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Rotoiti – Rotoma WWTP and Land Disposal System: Concept Design

Rotorua Lakes Council

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Rotoiti – Rotoma WWTP and Land Disposal System: Concept Design

Prepared for

Rotorua Lakes Council

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Limitations:

This report has been prepared on the basis of information provided by Rotorua Lakes Council, and others (not directly contracted by PDP for the work), including Mott MacDonald, AWT and Opus. PDP has not independently verified the provided information and has relied upon it being accurate and sufficient for use by PDP in preparing the report. PDP accepts no responsibility for errors or omissions in, or the currency or sufficiency of, the provided information.

This report has been prepared by PDP on the specific instructions of Rotorua Lakes Council for the limited purposes described in the report. PDP accepts no liability if the report is used for a different purpose or if it is used or relied on by any other person. Any such use or reliance will be solely at their own risk.

Executive Summary

Pattle Delamore Partners Ltd (PDP) has developed a conceptual design for a wastewater treatment plant and land disposal system to service Rotoiti and Rotoma. This treatment facility will utilise membrane bioreactor technology to produce a high quality effluent prior to disposal to rapid infiltration trenches.

The site of the proposed wastewater treatment and disposal facility is on the hillside behind Emery's Store on the southern edge of Lake Rotoiti. Due to possible future residential development in close proximity to the proposed site, the concept design has included provisions to minimise odour and noise at the treatment and disposal facility and to reduce the visual impact at the site.

The estimated capital cost of the wastewater treatment and land disposal facility is $8.6M. This cost estimate is exclusive of GST and includes a 30% contingency and 15% allowance for professional services. A sum of $400K has been included to cover further investigations and consenting of the scheme. The annual operating cost is $230K and the 40-year net present value of the facility is $13.5M.

The following recommendations are made to allow Rotorua Lakes Council to progress this project. Note that Items 1 to 3 should be undertaken to confirm the assumptions used in this concept design report prior to making the final decision to proceed with this project.

1. Collect additional geological information at the site of the proposed land disposal system and undertake infiltration tests;
2. Install groundwater monitoring bores to establish a groundwater baseline and undertake hydrogeological analysis;
3. Undertake an assessment of environmental effects and obtain regional council agreement for the concept design;
4. Develop preliminary design including receiving proposals from membrane equipment suppliers and update the capital and operating cost estimates which have been developed at this stage;
5. Prepare consent applications;
6. Throughout this process undertake stakeholder consultation.

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1.0 Introduction

1.1 Purpose of Report

The purpose of this report is to develop a concept design for a wastewater treatment plant (WWTP) and land disposal system (LDS) to service Rotoiti and Rotoma.

1.2 Scope of Work

The scope of work in preparing this report is outlined as follows:

- Develop a process train, layout and sizing of a membrane bioreactor (MBR) wastewater treatment plant, including recommending the most appropriate type of membrane;
- Recommend the most appropriate LDS to suit the physical characteristics of the site and develop sizing and layout;
- Recommend the most appropriate odour control for the WWTP;
- Develop a layout drawing and an artist's sketch of the WWTP and LDS;
- Develop indicative capital cost estimate for the WWTP and LDS (+/- 30% accuracy).

1.3 Background

Wastewater generated from the lakeside communities of Rotoiti and Rotoma is currently treated and disposed of using privately owned septic tanks and onsite disposal trenches. Rotorua Lakes Council (RLC) has investigated options for a sewerage scheme to service these communities in order to reduce the public health risk and environmental risk associated with the onsite septic tank and disposal systems.

The preferred option developed by RLC and key stakeholders is to collect and convey wastewater from the communities to a common WWTP located on an elevated site behind Emery's store in the vicinity of Gisborne Point on the southern edge of Lake Rotoiti as shown in Figure 2.

This concept design report builds upon previous work undertaken which is outlined in the following documents:

- AWT (2010). *Gisborne Point/Rotoma WWTP: Manawahe Road Site - Concept Feasibility (2010).* AWT Water Ltd, December 2010.
- MM (2014). *Rotoma MBR Options - Concept Cost Estimates (2014).* Mott MacDonald, November 2014.
- PDP (2014). *Rotoiti/Rotoma Sewerage Scheme - Peer Review of Cost Estimates.* Pattle Delamore Partners Ltd, July 2014.

- Opus (2015). *Lake Rotoiti Wastewater Scheme - Stage 1 Investigations.* Opus International Consultants, April 2015.

2.0 Design Criteria

2.1 Wastewater Collection System

The wastewater collection systems proposed at Rotoiti and Rotoma will utilise a low pressure sewerage scheme (LPSS) reticulation network. The LPSS scheme at each community will be slightly different as outlined as follows:

- ***Rotoma:*** Wastewater will be gravity fed from household pipework (laterals) to on-property grinder pump units (1 per property). The grinder unit will pump the raw wastewater into the LPSS.
- ***Rotoiti:*** Wastewater will be gravity fed from household pipework to an onsite Biolytix treatment system (1 per property) which will be retrofitted with a pump suitable for discharge into the LPSS. The Biolytix treatment systems provide removal of some wastewater contaminants as outlined in Section 2.2.

The LPSS will discharge to larger transfer pump stations which will pump the wastewater via rising mains to the common WWTP site behind Emery's store for treatment and disposal.

Further details of the preferred wastewater collection system are outlined in the report Rotoiti/Rotoma Sewerage Scheme - Peer Review of Cost Estimates (PDP, 2014). Concept design and cost estimates for the reticulation system are outside the scope of this report.

2.2 Flows and Loads

2.2.1 Flows

Influent flows to the proposed WWTP have been derived based on the design population that will be serviced by the proposed sewerage scheme.

PDP has utilised the total rateable household unit equivalent (HUE) data provided by RLC which allows for future development of existing lots and occupancy of existing vacant lots. The following average and peak design occupancy rates have been applied:

- Average: 2.0 persons/HUE;
- Peak: 4.0 persons/HUE.

A per person daily flow rate of 220 L/person/d has been used together with the above occupancy figures and the HUE data to calculate the design average daily flow (ADF) and peak daily flow (PDF). These design flow rates are outlined in Table 1.

Given that the Rotoiti/Rotoma sewerage system is pressurised from the on-property grinder units/Biolytix units to the WWTP, stormwater inflow and groundwater infiltration (I&I) into the network will be minor compared with a conventional gravity reticulation system. However, some allowance for I&I should still be made for flow into on-property gully traps, illegal roof connections and infiltration into on-site laterals and grinder/Biolytix units and transfer pump stations. A peak wet weather flow (PWWF) factor of 1.2 has been assumed for this report which is considered appropriate to cater for possible I&I sources upstream of the gully traps (GHD, 2014).

Table 1: Influent Flows

Location	HUE[1]	ADF[2]	PDF[3]	PWWF[4]
		m^3/d	m^3/d	m^3/d
Rotoma	344	151	303	363
Rotoiti	545	240	480	576
Total	**889**	**391**	**782**	**939**

Notes:

1. *Household unit equivalent data provided by RLC;*
2. *Average daily flows (ADF) have been derived using an occupancy of 2.0 persons/HUE and a per person flow rate of 220 L/p/d;*
3. *Peak daily flows (PDF) have been derived using an occupancy of 4.0 persons/HUE and a per person flow rate of 220 L/p/d;*
4. *A peak wet weather flow factor of 1.2 has been assumed.*

2.2.2 Loads

Influent contaminant concentrations have been adopted from Mott MacDonald (2014) with the exception of the phosphorus concentration which has been updated. Contaminant concentrations from Rotoiti are less than from Rotoma due to treatment provided by the on-property Biolytix treatment systems. Design loads have been calculated based on the design concentrations and the design flows outlined in Section 2.2.1 and are presented in Table 2.

The Rotoma contaminant concentrations reported by MM are considered to be reasonable as these are consistent with a medium to strong concentrations presented in Metcalf and Eddy (2003). Rotoiti contaminant concentrations reported by MM are generally consistent with results from the On-site Effluent Treatment National Testing Programme (OSET NTP) (Bay of Plenty Regional Council, 2013/2014). However, previous OSET NTP trialling (prior to 2010) of the Biolytix units has shown total phosphorus concentrations consistently higher than that reported by MM, hence, PDP has updated this design contaminant parameter.

