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WAIKATO REGIONAL COUNCIL

Raglan Land Treatment Options Report

Prepared for
Waikato District Council

June 2001

pdp

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Limitations:

The report has been prepared for Waikato District Council, according to their instructions, for the particular objectives described in the report. The information contained in the report should not be used by anyone else or for any other purposes.

Executive Summary

A feasibility study has been undertaken for Waikato District Council to investigate land disposal options for disposal of treated sewage from Raglan municipal sewage treatment plant. Options of disposal by slow rate irrigation to pasture and forest, rapid infiltration to sand dunes and several combination of these options with the existing ocean discharge have been investigated.

Land disposal sites were identified by Council and the community Working Group, and the study has focussed on these sites. Treatment plant upgrading requirements for each option have also been investigated and capital O + M costs prepared for the upgrading plus disposal options.

The study has concluded that there is insufficient land area available in the areas identified by Council for slow rate irrigation disposal. Additional land could possibly be obtained, but the capital cost of an irrigation system is high.

The cheapest option is to upgrade the existing stand alone ocean discharge, but this is not favoured by the Working Group. The most favourable land disposal option is considered to be rapid infiltration or the cheaper option of rapid infiltration plus ocean discharge. These options both warrant further investigation to refine their feasibility and cost estimates.

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1.0 Introduction

Pattle Delamore Partners Ltd (PDP) has been commissioned in March 2001 by Waikato District Council (WDC) to undertake a preliminary assessment of several potential land treatment sites for disposal of treated sewage (effluent) from the Raglan municipal sewage treatment plant (STP).

The project 'brief' required investigating several treatment and disposal options including using slow rate irrigation and/or rapid infiltration and discharge via the existing ocean outfall to service the existing and future needs of Raglan township. Treatment options were to focus on upgrading the existing STP to achieve the required effluent quality and accommodate the predicted future flows. The project is being undertaken by WDC in consultation with the Raglan Wastewater Working Group (Working Group) which together are currently considering the land treatment and disposal of effluent with the intention of progressively reducing and if possible, ultimately eliminating the quantity of effluent being discharged directly to natural water ways including the ocean at Raglan. From the outset, it was recognised by WDC that a move away from the existing ocean discharge disposal system would likely require a staged approach because of the cost implications to the community. In addition, it was recognised by WDC that a practical approach may be required whereby during periods of heavy rainfall, which are known to create high sewage flows because of stormwater and groundwater entry to Raglan's aging (25 – 28 years old) sewage reticulation, an option may be to discharge some proportion of the treated effluent to the ocean at these times to supplement the land disposal system.

This report presents the results of the assessment which included a site visit to Raglan and a walk-over of the land treatment sites, soil physical and chemical testing at the sites, inspection of the existing STP, discussions with local community and iwi groups and WDC and review of background data provided by WDC.

Several potential land disposal sites were identified by the Working Group. These have been investigated as part of this project to determine their suitability and capacity for receiving effluent in addition to the preliminary assessment of the likely environmental effects. The potential sites that have been identified at this stage by the Working Group include:

Slow Rate Irrigation to:

1. Wainui Reserve Northern Gullies;
2. Wainui Reserve Eastern Gullies;
3. Raglan Golf Course on Te Hutewai Road.

Rapid Infiltration to:

1. Sand dunes on Ngarunui Beach.

Two additional sites have been identified during the course of this investigation as potentially suitable for slow rate irrigation. These are; the aerodrome area between Marine Parade and Waimanu Beach, and the high ground of the Wainui Reserve that surrounds the northern and eastern gullies (separate to areas 1 and 2 above).

1.1 Liaison with Raglan Wastewater Working Group

As part of the site visit undertaken on 10 April 2001, a meeting was held with Mr Mike Safey of the WDC and members of the Working Group. The purpose of the meeting was to let the Working Group explain the issues that they wanted addressed as part of the study and the options that they want addressed. The meeting identified several key points which are listed below:

1. Disposal of effluent to both natural water courses and other receiving waters including the ocean is to be eliminated in the long term.

2. For land disposal, different plant species are to be looked at with respect to their evapotranspiration potential. For example, the possibility of using native species such as kahikatea, flaxes and other indigenous plants is to be considered. The golf course is considered to be a prime candidate for irrigation and for use of cascading water features in gardens.

