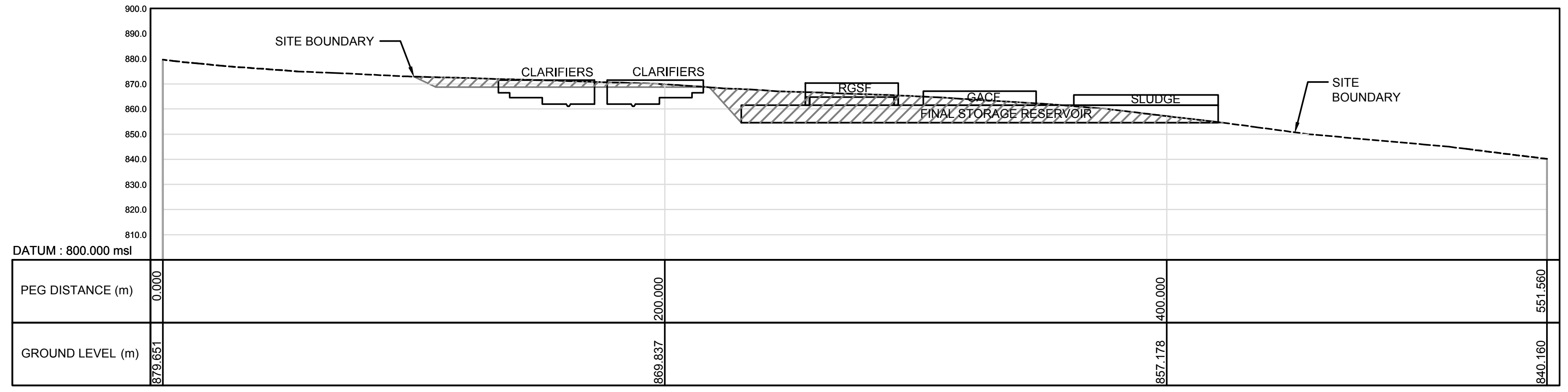

SECTION 2-J
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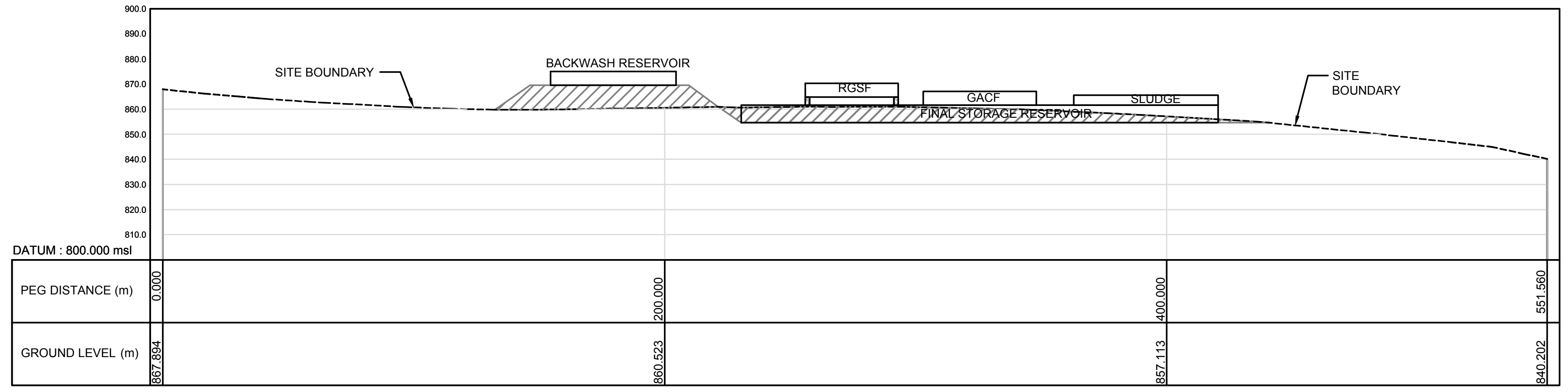

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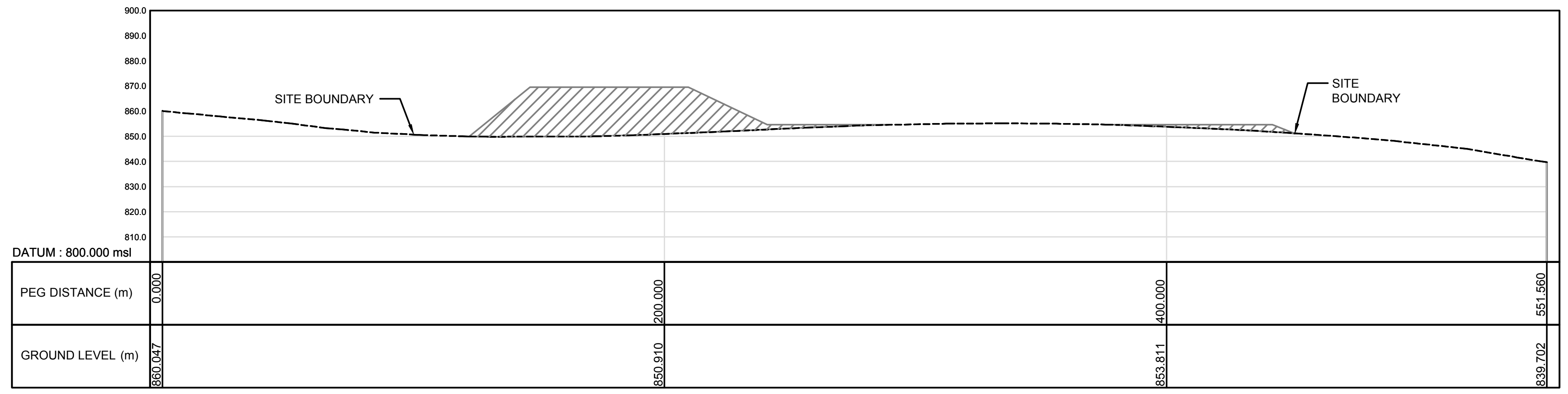

SECTION 2-L
FROM 0.000 TO 551.560
SCALE 1: 2500

Knight Piésold
CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	K.G	OCT 2014
DRAWING CHECK	P.G	OCT 2014
DESIGN	P.G	OCT 2014
DESIGN CHECK	A.D	OCT 2014
PROJECT ENGINEER	A.D	OCT 2014