Table 2: Influent Loads

Parameter[1]	Concentration (g/m^3)			Combined Ave. Daily Load	Combined Peak Daily Load
	Rotoma	Rotoiti	Combined	kg/d	kg/d
$cBOD_5$	235	7.6	96	37	75
TSS	294	11	121	47	94
TN	65	39	49	19	38
TP	7.7	7.0	7	2.8	5.7

Notes:

1. *$cBOD_5$ = Carbonaceous Biochemical Oxygen Demand, TSS= Total Suspended Solids, TN=Total Nitrogen, TP=Total phosphorus.*

The peak daily loads would increase in future if the Rotoiti Biolytix units were replaced with a conventional LPSS as these replacements would provide no on-site treatment. However, for the purpose of this report it has been assumed that the Biolytix units would be retained.

2.3 Effluent Quality

In the brief for this project RLC prescribed that influent to the WWTP is to be treated using membrane bioreactor (MBR) technology prior to disposal to land using a rapid infiltration method.

RLC has indicated that treatment requirements of the WWTP are for 92% removal of nitrogen and 73% removal of phosphorus. From PDP's experience a total nitrogen (TN) effluent concentration of 4.5 g/m^3 represents the limit that an MBR process can achieve with supplementary carbon dosing and without additional treatment (e.g. denitrifying filter), therefore, a nitrogen removal requirement of 91% has been assumed based on the influent data outlined in Table 2. Applying these removal rates indicates the target effluent quality as outlined in Table 3. Given the low influent carbon to nitrogen ratio (C:N approximately 2.0), supplementary carbon dosing will be required to achieve an effluent TN concentration of 4.5 g/m^3. It is also expected that alum dosing will be required to achieve the target effluent total phosphorus of 1.6 g/m^3.

Table 3: Target Effluent Quality

Parameter[1]	Influent		Removal Target	Effluent	
	Ave. Conc.	Ave. Load		Ave. Conc.[2]	Ave. Load
	g/m^3	kg/yr	%	g/m^3	kg/yr
$cBOD_5$	96	13,600	n/a	2	290
TSS	121	17,200	n/a	2	290
TN	49	7,000	91	4.5	640
TP	7.0	1,040	73	1.6	230

Notes:
1. *$cBOD_5$ = Carbonaceous Biochemical Oxygen Demand, TSS= Total Suspended Solids, TN=Total Nitrogen, TP=Total phosphorus;*
2. *It is assumed that supplementary carbon dosing will be required to achieve the target effluent quality, and that alum dosing will be required to achieve the target TP concentration.*

The effluent concentrations and loads outlined in Table 3 are considered to be average effluent 'targets' (e.g. average concentrations that the WWTP will reliably achieve). Therefore, appropriate Resource Consent median concentrations to allow some factor of safety for consent compliance would likely be 10/10/8/3 for $cBOD_5$/TSS/TN/TP respectively.

2.4 Land Disposal System

As per the brief from RLC, it has been assumed that the disposal system is to be designed to accommodate the hydraulic requirements only and not provide for residual 'land treatment' of the effluent. This type of system is commonly referred to as a rapid infiltration system (RIS), whereby rapid disposal of effluent is achieved in a small footprint.

RIS's are typically utilised where a high level of treatment is provided prior to land disposal, such as the chemical assisted MBR system proposed at this site. On this basis, the concept RIS outlined in this report has not been designed to comply with a specific nitrogen loading rate such as 150 kg/N/ha/yr which would be a permitted activity under Bay of Plenty Regional Council Rules for a 'land treatment system'.

2.5 Other Design Criteria

RLC has advised that future residential housing may surround the WWTP/LDS site. Accordingly, the following criteria have been taken into account when developing the concept for the WWTP and LDS:

1. ***Odour:*** provision has been made for collection and treatment of objectionable odour generated at the WWTP site;

2. ***Visual Impact:*** the design must minimise the visual impact of the WWTP and LDS;

3. ***Noise***: the design must minimise noise in order to comply with relevant acoustic standards for a residential setting;

4. ***Operation and Maintenance:*** while an MBR WWTP with chemical dosing is an advanced system which will require operator input, the design should minimise operator input and maintenance requirements where possible.

3.0 Wastewater Treatment Plant

Based on PDP's experience at other sites in New Zealand, an appropriately designed MBR as outlined in this report is expected to achieve the target effluent quality criteria outlined in Table 3. Detailed process design has not been undertaken to confirm this assumption and this would need to be undertaken as part of the next stage of this project.

The concept design which has been assumed for the Rotoiti/Rotoma WWTP is outlined in the following sections and a process flow diagram has been included as Figure 1.

3.1 Infrastructure Items

Infrastructure items for the WWTP are outlined as follows:

- Fine screening and grit removal unit (1/0.5mm aperture), such as a packaged Johnson Screen package (as installed at Maketu);
- A 4-stage Bardenpho biological nutrient removal (BNR) secondary wastewater treatment process, including:
  a) Stage-1: primary anoxic tank (including alum dosing for chemical precipitation of phosphorus and supplementary carbon dosing for enhanced denitrification);
  b) Stage-2: aeration tank (including caustic dosing for alkalinity adjustment);
  c) Stage-3: secondary anoxic tank;
  d) Stage-4: MBR tank (including recycle to Stage-1);
- Permeate storage tank, including permeate pumps and land disposal pumps;
- Emergency storage tank, including pumps;
- Secondary solids dewatering unit (including polymer dosing) and solids cake storage;

- Membrane cleaning system (including chemical storage);
- Control building including motor control centre (MCC), air blowers and chemical storage (in separate rooms);
- Foul air collection and treatment unit.

A simpler 2-tank Modified Ludzack Ettinger (MLE) BNR configuration could be adopted at Rotoiti/Rotoma as an alternative to the 4-tank Bardenpho configuration, however, the 4-tank system is the preferred option by PDP to reliably achieve the low effluent nutrient concentrations outlined in Table 3.

3.2 Indicative Tank Sizing

Indicative sizing of the key process tanks has been undertaken by applying conservative hydraulic retention times as summarised in Table 4. Detailed process design will be required to confirm tank sizing. For this concept design a maximum tank height of 5 m has been assumed.

Table 4: Process Sizing

Tank	HRT[1]	Volume[2]	Plan Area[3]
	(h)	m^3	m^2
Stage1: Primary Anoxic Zone	3	96	19
Stage 2: Aeration Zone	3	96	19
Stage 3: Secondary Anoxic Zone	3	96	19
Stage 4: MBR Tank	1.5	48	19
Total	**11**	**352**	**70**

Notes:
1. *Hydraulic retention times utilised for tank sizing are based on the PDF;*
2. *Tank volumes have been estimated based on design influent flows and typical hydraulic retention times;*
3. *Plan areas have assumed a tank depth of 5 m apart from the membrane tank which is assumed to be 2.5 m.*

3.3 Membrane Selection

Selection of a preferred membrane supplier has considered existing Flat Sheet and Hollow Fibre MBR systems in operation at Rotorua WWTP and at Taupo District Council's Turangi WWTP. Brief descriptions of these systems together with operational issues which have been reported are outlined in the following sections.

3.3.1 Rotorua MBR

PDP has discussed the operation of the Rotorua WWTP with RLC Plant Supervisor Andy Bainbridge in August 2015. The Rotorua MBR was constructed in 2012 and uses GE hollow fibre membranes. Permeate is drawn from the membranes using a pumped system. Since installation, some of the valves originally supplied with the MBR unit have been replaced and/or upgraded. However, the operator has been generally ‘happy’ with the performance of the membranes since installation and the support provided by GE technical staff in Australia.