2.0 Existing Sewage Treatment Plant

The existing STP comprises two oxidation ponds located to the west of the town. The primary pond (furthest from the highway and referred to throughout this report as the primary oxidation pond or pond 1) has an area of approximately 16,000 m^2 and the secondary pond (pond 2) an area of 19,000 m^2. These ponds have been in operation since the mid 1970's. There are no screening facilities at the inlet to the ponds and the outlet is an overflow weir that feeds to a discharge pumpstation that pumps effluent to the ocean discharge. The pumpstation operates on a water level control which maintains pond 2 at a relatively constant water level. The wave bands on both ponds are in poor condition and the concrete has slumped into the ponds in several locations. The inlet pipe into the primary pond is showing signs of severe external corrosion. From information that WDC has provided, it appears there is ample land owned by WDC to the south of the existing ponds to accommodate expansion of the treatment plant or upgrading of the existing facilities.

The primary oxidation pond contains 1 HPE surface aerator (which was not running at the time of the site visit on 10 April 2001), which is attached to the central embankment. The concrete wave band is badly broken and has slipped in several locations which would limit increasing the water depth or adding extra mechanical aeration in that pond without repairing the wave band. Currently there is about 600 mm of freeboard within the pond.

The secondary pond has a very steep concrete wave band, which in some locations has slipped and cracked and needs repair.

Overall, both ponds have embankments which are between 4 to 5 m wide and the stability of which appears adequate. Total free board is in the order of 600 mm for each pond, but may be slightly more in the second pond, perhaps up to 800 mm.

Stormwater cut-off drains run around the south and eastern sides of the ponds to prevent stormwater from the adjacent hillsides entering the ponds. In some locations these require upgrading and cleaning out.

A survey was undertaken for WDC by Global Dewatering in March 2000 which indicated an accumulated sludge thickness of between 0.4 and 0.9 m in pond 1 and between 0.25 and 0.4 m in pond 2.

2.1 Existing Ocean Outfall

Effluent from the existing treatment plant gravitates to a pump station adjacent to the treatment plant from where it is pumped in a pipeline along Wainui Road and Marae Road and discharged into the harbour via an outfall at Waimanu Beach. The outfall pipe is reported to be occasionally exposed at very low tides.

2.2 Removal of Oxidation Pond 2

A report was prepared in April 2000, by V.K. Consulting Environmental Engineers Ltd (V.K., 2000) which evaluated options for upgrading the quality of effluent from the existing treatment plant. The report was commissioned by WDC in response to the decision on the notice of requirement for designation related to the 1998 Resource Consent, which stated as condition 2 that essentially the existing oxidation pond (number 2) nearest the road, should be removed in its entirety, and the area rehabilitated and replanted to recreate/restore the 1944 tidal stream alignment. The V.K. report concluded that this effectively reduced the total land area available at the treatment plant from 8.2 ha down to 6.4 ha.

The concept plan developed in July 1998 for the WDC showed the treatment plant to be reconfigured to include five new ponds, two of which were aerated lagoons. The existing primary oxidation pond was to be converted into several cells incorporating a wetland using surface flow plus a gravel bed with UV disinfection and a final settling pond prior to the effluent being discharged.

The area presently occupied by the secondary oxidation pond would be restored to its 1944 tidal flow condition and replanted. The V.K. report considered options to take into account removal of pond 2 from the treatment process and included assessment of pre-screening, filtration and alternative wetland sign.

3.0 Existing Effluent Quality

Details of the effluent quality produced by the existing STP have been provided by WDC for the period 7 February 2000 to 22 February 2001 and the averages and maximums are shown in Table 1.

Table 1: Existing Effluent Quality

Parameter [1]	**Average**	**Maximum**
BOD_5	21	51
Suspended Solids	54	148
NH_3-N	15	21
TKN	21	28
NO_3-N	0.15	0.3
DRP	5.8	7.7
TP	8.4	10.1
Faecal Coliforms	19317	88000
Enterococci	1127	8700

Notes: 1. *All values are g/m³ except Faecal Coliforms and Enterococci which are MPN/100 ml.*

The median effluent concentrations required under the existing (1998) Resource Consent for discharge to the ocean are 10 g/m³ for BOD_5, 10 g/m³ for suspended solids and 150 faecal coliforms/100 ml. These values are significantly lower (better quality) than the average values shown in Table 1.

4.0 Proposed Effluent Quality

The effluent quality that is required to be produced by the upgraded STP is governed by the disposal system that is to be used and is dependent on environmental, cultural, public perception and public health considerations.