30300413/G06

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	10/2014	WATER TREATMENT WORKS – –OPTION 2 : CROSS SECTIONS	K.G	A.D	

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WATER • AMANZI

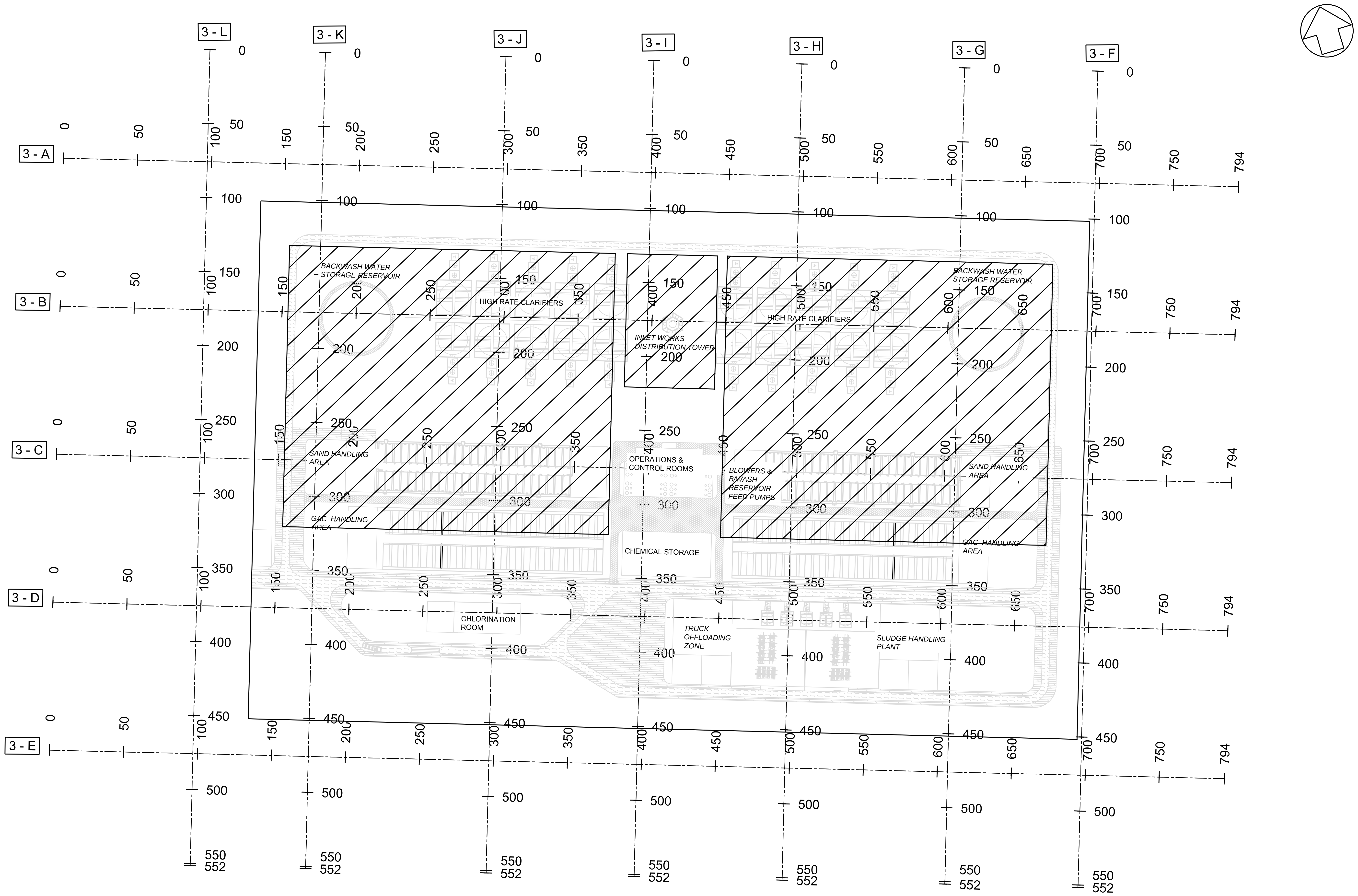
REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE

UMKHOMAZI WATER PROJECT

WATER TREATMENT WORKS OPTION 2
SITE B2 CROSS SECTIONS SHEET 2 OF 2

DRG. No. 30300413/G06 REV. A



PLAN
SCALE 1: 2500

Knight Piésold CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	K.G	OCT 2014
DRAWING CHECK	P.G	OCT 2014
DESIGN	P.G	OCT 2014
DESIGN CHECK	A.D	OCT 2014
PROJECT ENGINEER	A.D	OCT 2014

30300413/B08

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	10/2014	WATER TREATMENT WORKS – –OPTION 3 : SITE PLAN	K.G	A.D	

UMGENI WATER · AMANZI

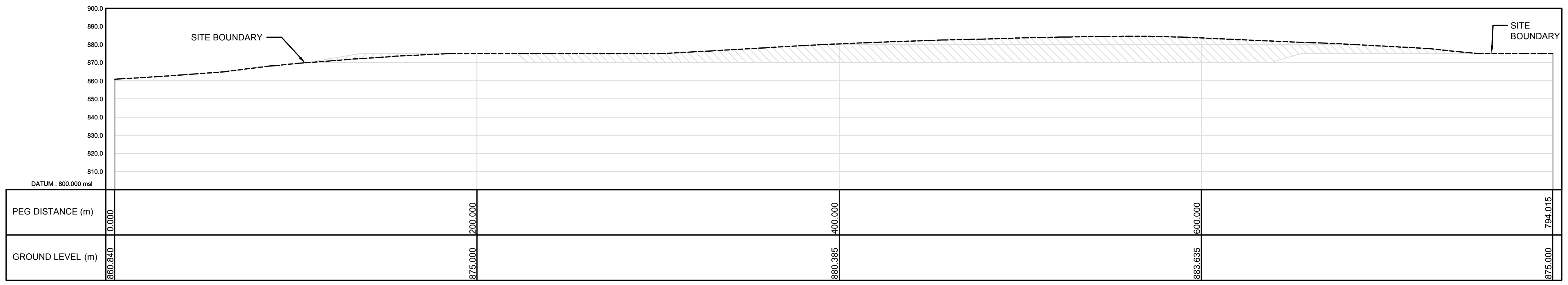
REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE

UMKHOMAZI WATER PROJECT

WATER TREATMENT WORKS
OPTION 3 SITE B3 PLAN

DRG. No. 30300413/B08 REV. A



SECTION 3-A
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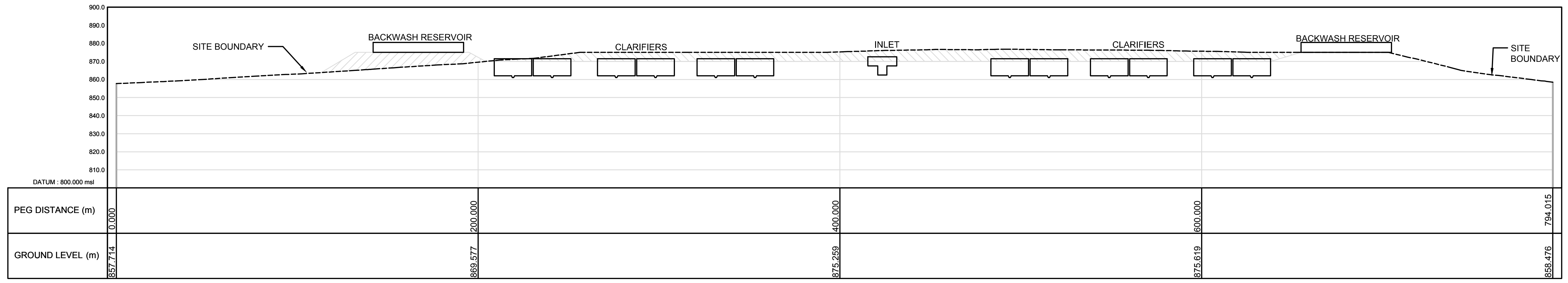

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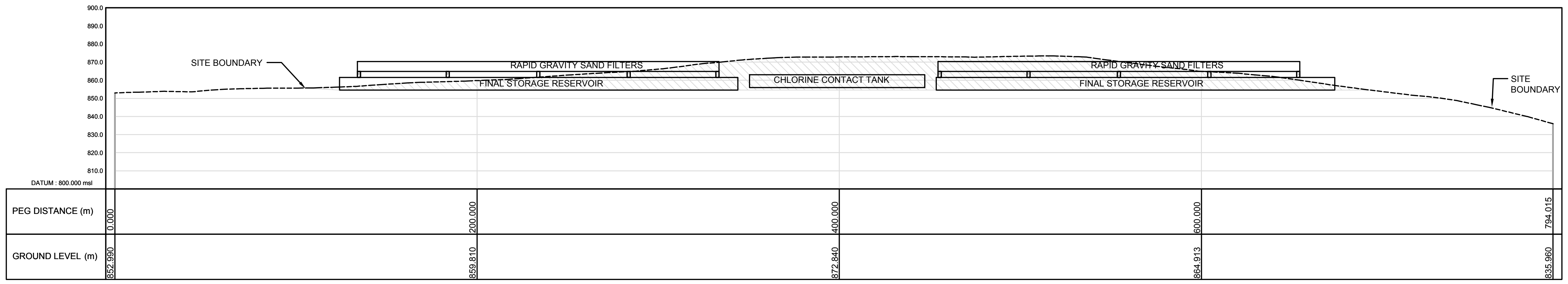