3.3.2 Turangi MBR

PDP has discussed the operation of the Turangi WWTP with Taupo District Council (TDC) operations personnel in August 2015. The Turangi MBR was constructed in 2006 which used Kubota Flat Sheet Membranes in an EK400 module consisting of two membrane cases which stack on top of one another. Permeate is drawn from the MBR under gravity flow (no permeate pump). Fouling was an ongoing issue and was attributed to inadequate air scour from the underlying coarse bubble diffusers.

In 2014, the Turangi WWTP changed to SINAP Flat Sheet membranes. Issues with fouling have since improved significantly and the operators have described the new membranes as ‘very good’. Recently, *E. coli* breakthrough has been observed and it is suspected that this is due to cracking of the plastic manifold which collects the permeate. Investigations have been undertaken to find the suspect cracking but these investigations have not been successful to date. The WWTP as a whole is still providing high levels of nitrogen, suspended solids and BOD removal.

3.3.3 Preferred Membrane Selection

Given that RLC has had experience with the GE membranes at the Rotorua WWTP, and to provide operational and maintenance consistency, GE membranes are preferred for the Rotoiti/Rotoma WWTP. Preliminary discussions with GE indicate that these units may also provide cost savings compared to SINAP membranes.

However, prudent decision making would suggest that further more detailed discussions should be held with operations personnel at RLC and TDC to more fully investigate the pros and cons of each supplier and formal proposals should be called for from GE and SINAP and evaluated in the detailed design stage to confirm membrane selection.

3.3.4 Membrane Design Considerations

Fine screening and grit removal is important for protection of the membranes against physical damage and build-up of lint and hair in the process tanks which can foul the membranes and coarse bubble diffusers.

The screen and grit removal system will be designed to handle the unusual volcanic pumice grit at Rotoiti/Rotoma which will be a mixture of sinking and floating grit.

Design of the coarse bubble aeration system should aim to provide adequate air flow and uniform air density to prolong the life of the membranes.

Provision of a de-aeration zone on the MBR recycle prior to the 'Stage 1 Primary Anoxic Zone' may also be required to ensure that anoxic conditions can be maintained in order to provide a high level of nitrogen removal.

3.4 Peak Flow Management

An appropriately designed pressure sewer reticulation system will convey a much lower peak flow rate to the Rotoiti and Rotoma WWTP than a conventional gravity reticulation system. This is a result of minimal inflows and infiltration associated during wet weather (as outlined in Section 2.2.1), and due to the storage at each property utilised to buffer peak diurnal flows.

Given that all reticulation in Rotoiti and Rotoma is to be pressure reticulation, peak instantaneous flows will much more balanced than conventional gravity reticulation, therefore, the need for additional flow balancing at the WWTP will be minimal.

A dedicated influent balancing tank will be a significant source of odour and would require extensive corrosion protection, and as the instantaneous peaking factor is likely to be minor, it is assumed that the influent balancing will not be required, and membrane sizing should be based on the PWWF.

3.5 Solids Management

Preliminary calculations indicate that the maximum volume of secondary sludge (waste activated sludge) that will need to be removed from the WWTP will be approximately 5 m^3/d (at approximately 1% dry solids). This low concentrated sludge will be pumped to the dewatering building where it will be dewatered using a screw press or a similar unit to achieve a dry solids concentration of 20 to 25% with polymer dosing (to aid flocculation). The supernatant will be returned to Stage 1 of the MBR process stream and the dewatered solids will be stored on-site in a skip prior to off-site disposal to landfill or to composting / vermicomposting together with solids from the Rotorua WWTP. Screenings separated at the inlet works to the WWTP would be stored in a wheelie bin and periodically transported back to the Rotorua WWTP for disposal together with screenings from this facility.

Alternatively, liquid solids could be transported back to the Rotorua WWTP for thickening and dewatering at this facility, however this would likely require 1 tanker truck every 2 to 3 days during peak periods which is considered to be excessive and has not been considered further.

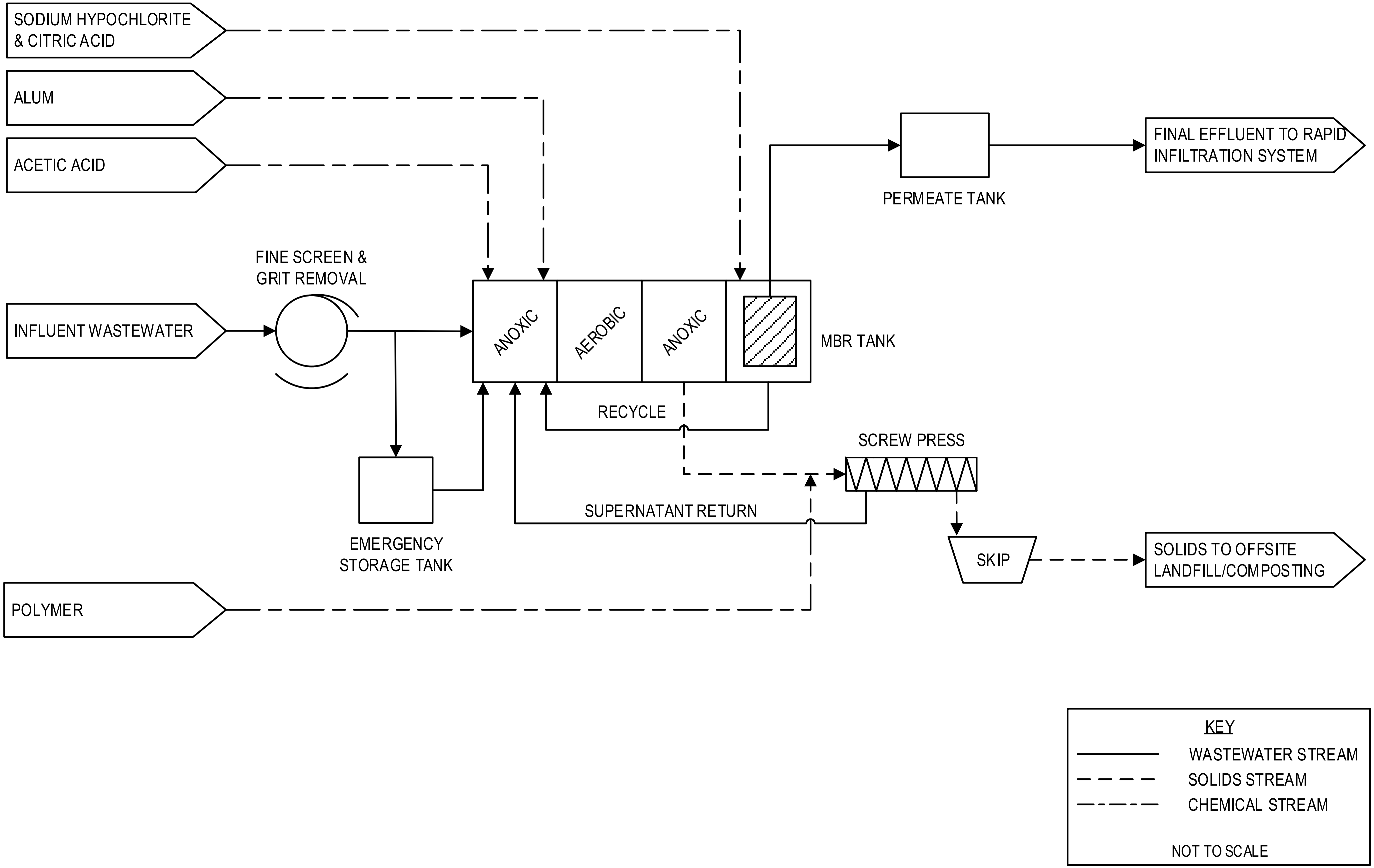


FIGURE 1: PROCESS FLOW DIAGRAM

3.6 Odour Management

The WWTP will have several sources of odour which will require careful consideration to ensure there is no odour generated at the site. This is particularly important given that residential housing may be in close proximity to the WWTP and LDS site in the future.

Odour generated from domestic wastewater can largely be attributed to hydrogen sulphide gas (H_2S) released under anaerobic conditions. The key WWTP locations where odour management is required include:

- Influent screening/grit removal;
- Dewatering and storage of secondary solids.