The WDC brief for this project stipulates that for any direct discharge to the ocean such as via the existing outfall the treatment plant must produce effluent of the quality shown in Table 2.

Table 2 also includes likely effluent quality for other methods of disposal. Options that use a combination of different discharge methods (such as slow rate irrigation + rapid infiltration + ocean) will be required to either achieve the better quality of effluent at the outset or have the capability to be upgraded to achieve the required quality for each discharge method.

The effluent quality data outlined in Table 2 has been compiled based on the final quality that is expected to be required to obtain a Resource Consent for the respective discharge system. This is based on experience with other similar systems in New Zealand and can maintenance issues if higher.

only be taken as a guide as actual quality will depend on the outcome of detailed environmental investigations undertaken during the consent phase.

Table 2: Median Effluent Quality Concentrations

Parameter[1]	SRI[2] Forest/Pasture	SRI[3]	Rapid Infiltration	Ocean[4]
BOD_5	50	20	10	10
Suspended Solids	50	30	10	10
Faecal Coliforms	1000	200	500	150
Enterococci	500	100	250	NA
Total Nitrogen	150 kg/ha/yr	150 kg/ha/yr	NR	NA
Dissolved Reactive Phosphorus	10	10	NR	NA

Notes:
1. *All units are g/m³ except Total Nitrogen (at kg/ha/yr) and Faecal Coliforms and Enterococci which are MPN/100 ml.*
   NA = not specified
   NR = no limit required
2. *SRI = Slow Rate Irrigation to forest or pasture on land at the Wainui Reserve.*
3. *SRI = Slow Rate Irrigation to either the Te Hutewai Road golf course or airstrip adjacent to campground.*
4. *Concentrations as stipulated in the 1998 Resource Consent for the existing ocean discharge.*

4.1 Slow Rate Irrigation

The effluent quality for SRI to pasture and forest in the Wainui reserve and northern gullies has been based on irrigating via concealed pop-up type sprinklers or subsurface drip irrigation in the pasture areas, and surface mounted sprinklers which would be of the solid set above ground type within the forest. The total area under irrigation would be divided into about 5 equal sized plots and irrigation could occur during daylight hours. Public access to the individual plot under irrigation on any one day would be prevented so as to avoid any health issues. Buffer zones (typically 20 m width) would be planted between adjacent plots to reduce spray drift. An annual nitrogen loading of 150 kg/ha/yr has been assumed for the irrigation system which is similar to that utilized by plantation forest and grazed pasture. This will ensure there will be no excess of nitrogen applied over and above what the crop will use.

There is considered to be a greater potential for public contact with the effluent irrigated by slow rate irrigation onto the golf course or around the airstrip because the public are in close proximity to the irrigation system in greater number. This would likely require a better quality effluent so as to minimise any public health risk. Irrigation in these areas would be undertaken in the early evening and at night time.

4.2 Rapid Infiltration

Rapid infiltration requires a good quality effluent with low concentrations of suspended solids and BOD to avoid physical and biological clogging of the infiltration system. Because the rapid infiltration area is close to the Ngarunui Beach which is used for recreation and swimming, it has been assumed that faecal coliform concentrations will need to be relatively low so as to mitigate any public health risk by contact with the effluent either on the beach or in the ocean.

5.0 Predicted Population and Flows

5.1 Population

For this project current and predicted future population data for Raglan has been provided by WDC in a planning report prepared by WDC in December 2000 (WDC, 2000). The predicted base populations for years 2001, 2011 and 2021 have been taken from the WDC report together with WDC estimates of holiday visitors (defined as those using accommodation in Raglan). However, the peak summer FTE (fulltime equivalent) populations have been reduced from those in the WDC report to take into account an allowance of 4 day visitors having the equivalent sewage flow as 1 permanent resident. The WDC report had allowed for 2 day visitors being equivalent to 1 permanent resident. The population predictions are shown in Table 3. For the purposes of this report, the peak summer period when peak summer FTE population occurs has been assumed to be a two week period over Christmas.