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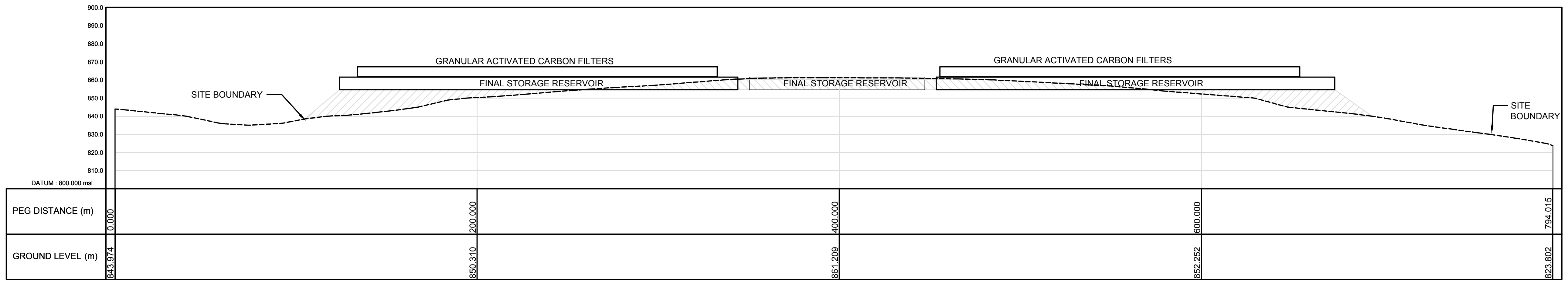

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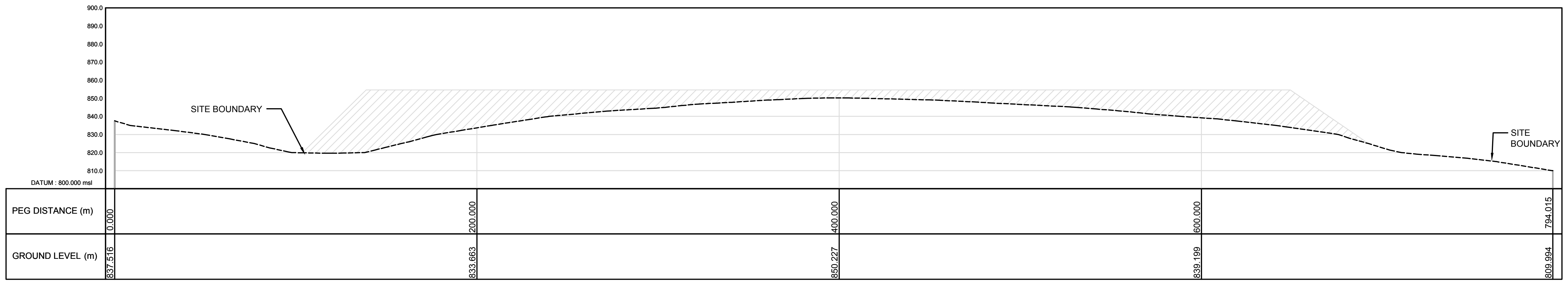


SECTION 3-E
FROM 0.000 TO 794.015
SCALE 1: 2500

Knight Piésold CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	K.G	OCT 2014
DRAWING CHECK	P.G	OCT 2014
DESIGN	P.G	OCT 2014
DESIGN CHECK	A.D	OCT 2014
PROJECT ENGINEER	A.D	OCT 2014

30300413/G07

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	10/2014	WATER TREATMENT WORKS – –OPTION 3 : CROSS SECTIONS	K.G	A.D	

UMGENI WATER · AMANZI

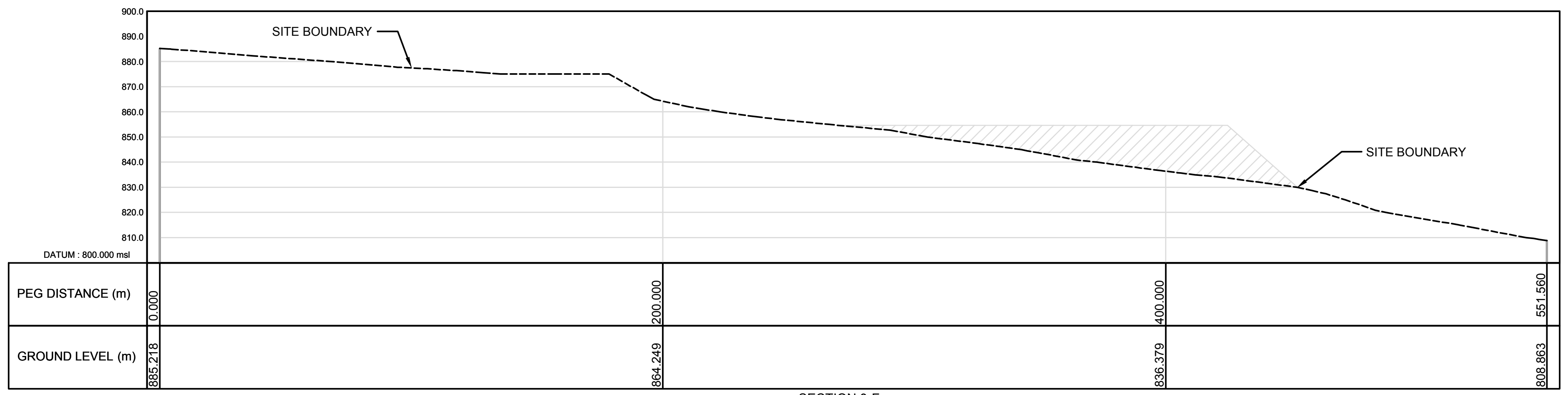
REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE

UMKHOMAZI WATER PROJECT

WATER TREATMENT WORKS OPTION 3
SITE B3 CROSS SECTIONS SHEET 1 OF 2

DRG. No. 30300413/G07 REV. A



SECTION 3-F
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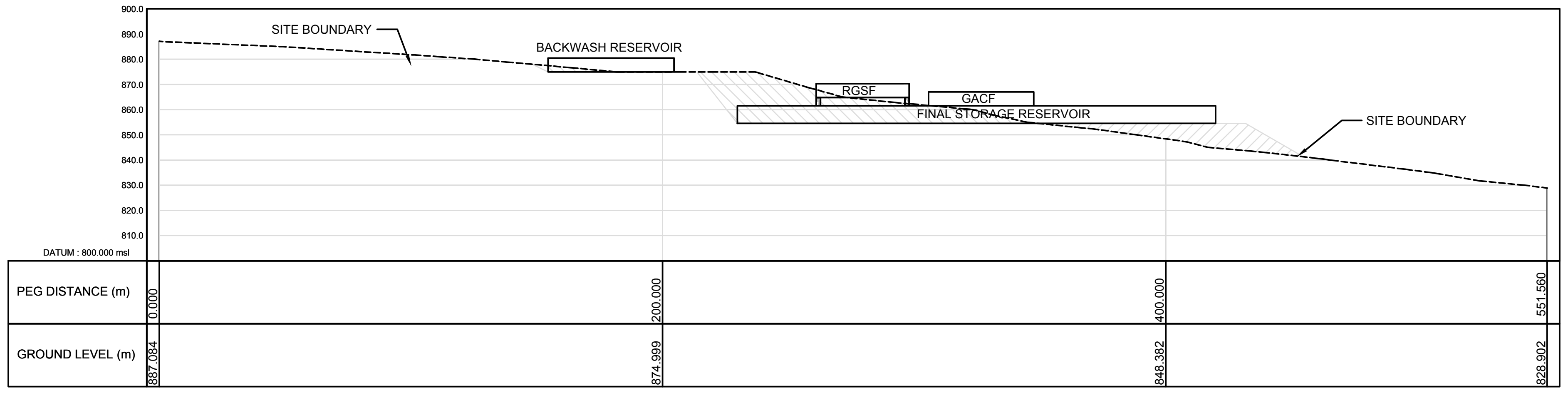
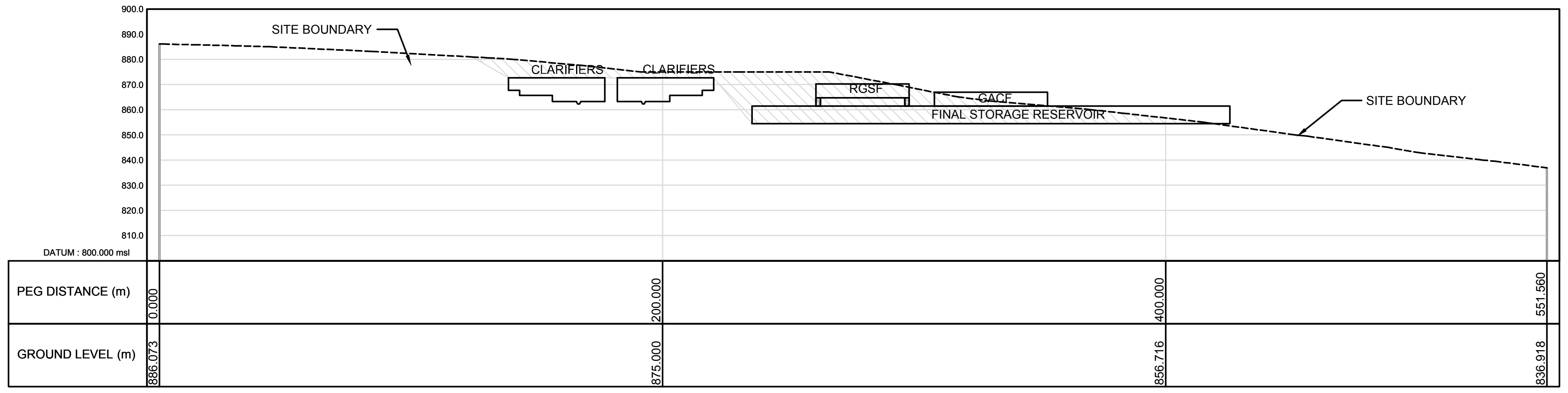


SECTION 3-G
FROM 0.000 TO 551.560
SCALE 1: 2500



SECTION 3-H
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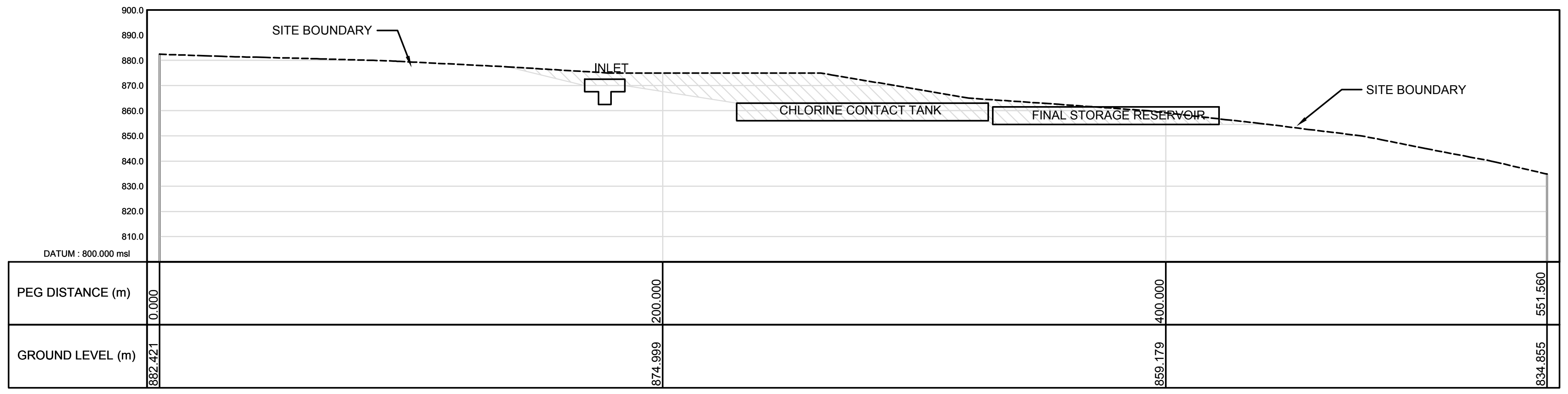

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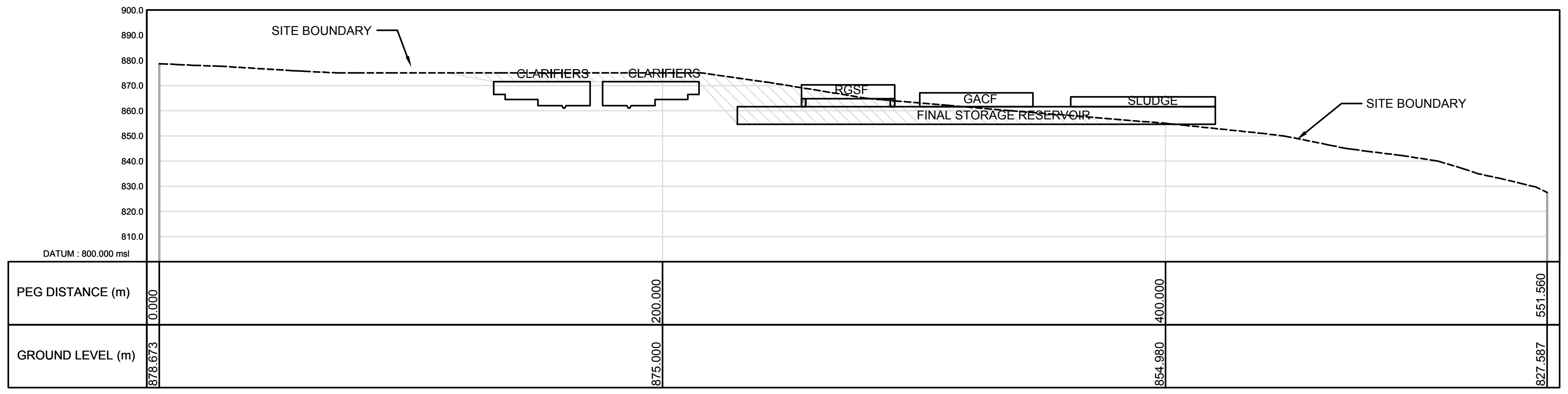

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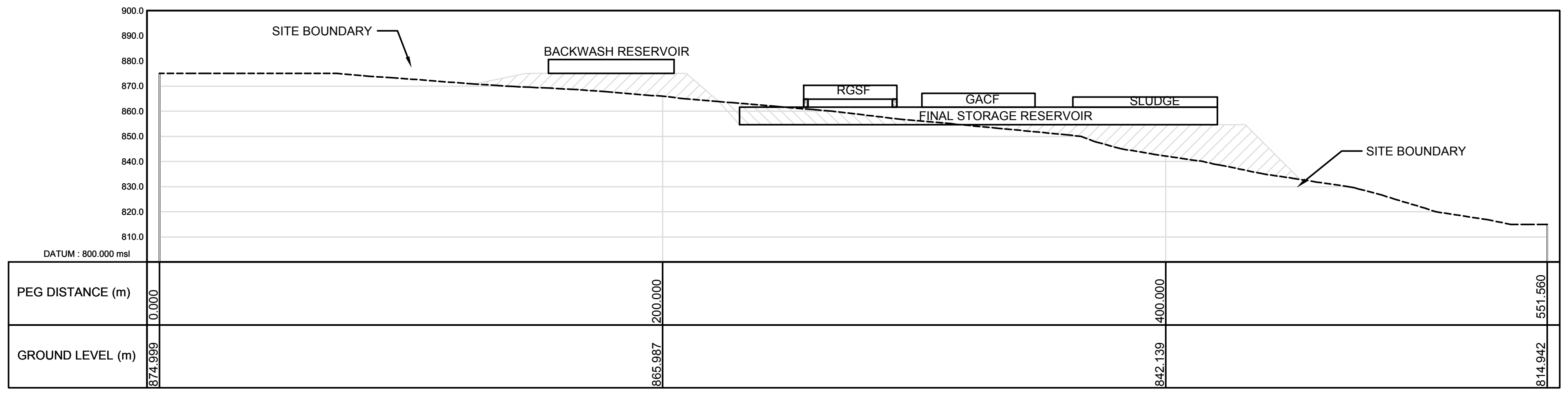