Given that the reticulation system is to comprise of a LPSS followed by a rising main discharging to the WWTP, the retention time in the network is likely to be significant and high concentrations of H_2S will likely be released at the head of the WWTP (Inlet screen). For this reason, enclosure of the inlet screen inside a building will be required in order to minimise odour release, with foul air extraction and treatment.

The concept design has assumed that secondary solids dewatering will be located inside the same building as the influent screening/grit removal system, with air extraction at a rate of 12 air changes per hour which would be discharged into an appropriately designed bark-bed biofilter or chemical wet-scrubber. At this concept design stage it has been assumed that foul air treatment would utilise a chemical scrubber (packed bed tower) with a recirculated chemical solution to transfer the H_2S from the gas phase to the liquid phase, with chemical oxidation of H_2S and discharge of oxidised sulphur constituents back into the WWTP. The costs for a wet-scrubber will be similar to a biofilter for the Rotoiti/Rotoma site, and advantages and disadvantages of these options can be further explored during detailed design.

It is not expected that odour control will be required for any of the main process tanks (anoxic, aerobic/MBR) provided that suitable retention times and dissolved oxygen levels are selected during detailed design. However, should the site experience a power failure, partially treated wastewater has the potential to turn anaerobic within a few hours, thereby resulting in fugitive odour discharge (as well as significant loss of treatment performance). Therefore, it is assumed that an emergency generator will be required to ensure the WWTP will continue to operate normally in the event of power failure.

Alternatively, the entire WWTP could be contained within a building, however, this is not considered necessary and has not been costed for at this stage.

3.7 Noise Management

Noise will be generated by mechanical equipment at the WWTP site. The most significant sources of noise will be required to be contained within on-site buildings. Sources of noise at the WWTP and controls that should be implemented are outlined as follows:

- ***Blowers:*** house inside a blower room with acoustic silencers on the inlet and specialised building design;
- ***Pumps:*** dry mounted pumps may require an acoustic housing while wet well pumps will not require noise control;
- ***Dewatering equipment:*** house indoors and consider specialised building design;
- ***Fans:*** will likely require acoustic enclosures.

Acoustic assessment of all sources of noise will be required during the detailed design phase to ensure compliance with RLC's Noise Limits for residential zoning.

3.8 Chemical/HSNO Requirements

Several chemicals will be used at the WWTP. In order of expected use/cost these are; ethanol, alum, caustic, sodium hypochlorite and citric acid.

The concept design has assumed that ethanol will be added to enhance denitrification. Ethanol is cost effective when compared to other carbon sources such as acetic acid, however, ethanol requires more stringent health and safety requirements.

A number of HSNO controls will be required at the site for the storage and use of these chemicals, such as the following:

- Secondary containment for the storage of all chemicals. Separation of incompatible chemicals;
- The tanker delivery area will provide for containment of 110% of the largest tanker compartment;
- HSNO signage at the site entry and at the chemical storage/dosing area;
- Emergency showers/eye wash stations;
- Fire extinguishers;
- Safety data sheets and chemical inventories;
- A level 3 emergency management plan;
- Stationary container test certificates;
- Approved handler documentation.

3.9 General Civil Works

A concept layout plan of the WWTP, LDS and access road is shown in Figure 1. Indicative sketches are shown in Figures 3, 4 and 5.

3.9.1 Access Road

Significant civil works will be required to construct an access road to the WWTP site. RLC has identified one potential route, which based on a site visit by PDP in July 2015, appears to be the most practical road alignment.

The route follows an incised valley at a grade of approximately 10% for the first 300 m before flattening out (to around 7% grade) for another 400 m. It is expected that significant earthworks will be required to raise the floor of the existing valley, using soil borrowed from elsewhere on the site, to minimise the need to cut into the steep hillsides. Stormwater drainage will be required to cater for overland flow through the existing valley floor as well as runoff from the new road which will likely need to be sealed to prevent erosion of the road surface.

Additional investigations are required to confirm the most practical alignment for the access road and to obtain geotechnical information to develop a design for the road construction.

3.9.2 WWTP Compound

As previously described, certain unit processes at the WWTP will be contained inside buildings to prevent noise and odour issues. Concrete and steel tanks will be situated adjacent to the buildings and an access road will enable vehicle access to the site. No tanks or buildings will be more than 5 m high. There will be a security fence around the entire WWTP compound which will be approximately 0.3 ha in area (55 m by 55 m). The compound will be screen planted to prevent direct visual contact and to soften the overall impact of the facility.

3.9.3 Power Supply

A power supply will be required at the site and it is assumed that overhead 11 kV power lines will be required to be installed approximately 600 m from State Highway 30 to the WWTP site. It is estimated that a 100 to 150 kVA transformer will be required at the WWTP site. An emergency generator will also be required to cater for power outages.

3.9.4 Water Supply

A water supply will be required for general amenities (toilet, shower and basin) as well as for wash-down activities within the WWTP. It is assumed that roof water collection can be used for general amenities and it is assumed that permeate (final effluent) can be used for WWTP wash-down activities.

3.9.5 Security Fences

The WWTP will need to be surrounded by a chain-link security fence to exclude general access from the public. Security cameras are also recommended to deter theft and vandalism. It is assumed that stock fencing will be adequate to surround the LDS.

3.10 Procurement

Procurement options for the WWTP include the following:

- ***Traditional Design then Construct:*** This requires tendering the design followed by selection of a main Contractor. During the design process the membrane equipment would be selected and either procured by the Principal or by the Contractor.
- ***Design/Construct or Design/Construct/Operate***: This involves a Contractor designed system, with inclusion of an operating period as part of the Contract works preferred to ensure satisfactory performance for a prescribed time period.
- ***'Packaged' WWTP****:* This involves an off-the-self system which can be provided by some suppliers such as Filtec or Hynds Environmental. This involves the majority of process equipment supplied as packaged system and the civil and other mechanical and electrical items designed around the supplier proposal. This option is typically only available and recommended up to a flow of approximately 200 m^3/d.

In all cases, a comprehensive specification must be prepared by the Principal (or their representative) to ensure that all equipment is fit for purpose so that design requirements can be achieved.

Given the size of this project, the environmental sensitivity and the constraints with regard to odour, noise and aesthetics due to the close proximity of future residential areas, PDP considers that a traditional Design and Construct procurement approach is preferred over the other options as it will provide greater control and ability to achieve the desired outcomes.

4.0 Land Disposal System

4.1 Geological Information Review

Limited hydrogeological data was obtained during preliminary field investigations carried out in March 2015 by Opus International Consultants in the vicinity of the WWTP site. Soil descriptions were produced based on observations from 4 No. hand auger investigations (to approximately 4 m BGL) and one borehole drilled to 18.5 m BGL.

4.1.1 Stratigraphy

The deeper borehole investigation indicates predominance of sands and gravels to a depth of approximately 15 m BGL, with rhyolite rock encountered below this depth. Two layers of silty sand were encountered at about 2 and 11 m BGL, respectively. These layers were around 1 m thick and could potentially act to impede some vertical drainage. No record of perched groundwater was noted in the results of the preliminary field investigations.

The upper silt/ash layer was also encountered in the shallow investigations, albeit at depths ranging from 0.6 to 1.7 m BGL. Other sediments encountered in the profile included sands and gravels, consistent with findings of the borehole investigation.

The 1:250,000 scale geological map of the Rotorua area (GNS, 2010) shows that the site is underlain by Pleistocene rhyolite lava deposits of the Te Rere Formation within the Okataina Group volcanics.

4.1.2 Hydrogeology

It is expected that the site overlies an unconfined aquifer hosted within the Te Rere Formation rhyolite. The aquifer is recharged through infiltration of rainfall from the land surface and vertical percolation via the unsaturated zone. Groundwater flow is likely to be in a northerly direction following the general land contour and discharging into Lake Rotoiti.