Table 3 : Population Predictions

Year	Base Population[1]	Peak Summer Reticulated Population[2]	Day Visitors	Total Peak Summer Population	Peak Summer FTE Population[3]
2001	2,900	6,100	3,000	9,100	6,850
2011	3,400	7,625	4,660	12,285	8,790
2021	4,100	9,345	5,968	15,313	10,837

Notes:
1. *Base population from WDC, December 2000.*
2. *Includes holiday makers (bachs etc.), camping ground guests, and reticulation extensions (data from WDC, December 2000).*
3. *Total as FTE based on 4 day visitors = 1 FTE.*

5.2 Predicted Flows

Based on the population data in Table 3 and an allowance of 225 l/p/day (a typical value from other small community sewerage systems) the predicted peak summer flows are as shown in Table 4. The base flow in Table 4 is taken as 900 m^3/d from observation of the recent WDC records for treatment plant outflow, and the future values in 2011 and 2021 have been calculated from the predicted base population numbers (Table 3) with an allowance of 225 l/p/day and an additional component of 250 m^3/d to allow for rainfall on the ponds and infiltration inflow into the existing sewage reticulation. This 250 m^3/d allowance has been calculated based on the difference between the current average base flow of 900 m^3/d and the theoretical flow for the 2001 predicted population of about 650 m^3/d.

Table 4: Predicted Flows

Year	Predicted Base Flows (m^3/d)	Peak Summer Flow[1] (m^3/d)
2001	900	1,541
2011	1015	1,978
2021	1173	2,438

Notes: *1. Peak summer dry weather flow.*

From observation of the WDC records for flow out of pond 2, the predicted 2001 peak summer flow is much higher than the actual flows over the summer holiday period when rainfall is not influencing the flow. Whilst this indicates that the population prediction for 2001 maybe higher than what has actually occurred, influences such as pond storage capacity having a buffering effect on the outflow of the pond make it difficult to draw accurate conclusions.

Observation of pond 2 flow data for the period of 24 October 1997 to 9 January 2001 shows very significant rainfall influences, with winter peak flows in the order of twice the peak summer dry weather flows. The effect of rainfall on the open pond area accounts for much of this effect. However, stormwater inflow and groundwater infiltration (I/I) is also noted to be a significant problem based on the Marine Parade pump station flow records. Direct comparison of the flow data provided by WDC between the flows recorded at Marine Parade and the Pond 2 outflow was not possible due to apparent inconsistencies between the two flow recordings. However, from a graph of the full record of pond outflows (Appendix 1) it is noted the dry weather flow is relatively constant at around 900 m^3/d over the period from 1997 to the present, despite expected population growth and expected summer peaks.

Based on research on 16 wastewater treatment systems in New Zealand (G. Hauber, 1995) an annual average flow of 350 l/p/day was found for systems that would be expected to have significant I/I flows. It is notable that the 1996 census population value for the Raglan Area Unit (WDC, 2000) of 2634 multiplied by an allowance of 350 l/p/day gives a typical average flow over the 1997 to 2001 period of 922 m^3/d which agrees closely to the observed average of 900 m^3/d. For the purposes of this report the observed average flow of 900 m^3/d has been used for preliminary design.

Summer peaks are not able to be identified in the pond 2 outflow graph (Appendix 1) because the peak flows have been smoothed due to pond storage, and, even in summer the flow without rainfall influence remains at around the winter average of about 900 m^3/d. Flow data from Marine Parade does show peaks in the summer flow, however, as this does not include rainfall on the ponds, and the actual numerical values do not correlate with the pond 2 data, it has not been used in the above predictions.

From an operational point of view, it is important that correlation between the Marine Parade pumpstation flows and pond 2 outflows is established. At present the Marine Parade data suggests higher flows than the pond 2 data which is not realistic. It is recommended that WDC rectify this problem so correct data can be obtained.

5.3 Peak Flow Storage

For the purposes of this report, a requirement for storage of partially treated effluent has been allowed to be included in the upgraded treatment plant to balance out the effect of peak inflows over the two week Christmas period. The effect of providing this storage capacity will be to enable the treatment and disposal unit processes to be sized to handle the average (base) flow rather than the peak flow. In addition, this storage capacity will be useful to store effluent during short periods when ground and/or weather conditions make disposal to land impractical. Initial storage requirements have been calculated assuming sufficient storage will be provided to store the difference between the peak flow and base flow over two weeks in the year of interest and the storage volumes are shown in Table 5. More detailed flow modelling will need to be undertaken to further refine the actual volume of storage required once the disposal options have been finalised.

Table 5: Predicted Storage Volume Required

Year	Volume (m^3)
2001	12,500
2011	17,000
2021	22,000

For this report it, has been assumed the storage volume of 22,000 m^3 will be constructed during the initial upgrading of the STP. This should provide surplus balancing storage capacity until at least year 2011.