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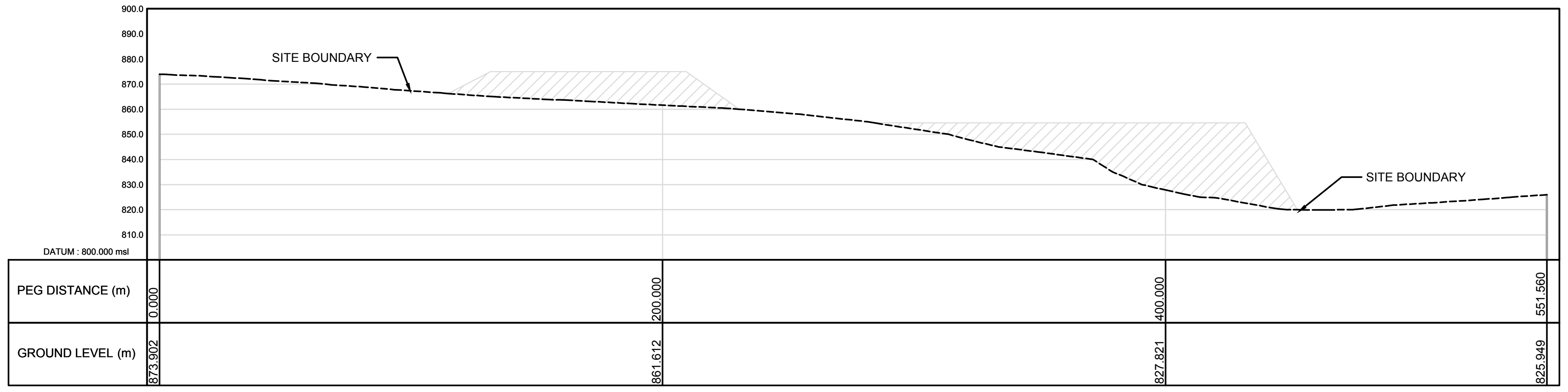


SECTION 3-L
FROM 0.000 TO 551.560
SCALE 1: 2500

Knight Piésold CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	K.G	OCT 2014
DRAWING CHECK	P.G	OCT 2014
DESIGN	P.G	OCT 2014
DESIGN CHECK	A.D	OCT 2014
PROJECT ENGINEER	A.D	OCT 2014

30300413/G08

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	10/2014	WATER TREATMENT WORKS – –OPTION 3 : CROSS SECTIONS	K.G	A.D	

UMGENI WATER · AMANZI

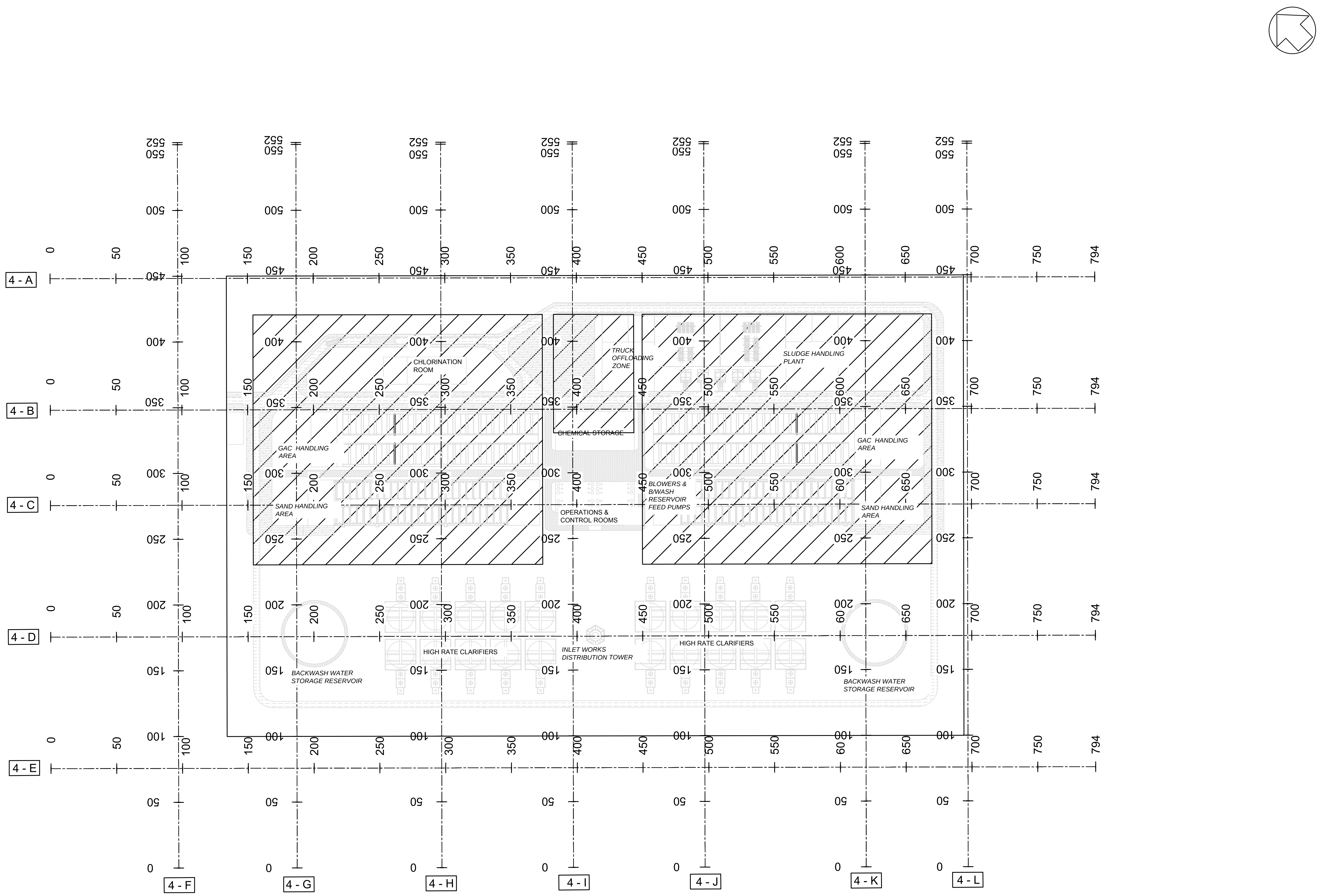
REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE

UMKHOMAZI WATER PROJECT

WATER TREATMENT WORKS OPTION 3
SITE B3 CROSS SECTIONS SHEET 2 OF 2

DRG. No. 30300413/G08 REV. A



PLAN

SCALE 1: 2500

PRIMARY DISCIPLINE	CIVIL	
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DRAWING CHECK	P.G	OCT 2014
DESIGN	P.G	OCT 2014
DESIGN CHECK	A.D	OCT 2014
PROJECT ENGINEER	A.D	OCT 2014

30300413/B09

Knight Piésold CONSULTING

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	10/2014	WATER TREATMENT WORKS – –OPTION 4 : PLAN	K.G	A.D	