The depth of the local water table was not encountered during the borehole investigation which was undertaken in summer, indicating that at this time the water table is situated at some depth below 18.5 m BGL (at the location of BH1). It is possible, although unlikely, that the water table may be at a depth closer to the surface than observed in March. The elevation of Lake Rotoiti water level is approximately 280 m ASL, compared to elevation around the area of the proposed LDS of 355 to 365 m ASL. It is not uncommon in rhyolite geology for groundwater gradients to be low due to relatively high permeability. Therefore, groundwater levels at the proposed LDS site would likely be no more than 65 to 85 m BGL, but probably deeper than 20 m BGL. The depth to groundwater would need to be checked and confirmed prior to proceeding with the project.

4.1.3 Hydraulic Characteristics

A range of methods and test locations/depths were selected for permeability testing during preliminary site investigations and are presented in Opus (2015). A summary of these results is provided in Table 5.

Table 5: Permeability Test Results

Location	ID	Depth	Method	Strata	K (m/s)
BH1	Piezo 1	3 - 9 m	Falling head in piezometer	Sand/Gravel	2.2×10^{-3}
BH1	Piezo 2	12 - 18 m	Falling head in piezometer	Sand/Rock	1.5×10^{-2}
HAS1	Auger hole 1	0.73 m	In situ constant head	Fine sand	1.4×10^{-5}
HAS2	Auger hole 2	0.7 m	In situ constant head	Medium sand	5.3×10^{-6}
BH1	Push tube sample 1	4.5 - 5 m	Lab constant head	Sand with some gravel and minor silt/clay	3.6×10^{-8}
BH1	Push tube sample 2	12 - 12.5 m	Lab constant head	Clay/silt with minor sand	9.5×10^{-8}

Notes:
1. *Data summary from preliminary ground investigations undertaken by Opus (2015).*

The large variation in hydraulic conductivity results is reflective of both the range of sediments encountered and the method by which the tests were conducted. Laboratory tests on small push tube samples tend to result in lower conductivity values when finer sediments are targeted. In contrast, in situ tests performed over larger surface areas better represent natural heterogeneity in sediment textures and tend to result in higher conductivity values. Larger scale tests representing bulk hydraulic properties are considered more analogous to the expected performance of an LDS and should be undertaken as part of the next stage of work.

For the purposes of the LDS concept design outlined in this report, a bulk hydraulic conductivity of 1 m/d (1.16×10^{-5} m/s) has been provisionally assumed. There is not sufficient information presently available to accurately size the trench system and sizing can only be accurately undertaken once a partial scale infiltration test and hydrogeological analysis is undertaken.

4.2 Hydrogeological Risks

The key hydrogeological risks surrounding the operation of a rapid infiltration system are:

- Insufficient infiltration rate to accommodate the peak design flow. The concept design will require refinement following additional field investigations.

- Localised daylighting of disposed effluent could occur due to interface flow along top of low permeability layer. This would depend on the layer distribution and outcropping down gradient of the LDS. Additional investigation boreholes would improve the understanding of sediment distribution and the likelihood of this risk.
- Localised nutrient loading to lake shore (near shore effects) will require analysis and may require dispersion/mixing analysis to assess rainfall recharge/dilution of the plume as it migrates towards Lake Rotoiti.

To reduce the risks from geological and hydrogeological aspects further site investigative work is required as outlined in Section 5.0.

4.3 LDS Concept Design

4.3.1 Design Assumptions

The LDS is designed on a hydraulic load only. Little to no treatment of the effluent is expected to be required following disposal given the highly treated nature of the wastewater. However, this assumption will need to be carefully considered from an environmental effects perspective and should be discussed with the Regional Council.

The proposed LDS is situated on land with a fall of approximately 7%. Given the significant slope, and due to aesthetic and technical considerations, a trenched RIS has been adopted as the preferred concept design. A basin system could not be constructed without substantial earthworks.

The following design assumptions have been made to allow indicative sizing and siting to be undertaken. It needs to be noted that these parameters may need to be altered depending on the results of more detailed site investigations which must be undertaken prior to progressing this project further.

- PWWF (design flow): 940 m^3/d;
- Trench width/depth: 2.0/1.5 m;
- Trench spacing: 5.0 m or greater;
- Assumed infiltration rate: 1 m/day. Further investigations are required to confirm this (refer Section 5.0).

4.3.2 Trench Sizing, Siting and Design

A total trench length of 470 m is required to cater for the peak wet weather flow. This will consist of a minimum of 3 No. 160 m long trenches spanning the full width of the available land disposal site as shown in Figure 2. The total footprint of the LDS based on a 20 m spacing will be approximately 1 ha (56 m by 180 m). Given the uncertainty around the infiltration rate and the number of disposal trenches that will be required to manage the design flowrate, at this stage a land area of 2.0 ha is assumed for the LDS. A stock proof fence is recommended around the full perimeter of the LDS for the purpose of demarcation.

Utilising the full width of the disposal site will provide a greater spread of the resulting plume as it travels and enters Lake Rotoiti and aims to minimise environmental impacts and any near-shore effects at the lake edge.

There are a number of possible RIS design options in use around New Zealand including:

- In ground trench with timber retaining and removable covers (no backfill) to allow for access for maintenance purposes (e.g. Thames Coromandel District Council's Pauanui RIS);
- In-ground trench backfilled with gravel media and covered with geotextile and reinstated with natural soil (Taupo District has 5 No. such RIS's including Kinloch).

Given the low solids concentration achieved with the proposed WWTP, the lower cost backfilled option has been assumed to be appropriate at this stage. Key design features of this system are outlined as follows:

- The trench shall follow the natural contours of the land;
- The trench shall be divided into sections, each approximately 20 m long, which can be isolated from adjacent sections for maintenance purposes (i.e. an effluent distribution manifold with inlets to each 20 m long trench section);
- The manifold inlets shall enter a slotted drainage pipe laid at a common elevation along the length of each trench section.

The trench will be lined with a geofabric (with the exception of the base of the trench which will be unlined) to prevent migration of fines into the trench and backfilled with a clean aggregate to allow dispersion of the effluent prior to infiltration into the underlying soils. The final surface of the trench can be grassed and reinstated to match the existing surface providing a low-impact visual design and to transport stormwater over the top of the RIS.

4.3.3 Operation and Maintenance

Drying (or resting) periods are recommended between applications to reaerate the soil and allow residual organic matter to aerobically degrade and minimise the risk of fouling of the disposal trenches. This can be achieved by cycling the discharge to each section of the disposal trenches, either by manually opening/closing valves on the discharge manifold, or by utilising actuated valves on timer control. At this stage it has been assumed that the preferred option is for an automated system utilising hydraulically controlled actuated valves in order to minimise operator input.

Wye joints included at regular intervals along the slotted drainage pipe should also be provided for high pressure jetting of the disposal manifold.

4.3.4 Emergency Storage Tank

It has been assumed that a 200 m^3 emergency storage tank would be included to provide emergency storage of untreated during day-to-day operation or of poorly treated wastewater in the event of operational issues at the site during commissioning. This tank has been sized to provide 12 hours emergency storage at ADF.

This tank would remain empty and would only be used in the event of an emergency to prevent untreated wastewater from being discharged to the RIS which could lead to lasting operational issues at the site.

4.4 Environmental Effects

The proposed WWTP and LDS concept design has been developed on the basis that the system will provide an overall reduction in nutrient discharges to Lake Rotoiti and Rotoma. By replacing the existing septic tank based systems and providing a high level of nutrient removal via the WWTP there will likely be a reduction in the overall nutrient loading although this nutrient loading will now be localised into Lake Rotoiti.

5.0 Further Investigations

5.1 Groundwater, Geology and Hydrogeology

5.1.1 Groundwater Monitoring

It is recommended that groundwater monitoring bores are installed to collect baseline groundwater quality and level information so this can be used to confirm the concept design. The monitoring bores will require installation of piezometers (50 mm PVC casing and screens) to depths sufficient to encounter the regional water table. A minimum 5 m additional depth below the summer water table should be provided to allow for any natural groundwater level variation.