6.0 Land Treatment and Disposal Concepts

The two methods of land treatment and disposal investigated as part of this project include slow rate irrigation systems and rapid infiltration systems. Both methods are being used in New Zealand.

6.1 Slow Rate Irrigation Systems

Irrigation to land can occur using either surface spray irrigation or subsurface drip irrigation. Each type of irrigation has varying options for the hardware that performs the irrigation. For example, spray irrigation can be undertaken by either travelling irrigators or fixed (solid set) sprinklers. Travelling irrigators are less expensive to establish but require more operational input and are only suitable for relatively flat land planted in pasture or crops of low stem height. Overall, travelling irrigators are not considered suitable for the Raglan sites.

Solid set impact sprinklers (sprinklers mounted on 1.2 m high stakes, generally at a 20 m spacing) are typically in used areas where there are no stock and visual impact is not an issue, such as in a plantation forest. Alternatively, surface irrigation can be by pop-up type sprinklers in areas where there is a requirement that irrigation hardware is not visible unless in operation. Pop-up sprinklers are set into the ground and are hydraulically forced out of the ground while in operation. They are often used on golf courses and rugby fields.

For subsurface drip irrigation the entire system is buried and water is irrigated into the soil by drippers at typically 1 m spacings along a delivery pipe. The dripper is mounted inside the pipe on the pipe wall and allows a very small flow to be evenly dripped out into the surrounding soil. The dripper lines are typically placed in a grid at 1 m spacings between rows. Effluent must be well filtered to prevent blocking of the drip emitters.

6.2 Rapid Infiltration Systems

Rapid infiltration is the process whereby effluent infiltrates vertically into the ground when applied to basins formed in soils with high permeability (such as sands). The basins are periodically flooded and this cycle of loading and resting restores aerobic conditions in the soil and can help improve nitrogen removal from the effluent.

Unlike slow rate irrigation, during rapid infiltration, evaporation of effluent is minimal due to relatively short periods of infiltration. Effluent usually percolates to the groundwater and eventually flows to a surface water body without being used directly by plants. The land area required for disposal of an equivalent volume of effluent by rapid infiltration is usually much smaller than for disposal by slow rate irrigation and rapid infiltration is not adversely affected by antecedent weather conditions.

6.3 Deficit and Non-Deficit Slow Rate Irrigation

Once an irrigation system is installed the amount of water irrigated is controlled such that the irrigation regime is termed either "deficit" or "non-deficit" irrigation, depending on the amount of water added and the state of the soil moisture conditions at the time of irrigation.

In a deficit irrigation regime water is only added to soil if it can be stored within the plants root zone and used by the plant or evaporated from the ground. Soil moisture is measured to determine how much water can be irrigated. For example, the plant water use over summer may be in the order of 5 mm /day (i.e. 50 m^3 per ha per day). Hence if there was no rainfall during a period of a week, then 35 mm of water could be irrigated. Or, if there was 5 mm of rain on one day and it was fine for the rest of the week, then only 30 mm of water could be irrigated. Under the deficit irrigation regime, after recent rain or over winter the amount of irrigation possible will reduce and may be zero for much of winter.

A deficit irrigation regime for effluent application is used when soils at the site are such that applying effluent at rates higher than deficit is considered likely to lead to surface runoff or to have a potential adverse environmental effect.

In a non-deficit regime effluent is added at amounts above that required for plant uptake and evaporation (evapotranspiration (ET)). The amount of effluent applied is controlled by other factors such as soil infiltration rates, or the requirement to not produce seepage springs elsewhere, or induce slope instability.

Due to the geology of the irrigation areas under consideration and the relatively low permeability of the subsoils at Raglan, in this report a deficit irrigation scenario has been proposed for a five month summer period, and a limited deficit irrigation scenario proposed, over a 7 month winter period. During the winter period, an assumed average of 1 mm of effluent is applied per day in all slow rate irrigation areas except for the golf course. No winter irrigation has been allowed for on the golf course. The irrigation assumptions made for the winter months will need to be confirmed by detailed site testing and modelling work prior to advancing this option further.

7.0 Land Application Options and Areas

Three methods for land application of the effluent have been considered. These are by spray irrigation onto land, drip irrigation onto land, and by rapid infiltration into sand dunes.

The areas for rapid infiltration considered were on Ngarunui Beach, near the northern end of the beach (Figure 1).