UMGENI WATER · AMANZI

REFERENCE DRAWINGS

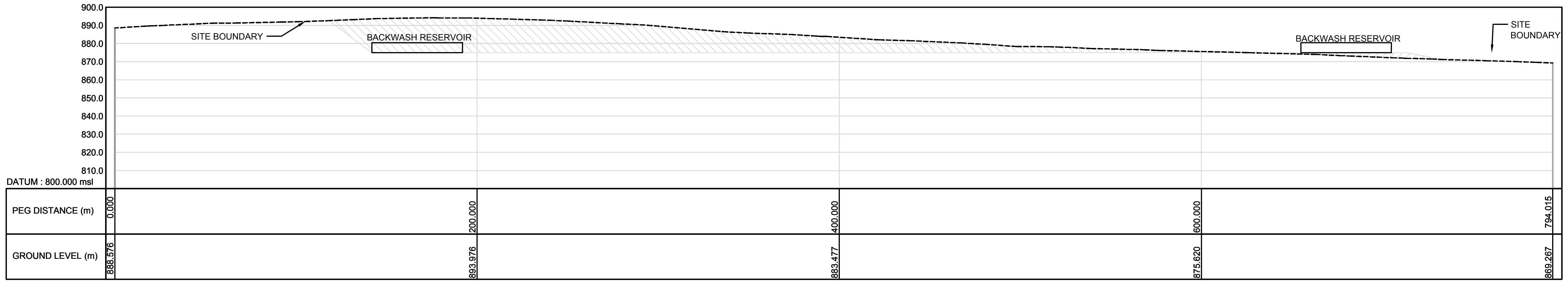
DRAWING No.	MAKERS No.	TITLE

UMKHOMAZI WATER PROJECT

WATER TREATMENT WORKS OPTION 4

SITE M1 PLAN

DRG. No. 30300413/B09 REV. A



SECTION 4-A
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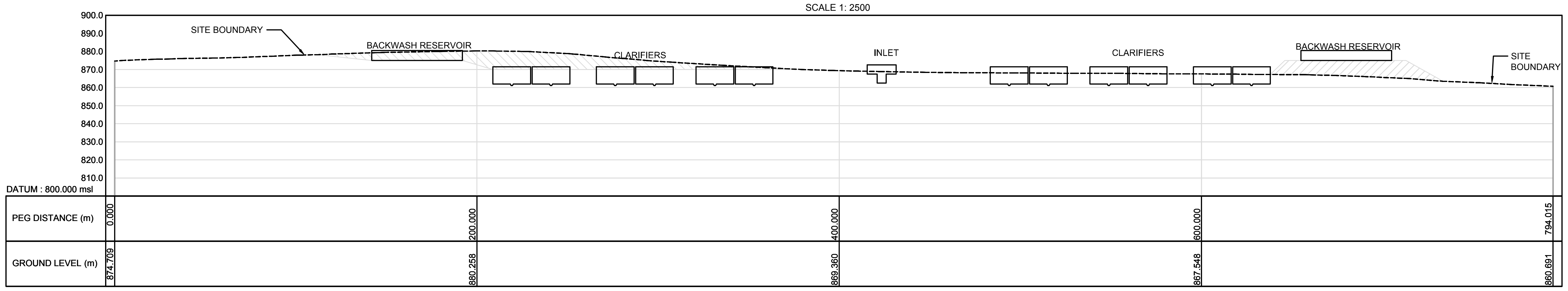

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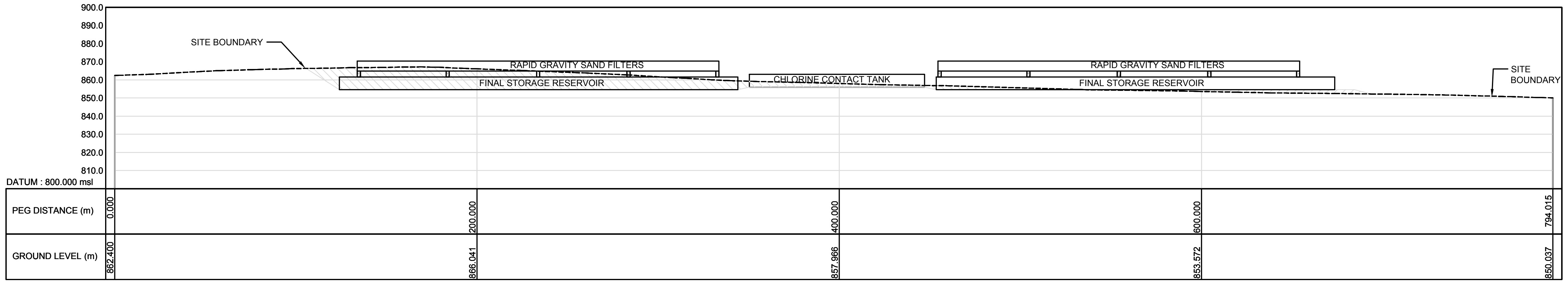

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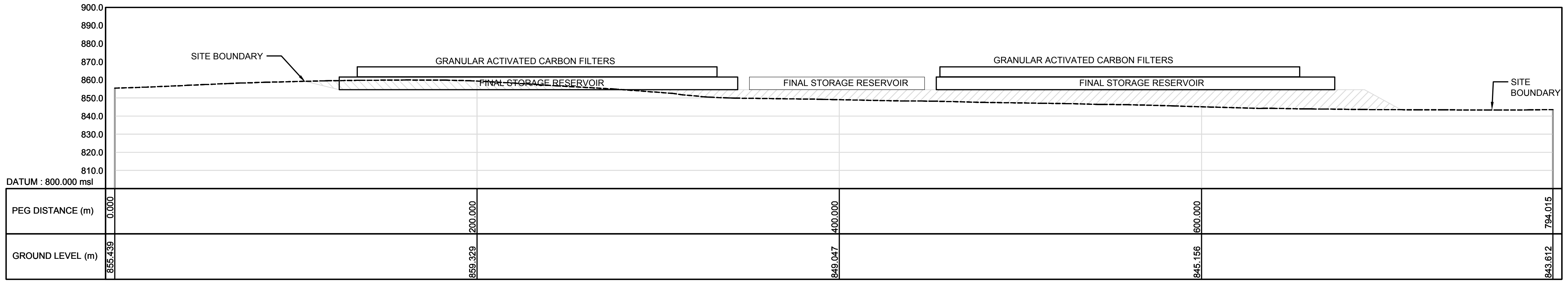

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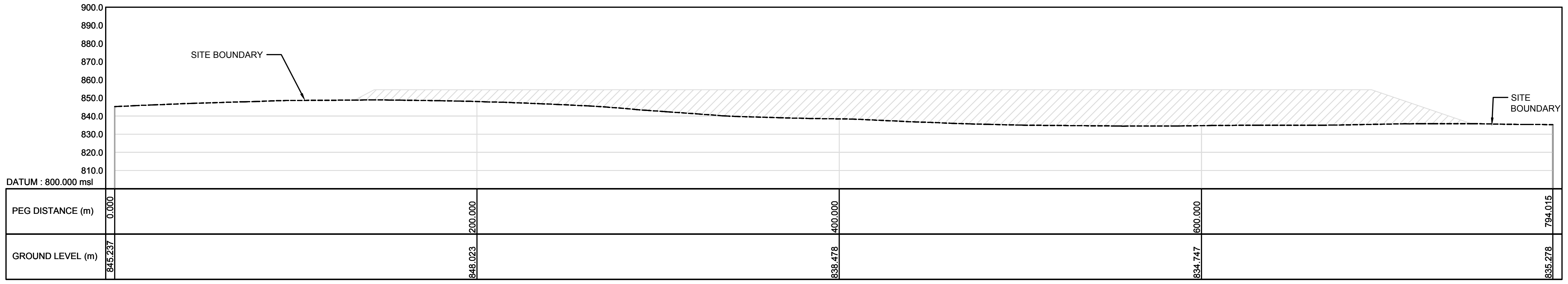


SECTION 4-E
FROM 0.000 TO 794.015
SCALE 1: 2500

Knight Piésold
CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	K.G	OCT 2014
DRAWING CHECK	P.G	OCT 2014
DESIGN	P.G	OCT 2014
DESIGN CHECK	A.D	OCT 2014
PROJECT ENGINEER	A.D	OCT 2014