The following is recommended:

- Install 1 No. up gradient bores, as far up gradient as practical from the LDS (i.e. at the break in slope);
- Install 2 or 3 No. down gradient bores, at different locations to enable groundwater gradient evaluation;
- Install 1 No. bores in the centre or down gradient edge of the LDS;
- Install 2 No. bores at the lake edge.

A recommended monitoring bore layout is shown in Figure 1. Sediments encountered during drilling should be logged by a suitably qualified professional.

5.1.2 Geological Information

Additional test pits are recommended to be excavated to at least 1.5 m deep and preferably to 4 m deep along the intended LDS trench alignments to confirm the concept design. These will provide further information regarding sediment texture distribution at the intended depth of infiltration.

To provide infill geological data, it is recommended that additional geological boreholes are drilled to a depth of at least 15 m and logged by a suitably qualified professional. These boreholes should be situated in the vicinity of the proposed LDS and in areas not already covered by a monitoring bore. These bores can be drilled in the same campaign as the monitoring bores and would not require a permanent casing to be installed.

To provide infill geological information, it may be of benefit to undertake a geophysical survey of the site using ground penetrating radar (GPR) or similar. The results of this would be able to be correlated against geological logs from piezometer drilling. The aim of a GPR survey would be to characterise the lateral continuity of lower permeability layers potentially impeding vertical drainage or creating a perched zone of saturation and associated lateral flow.

5.1.3 Infiltration Testing

Infiltration tests are recommended to be carried out in all test pits using double ring infiltration tests. Larger scale infiltration tests should also be considered in selected test pits. These tests should be undertaken to confirm the concept design.

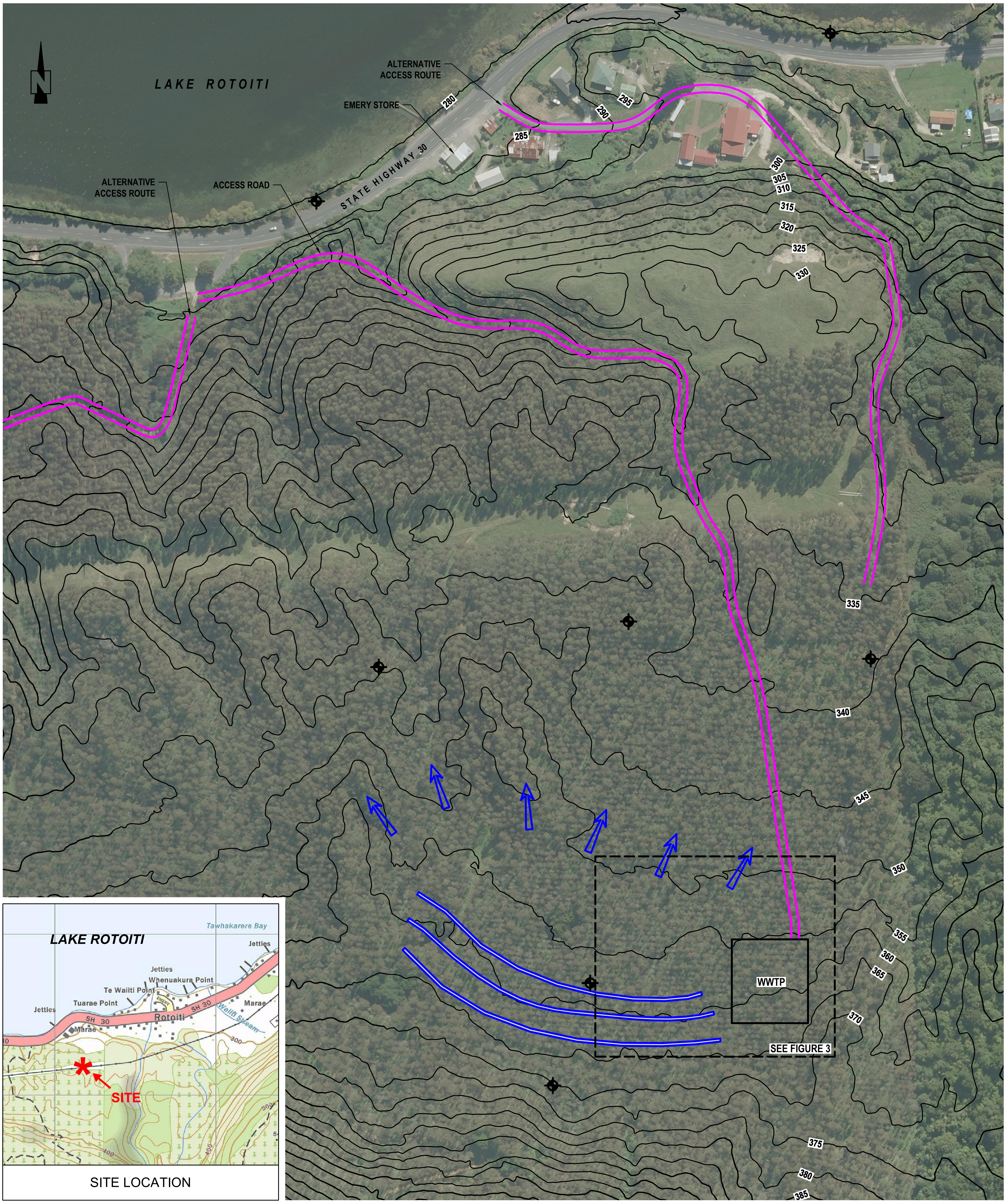
5.1.4 Hydrogeological Analysis

Monitoring and sampling of the groundwater monitoring bores will establish groundwater levels, gradient, flow direction, baseline groundwater quality and seasonal variation. This work will need to be undertaken prior to progressing further with the project.

The monitoring bores could subsequently be used for monitor groundwater quality once the LDS is operational.

5.2 Assessment of Environmental Effects

Further work is required to assess the localised environmental effects in the vicinity of the proposed WWTP and LDS. This assessment will utilise findings from the hydrogeological analysis outlined in section 5.1 and flow and effluent quality data outlined in Section 2.0. An assessment of environmental effects (AEE) including nutrient effects on the Lake Rotoiti foreshore area will be required as part of the discharge consent application.



SITE LOCATION

KEY :

- EXPECTED DIRECTION OF GROUNDWATER FLOW
- RAPID INFILTRATION TRENCH
- GROUNDWATER MONITORING BORE
- 285 EXISTING CONTOUR

SOURCE:
1. AERIAL IMAGERY & LiDAR DATA (2011) SUPPLIED BY BOPLASS LTD, DOWNLOADED FROM RCC GIS VIEWER AND LICENSED BY BOPLASS LTD FOR RE-USE UNDER THE CREATIVE COMMONS ATTRIBUTION 3.0 NEW ZEALAND LICENCE.
2. INSET DERIVED FROM LINZ DATA.

NOTES :

NO.	REVISION	DATE	APP.
A	ISSUED FOR REVIEW	AUG 15	

CLIENT :

ROTORUA LAKES COUNCIL
Te kaunihera o ngā roto o Rotorua

PROJECT :

ROTOITI/ROTOMA WWTP AND LDS CONCEPT DESIGN

TITLE :

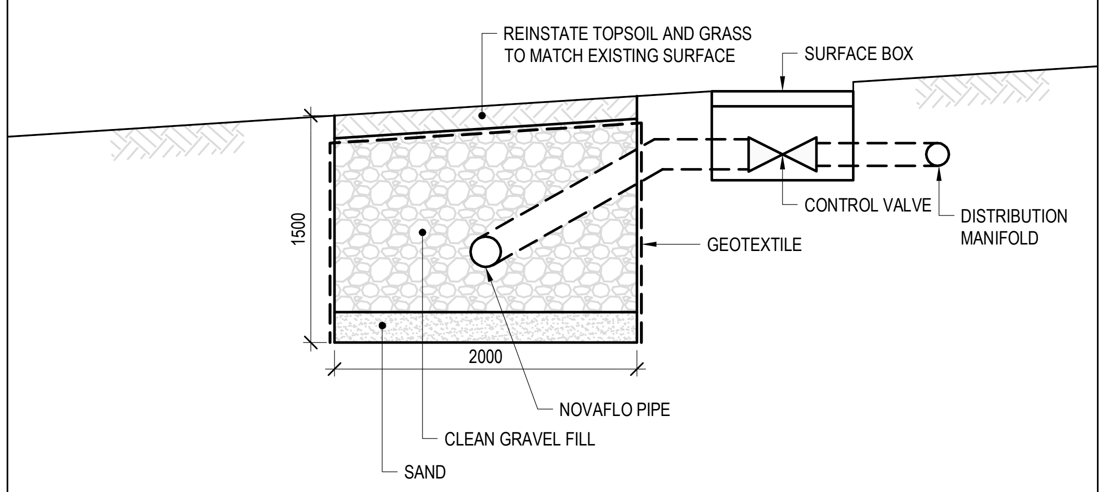
CONCEPT LAYOUT PLAN

pdp solutions for your environment
PATTLE DELAMORE PARTNERS LTD
Auckland **Tauranga** Wellington Christchurch