The areas for spray and /or drip irrigation considered were:

(a) Wainui Reserve Eastern Gullies (Figure 1);

(b) Raglan Golf Course (Figure 2)

(c) Wainui Reserve Northern Gullies (Forest Block) (Figure 1)

(d) Wainui Reserve (majority of reserve area on the hilltops, apart from (a) and (c) above) (Figure 1).

The land area available for irrigation in each of the above, was estimated taking into account factors of land use, slope angle, and evidence of past slope instability. Available areas are shown in Table 6. Limited slope angle measurements were taken during the site visit of 10 April 2001 and were used to reject pasture areas steeper than 20° or that showed visual signs of instability. The suitability of some of the areas was estimated from visual observations during the site visit and/or interpretation of air photos. The exact areas that are considered too steep would need to be evaluated through a detailed slope survey to ascertain this more accurately. The Wainui Eastern Gullies area has been excluded from further consideration for reasons discussed in Section 7.2.

Table 6: Approximate Net Irrigation Area Available

Area	Irrigation Area (ha)
Raglan Golf Course[1]	24.1
Wainui Reserve Northern Gullies (Forest Block)[2]	8.3
Wainui Reserve (remainder on hilltops)	20.1
Total net area available of all three sites.	**52.5**

Notes:
1. *Golf Course area assumes the entire area is irrigated, including the bush covered areas (7.7ha).*
2. *This area assumes 30% is too steep (generally being steeper than about 20 degrees) for irrigation.*

Areas shown in Table 6 are the net areas required after taking into account areas that will not be irrigated because of land slope, land use or buffer zone areas.

The results of the investigation for each of the above areas are outlined in the following sections.