30300413/G09

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	10/2014	WATER TREATMENT WORKS – –OPTION 4 : CROSS SECTIONS	K.G	A.D	

UMGENI
WATER • AMANZI

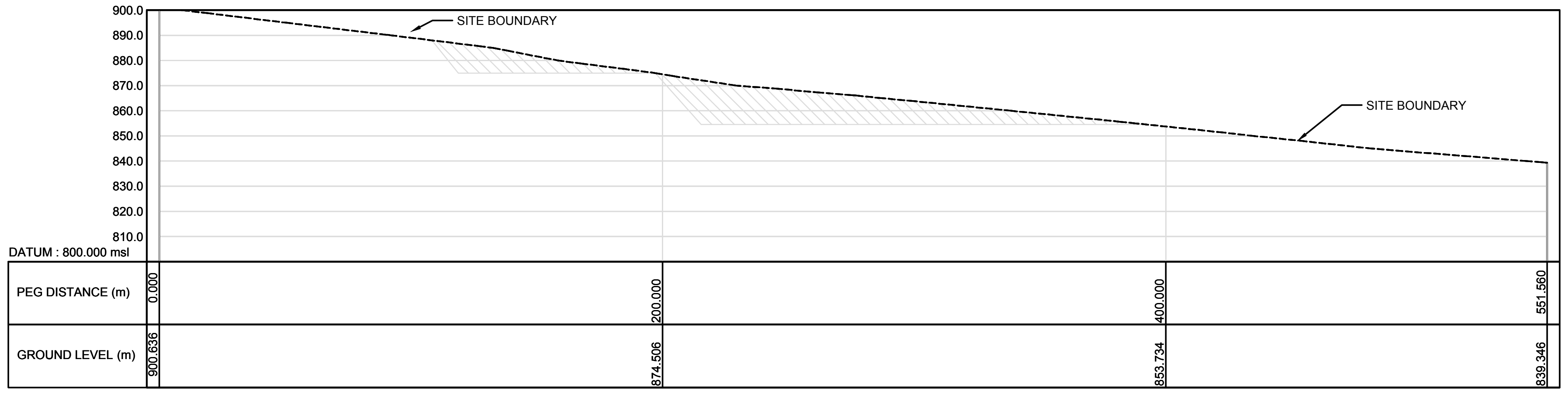
REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE

UMKHOMAZI WATER PROJECT

WATER TREATMENT WORKS OPTION 4
SITE M1 CROSS SECTIONS SHEET 1 OF 2

DRG. No. 30300413/G09 REV. A



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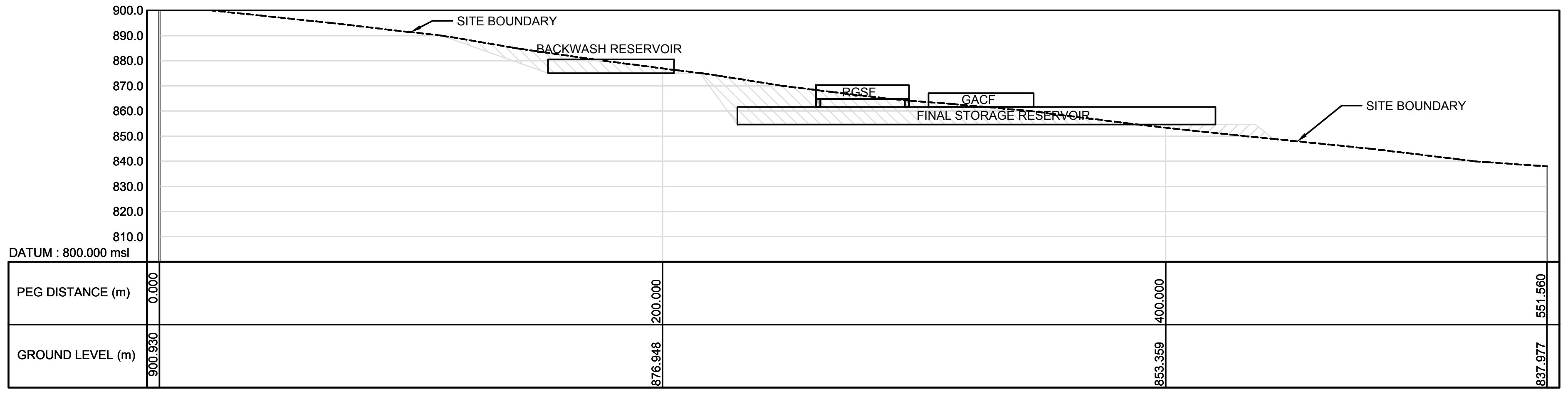

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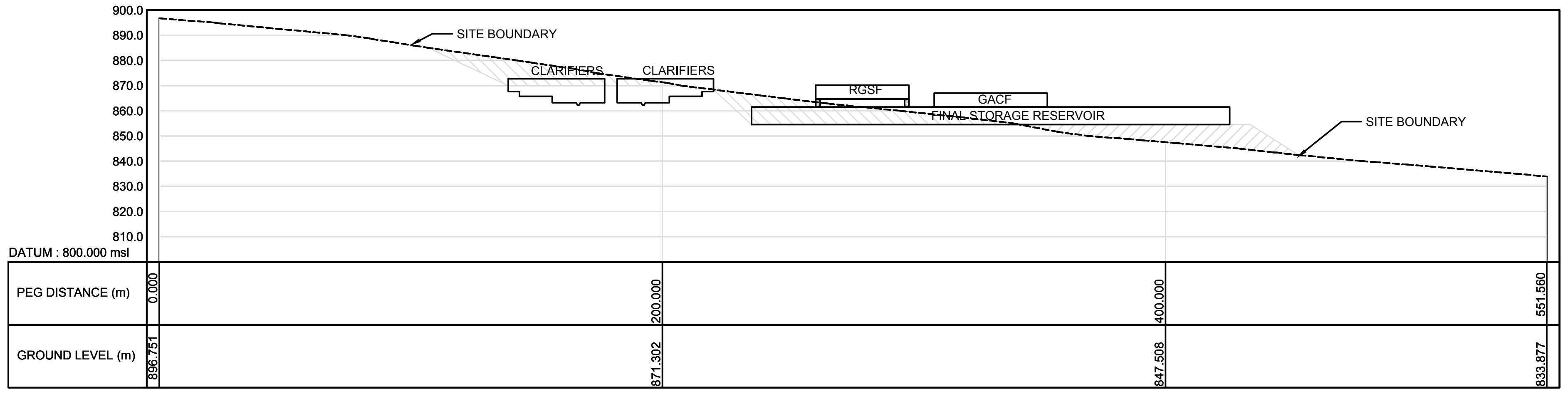

SECTION 4-H
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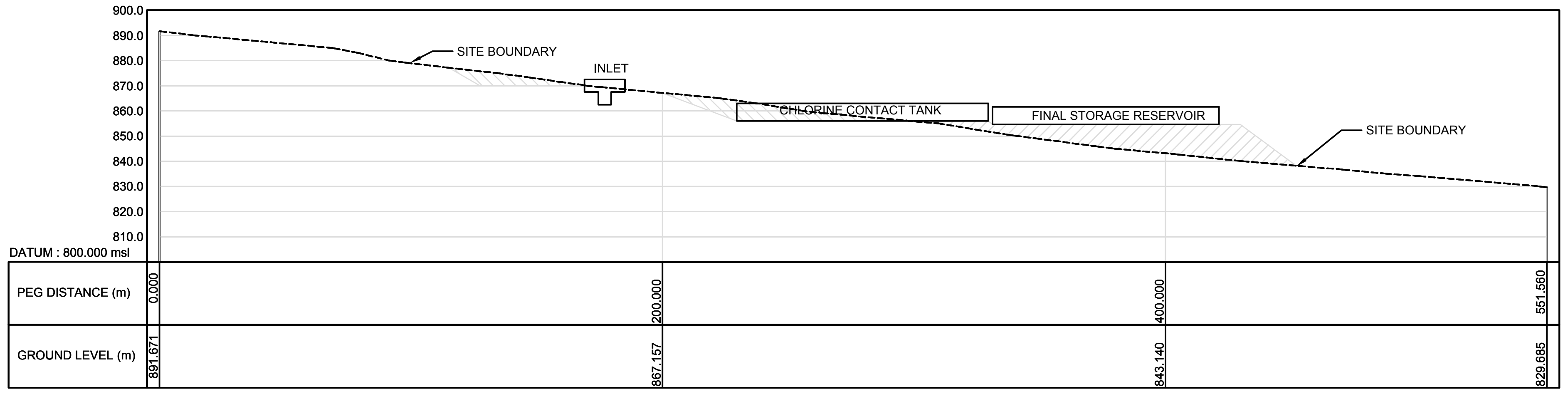