SCALE 1:2,500 (A3)
0 50 100m

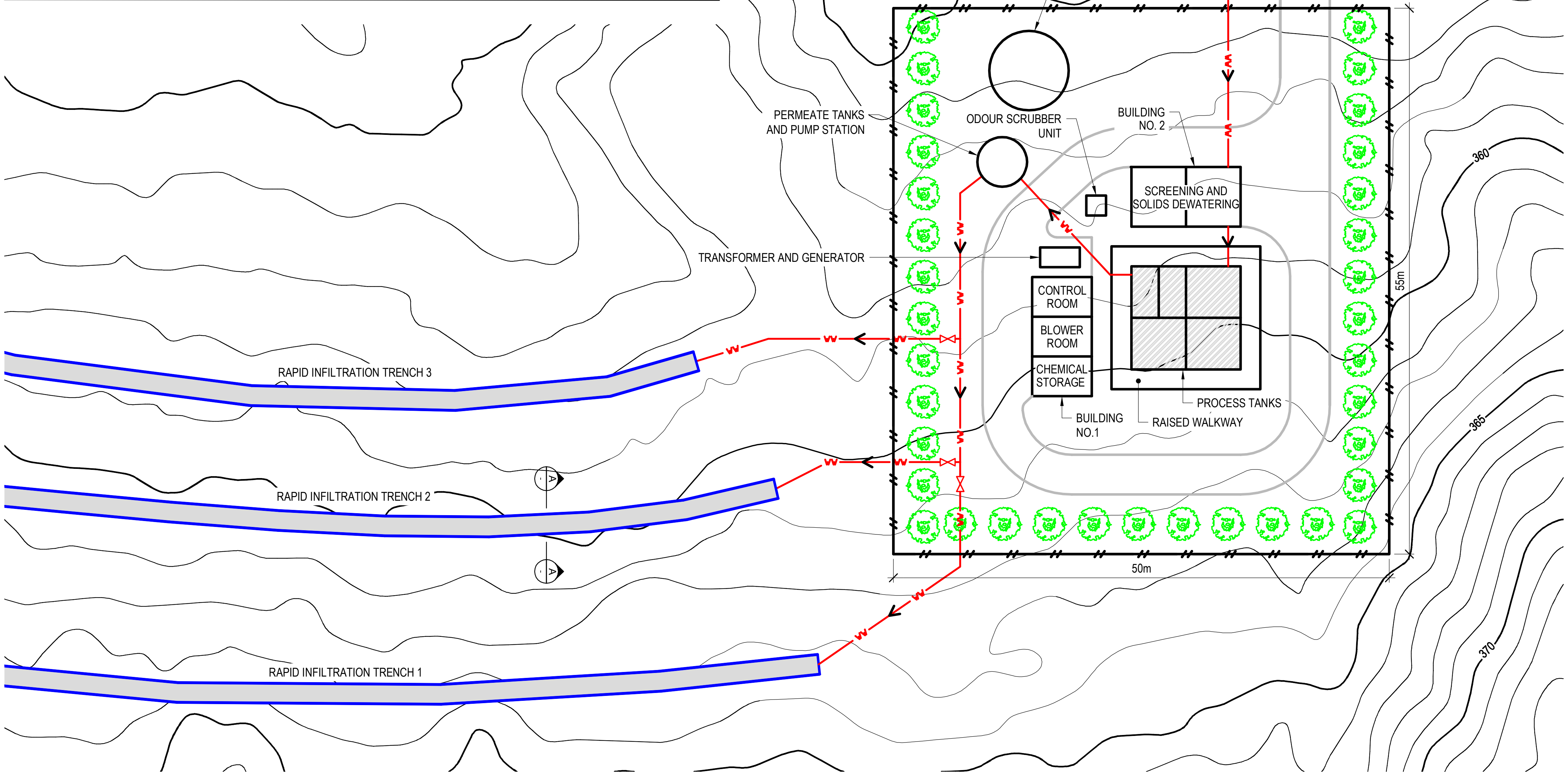
PROJECT NO. :	FIGURE NO. :	REVISION :
T01548203	2	A

FILED : T01548203D001_Rev A.dwg



SECTION A -

RAPID INFILTRATION TRENCH TYPICAL CROSS SECTION
SCALE N.T.S.



KEY :

- VALVE
- RAPID INFILTRATION TRENCH
- TREES
- PIPELINE
- SECURITY FENCE
- 355 EXISTING CONTOUR

NOTES:
1. ALL DIMENSIONS ARE IN mm UNLESS OTHERWISE SPECIFIED.

SOURCE:
1. LiDAR DATA (2011) SUPPLIED BY BOPLASS LTD, DOWNLOADED FROM RCC GIS VIEWER AND LICENSED BY BOPLASS LTD FOR RE-USE UNDER THE CREATIVE COMMONS ATTRIBUTION 3.0 NEW ZEALAND LICENCE.

NO.	REVISION	DATE	APP.
A	ISSUED FOR REVIEW	AUG 15	

CLIENT :
ROTORUA LAKES COUNCIL
Te kaunihera o ngā roto o Rotorua

PROJECT :
ROTOITI/ROTOMA WWTP AND LDS CONCEPT DESIGN

TITLE :
CONCEPT LAYOUT DETAIL PLAN

pdp solutions for your environment
PATTLE DELAMORE PARTNERS LTD
Auckland **Tauranga** Wellington Christchurch

SCALE 1:500 (A3)
0 10 20m

PROJECT NO. :	FIGURE NO. :	REVISION :
T01548203	3	A

FILED : T01548203D001_Rev A.dwg

KEY :

NO.	REVISION	DATE	APP.
A	ISSUED FOR REVIEW	AUG 15	

CLIENT :

ROTORUA LAKES COUNCIL
Te kaunihera o ngā roto o Rotorua

PROJECT :

ROTOITI/ROTOMA WWTP AND LDS CONCEPT DESIGN

TITLE :