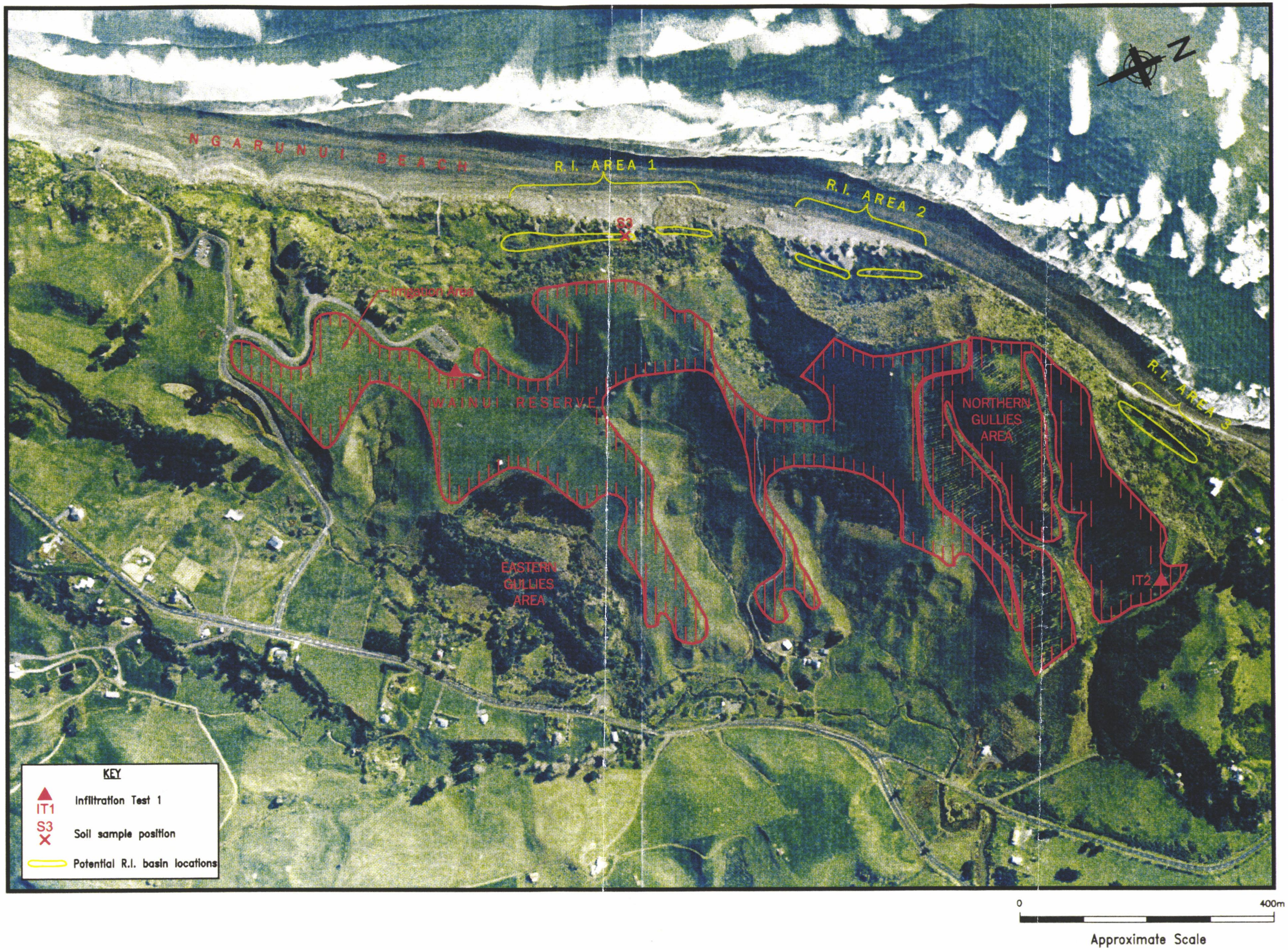


Figure 1 : WAINUI RESERVE IRRIGATION AREAS AND NGARUNUI BEACH RAPID INFILTRATION AREAS

Figure 2 : RAGLAN GOLF COURSE IRRIGATION AREA

7.1 Raglan Golf Course

This area is approximately 27.7 ha (total area) and is located on Te Hutewai Rd. Approximately 24.1 ha has been estimated as suitable for irrigation, assuming agreement with the golf club. This area assumes the entire golf course is irrigated and includes the bush covered areas (7.7 ha), but excludes an area around the clubhouse, a 20 m wide buffer strip around the perimeter of the property, and the lake area.

Irrigation would best be undertaken utilising either subsurface drip irrigation, or spray irrigation using pop-up sprinklers at night time. Irrigation of the bush covered areas would be undertaken using solid set impact sprinklers.

Deficit irrigation has been assumed to occur for 5 months of the year over summer, and no irrigation over the remaining winter period.

7.2 Wainui Reserve Eastern Gullies

This area is approximately 5.7 ha (total area) and is located on the eastern side of the Wainui Reserve (Figure 1). Part of the area has recently been planted in native trees. Approximately half of the area is covered in dense established bush and about 20% is covered in pine trees at the western end. The area is used as a public bush walk. A site walkover revealed some areas of slope instability in the stream valley that runs through the area.

Irrigation within this area is not considered feasible due to several factors including; the high percentage of steep slopes (exceeding 20 degrees), the presence of some slope instability, the public access and bush walk through the centre of the area, and the destruction of some of the planted area required for installation of an irrigation system in this area.

7.3 Wainui Reserve Northern Gullies

This area contains approximately 11.9 ha of forest in total, with 8.3 ha estimated as suitable for irrigation. The area is located at the northern edge of the Wainui Reserve (Figure 1) and is planted in pinus radiata trees. A partial site walkover of the northern edge of the area revealed some areas where slope steepness would preclude spray irrigation. Irrigation of the forest would be undertaken using solid set impact sprinklers, and carried out during the daytime to maximise evaporative losses.

7.4 Wainui Reserve (Remainder)

This area makes up the central part of the greater Wainui Reserve (Figure 1), but excludes the eastern and northern gullies discussed above. A net area of 20.1 ha is estimated to be available for irrigation. The majority of the area is grass pasture and is currently farmed and managed by a farm manager that lives on-site. The area is used by the public for recreational purposes.

A site walkover revealed some areas of past slope instability in the steep valleys that drain to the east, and in places on the western edge as well. The proposed irrigation area in Figure 1 is shaped to take account of these areas.

Irrigation over this area would either be by subsurface drip irrigation or by pop-up sprinklers. If the area was left in pasture solid set impact sprinklers would be unlikely to be acceptable due to the visual impact. Ideally, surface irrigation over as much of the area as possible would be undertaken during the daytime to maximise evaporative losses. To enable the reserve to still be in use by the public during the day if surface spray irrigation is used, it would be necessary to rotate irrigation through different areas of the reserve at different times of the day. The areas in use plus a buffer zone around them would be physically isolated from public access by fences and warning signs. Irrigation scheduling has not been examined in detail at this stage, however, combinations of surface and subsurface irrigation along with rotation between sub-areas of the reserve could enable day use of parts of the reserve in parallel with irrigation with a proportion of the reserve irrigation undertaken at night.