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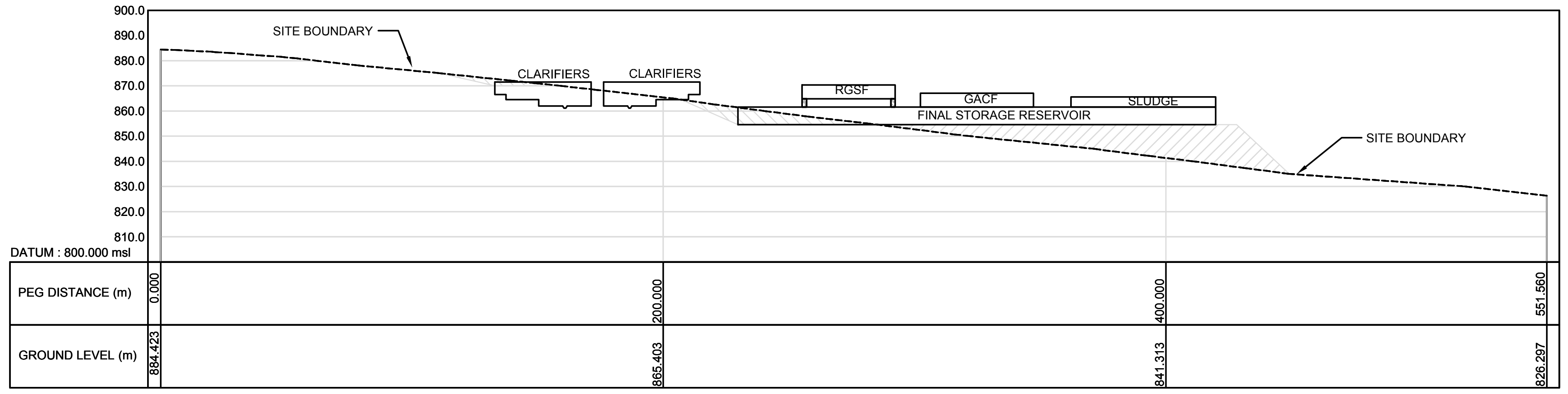

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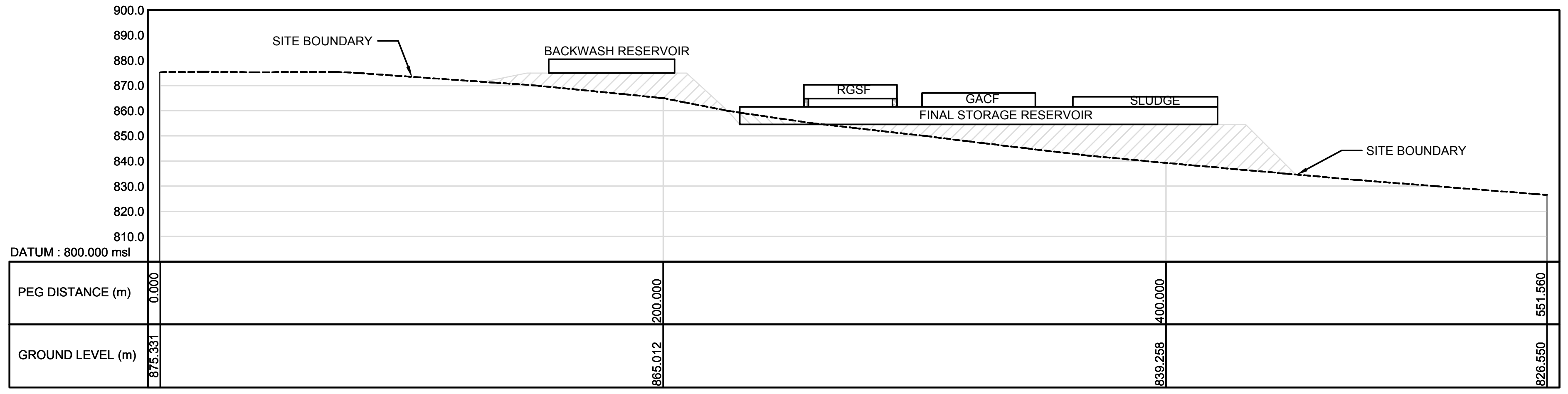

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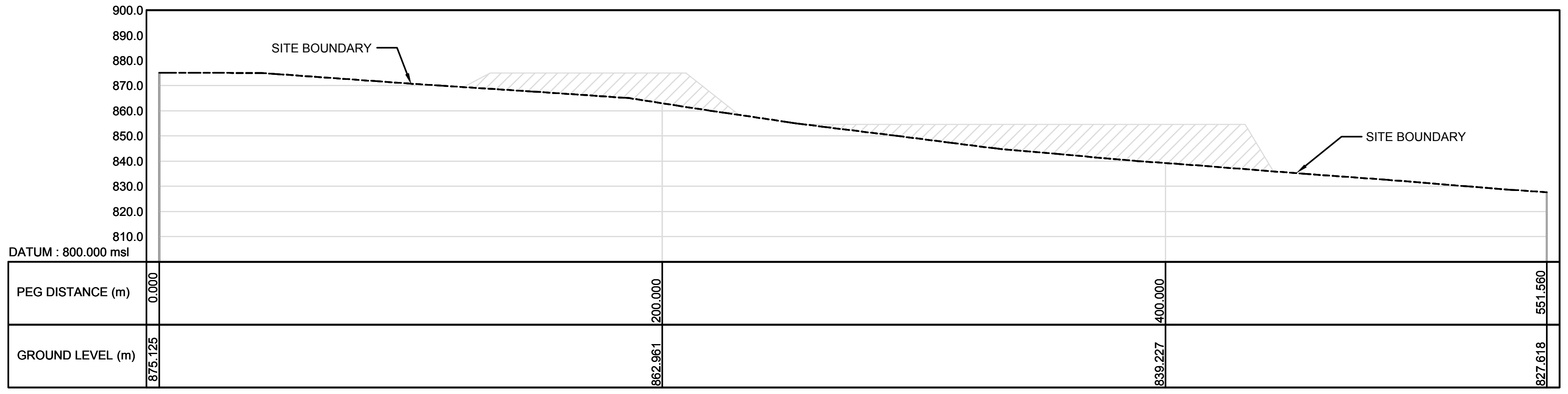

SECTION 4-L
FROM 0.000 TO 551.560
SCALE 1: 2500

Knight Piésold CONSULTING

PRIMARY DISCIPLINE	CIVIL	
DRAWN	K.G	OCT 2014
DRAWING CHECK	P.G	OCT 2014
DESIGN	P.G	OCT 2014
DESIGN CHECK	A.D	OCT 2014
PROJECT ENGINEER	A.D	OCT 2014

30300413/G10

REVISION

REV.No.	DATE	DESCRIPTION	DRAWN	CHKD.	APPD.
A	10/2014	WATER TREATMENT WORKS – –OPTION 4 : CROSS SECTIONS	K.G	A.D	

UMGENI WATER · AMANZI

REFERENCE DRAWINGS

DRAWING No.	MAKERS No.	TITLE

UMKHOMAZI WATER PROJECT

WATER TREATMENT WORKS OPTION 4
SITE M1 CROSS SECTIONS SHEET 2 OF 2

DRG. No. 30300413/G10 REV. A