PERSPECTIVE VIEW OF WWTP AND LDS

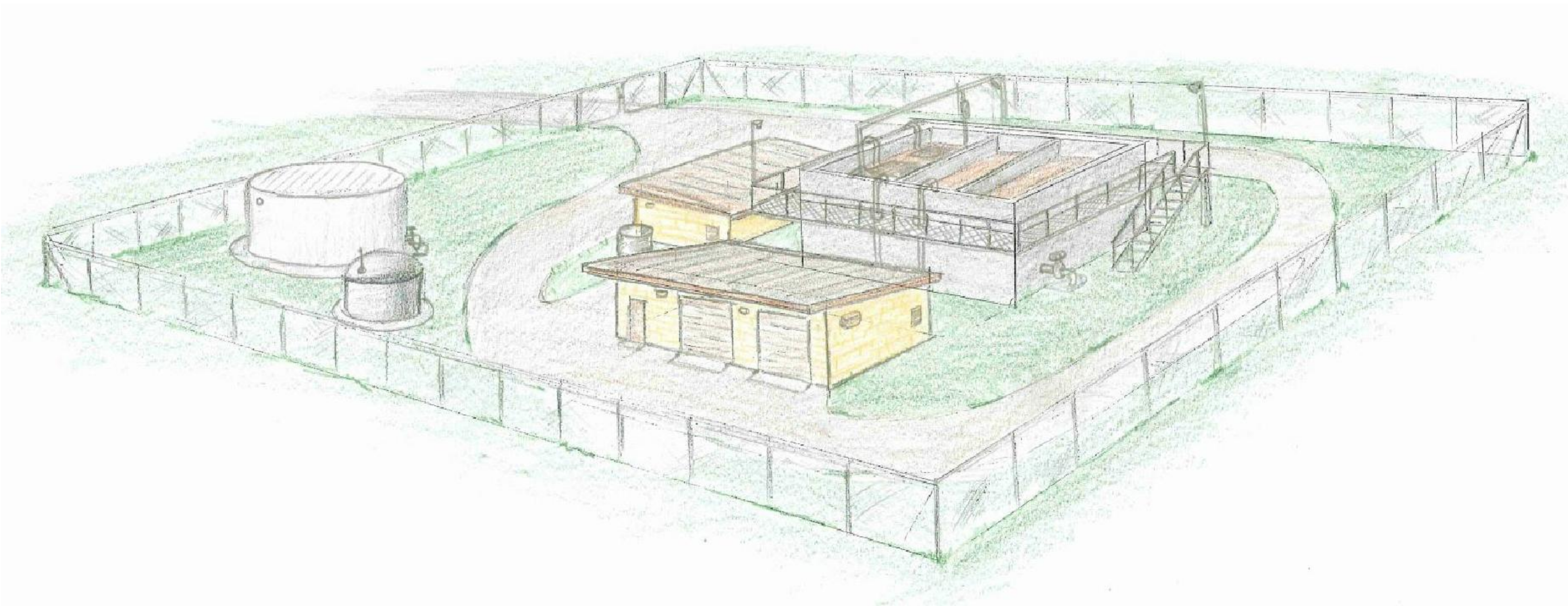
pdp solutions for your environment

PATTLE DELAMORE PARTNERS LTD
Auckland Tauranga Wellington Christchurch

SCALE NTS (A3)

PROJECT NO. :	FIGURE NO. :	REVISION :
T01548203	4	A

FILED : T01548203D001_Rev A.dwg



KEY :

NOTES:

THIS SKETCH SHOWS:

1. TANKS UP TO A MAXIMUM OF 5m HIGH.
2. A SECURITY FENCE 2.5m HIGH.
3. SCREEN PLANTING AROUND THE FENCE PERIMETER IS OMITTED FOR CLARITY.

A	ISSUED FOR REVIEW	AUG 15	
NO.	REVISION	DATE	APP.

CLIENT : ROTORUA LAKES COUNCIL Te kaunihera o ngā roto o Rotorua

PROJECT : ROTOITI/ROTOMA WWTP AND LDS CONCEPT DESIGN

TITLE : ARTIST SKETCH OF WWTP

pdp solutions for your environment

PATTLE DELAMORE PARTNERS LTD

Auckland Tauranga Wellington Christchurch

SCALE NTS (A3)

PROJECT NO. :	FIGURE NO. :	REVISION :
T01548203	5	A

FILED : T01548203D001_Rev A.dwg

6.0 Cost Estimates

PDP has developed 'concept level' capital, operating and 40-year NPV cost estimates for the proposed wastewater treatment and disposal system. All costs are in NZD and are exclusive of GST.

The estimated capital cost of the WWTP and LDS is $8.6M which includes a 30% contingency and 15% allowance for professional services. No allowance has been made for RLC financial and legal costs or for land purchase/lease. A sum of $400K has been included to cover further investigations and consenting of the scheme.

A summary of the capital cost estimates is outlined in Table 6 and a breakdown of these cost estimates is included as Appendix A.

Table 6: Estimated Capital Costs

Item	Cost Estimate[1,2]
Wastewater Treatment Plant	$4.6M
Land Disposal System	$530K
General Site Works (Road, Water and Electricity Supply)	$550K
Subtotal	**$5.7M**
Contingency (30%)	$1.7M
Professional Services (15%)	$800K
Further Investigations and Consenting	$400K
Total	**$8.6M**

Notes:

1. *Costs are in NZD and are exclusive of GST;*
2. *No RLC financial and legal costs have been included in these estimates.*

The estimated annual operating cost of the WWTP and LDS is $231,000 per annum and a 40-year net present value (NPV) is $13.5M (using a 3.5% discount rate).

Table 7: Estimated Operating Costs

Item	Cost Estimate[1]
Electricity	$23,000
Chemical Use	$29,000
Solids Disposal	$28,000
Operator and Consent Compliance	$63,000
Maintenance	$74,000
Generator Rental	$14,000
Total	**$231,000**

Notes:

1. Costs exclude GST.

7.0 Summary and Recommendations

7.1 Summary

PDP has developed a conceptual design for a wastewater treatment plant (WWTP) and land disposal system to service Rotoiti and Rotoma utilising membrane bioreactor (MBR) technology prior to disposal to rapid infiltration trenches.

The site of the proposed wastewater treatment and disposal facility is on the hillside behind Emery's Store on the southern edge of Lake Rotoiti. Due to possible future residential development in close proximity to the proposed site, the concept design has included provisions to minimise odour and noise at the treatment and disposal facility and to reduce the visual impact at the site.

The estimated capital cost of the WWTP and LDS is $8.6M. The annual operating cost is $230K and the 40-year net present value of the facility is $13.5M.

Further onsite investigations and analysis is required to confirm that the concept design presented in this report.

7.2 Recommendations

The following recommendations are made to allow Rotorua Lakes Council to progress this project. Note that Items 1 to 3 should be undertaken to confirm the assumptions used in this concept design report prior to making the final decision to proceed with this project.

1. Collect additional geological information and undertake infiltration tests;
2. Install groundwater monitoring bores to establish a groundwater baseline and undertake hydrogeological analysis;

3. Undertake an assessment of environmental effects and obtain regional council agreement for the concept design;
4. Develop preliminary design including receiving proposals from MBR suppliers and update capital and operating cost estimates;
5. Prepare consent applications;
6. Throughout this process undertake stakeholder consultation.

8.0 References

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Appendix A

Cost Estimates

PE PRE-DESIGN ESTIMATE

PATTLE DELAMORE PARTNERS LTD

Name: CONCEPT DESIGN OF ROTOITI/ROTOMA WWTP AND LDS
Job No. T01548203

CAPEX ESTIMATE				
Level of Accuracy: ± 30%				
1	**MBR WASTEWATER TREATMENT PLANT**	PRELIMINARY & GENERAL	$	720,000
		INLET WORKS + SOLIDS MANAGEMENT	$	403,500
		REACTOR TANKS	$	877,600
		MEMBRANE PLANT	$	1,240,000
		PERMEATE AND CHEMICAL CLEANING SYSTEMS	$	370,000
		COMPRESSED AIR SYSTEM	$	30,000
		ODOUR CONTROL	$	150,000
		ELECTRICAL & CONTROL	$	718,400
		SUBTOTAL 1:	**$**	**4,510,000**
2	**LAND DISPOSAL SYSTEM**	PRELIMINARY & GENERAL	$	75,000
		LAND DISPOSAL SYSTEM	$	376,809
		GROUNDWATER MONITORING BORES	$	63,000
		SUBTOTAL 2:	**$**	**515,000**
3	**GENERAL CIVIL**	ACCESS ROAD	$	350,000
		ELECTRICITY SUPPLY	$	20,000
		SECURITY FENCE	$	15,000
		SUBTOTAL 3:	**$**	**385,000**
	CONTINGENCY	estimated at 30% of works	**$**	**1,620,000**
	PROFESSIONAL'S FEES	estimated at 15% of works	**$**	**810,000**
	CONSENT AND FURTHER INVESTIGATIONS		**$**	**400,000**
	TOTAL ESTIMATED PROJECT COST		**$**	**8,240,000**

Date of Estimate:	11-Aug-15
Estimate prepared by:	W McKenzie
Estimate reviewed by:	D Garden