A future detailed design would also need to consider stock management within the area if it was continued to be grazed by beef stock, with issues of stock rotation and acceptable impacts on irrigation hardware (e.g. issues of hoof damage to pop-up sprinklers) as primary issues for consideration.

7.5 Ngarunui Beach Sand Dunes

The sand dunes are located in three distinct areas of Ngarunui Beach (Figure 1). Rapid infiltration basins behind the foredunes are proposed as a possible disposal option in this area. The sand dunes have not been considered for slow rate irrigation at this stage because of potential adverse public perception of irrigating effluent into the dunes and excluding public from them at certain times. A walkover inspection of the dunes has been undertaken in the vicinity of the Waimanu surfclub and east of the beach access walkway beneath the Wainui Reserve. The dunes by the Waimanu surf club are not considered particularly appropriate for high rate rapid infiltration because of the very flat beach gradient which may lead to seepage occurring onto the beach at certain times. However, a lower rapid infiltration application rate to the dunes in this location may well be feasible. Additional small areas maybe suitable west of the Wainui Reserve beach access walkway. The two main areas identified as potentially suitable cover a length of about 600 m and it has been assumed that about 300 m of this length would be available for rapid infiltration.

A sand sample was taken from site S3 (Figure 1) during the field visit. The sample was sent to a geotechnical laboratory and a dry grading was undertaken. The laboratory results are provided in Appendix 3 and show a fine sand with a D_{10} (10 percentile particle size) of approximately 0.1 mm. Preliminary infiltration capacity modelling has been undertaken and is discussed in Section 8.2.

Each rapid infiltration basin would be about 3 m wide by 9 m long with a depth of 1 m. The basins would be spaced in clusters along the sand dunes parallel with the beach.

The basins would be located about 50 m upgradient of the high water mark. Inspection of the site shows this land is available. Construction of the basins would be undertaken so as to minimise disturbance of the dunes and replanting of disturbed areas would be undertaken. Locked wooden lids would be constructed over the basins to prevent entry by the public. The basins would not be visible from the beach and as such would not be visually intrusive.

Effluent loading of the rapid infiltration basins would be operated automatically on a time clock system although daily inspection by the operator would be required. Initial assumptions are based on a system of approximately 50 basins operated 5 days a week and then rested for 2days. A storage tank of approximately 900 m^3 capacity would be constructed on the hill above the areas marked as R.I. Area 1 and 2 on Figure 1 and would supply the effluent to the basins (allowing a smaller diameter pumping main supplying effluent to the tank over 23 hours per day). Each of the basins would fill sequentially and then they would be left to drain.

8.0 Disposal Options

8.1 Slow Rate Irrigation

As shown in Table 7 the total area that is available for slow rate irrigation is 52.5 ha with a possible extra 5 to 6 ha at the airfield. In order to determine the irrigation capacity of the SRI area under a deficit irrigation scenario, calculations have been made based on the likely crop water requirements for irrigating an area of 52.5 ha. A value of 350 mm was calculated as a likely annual deficit irrigation requirement based on modelling carried out by Landcare Research for Environment Waikato (Watt et al, 1997). The Landcare work modelled soil moisture contents under irrigation considering several different soils and climate data from a 24 year period (1972 – 1996) from the Waikato. While the same soils as found at the Raglan sites were not used in the model, the annual value of 350 mm falls at the lowest end of the range found for different climates and soils through out the survey area (except for peat type soils). The 350 mm of water requirement corresponds to the lower end of the average annual demand (50 percentile return period), and so while it is conservative there is still opportunity for a wet year to reduce the crop water demand beneath this value. This would have the effect of reducing the depth of water that could be irrigated to this site to less than 350 mm per year.

The period of crop water demand is mostly spread over 5 months from November to March, but with minor demand also in October and April, and negligible demands in September and May. Outside this 5 month period for the remaining seven months a limited deficit irrigation scenario has been assumed of 1 mm/day. The crop water demand changes with the month of the year, with January the highest demand month, and other months at lower values. Because of the varying crop water demand, it is possible over the December to February period (3 months) to dispose of the effluent volume predicted in year 2021 by undertaking irrigation over the full 52.5 ha area available. There would be no discharges required to any other disposal system, other than storm peaks even at year 2021 flows. This assumes the summer two week peak flow volume (above the base flow component) is stored for later gradual release.

During winter, slow rate irrigation is considered unlikely to be feasible on the golf course due to the soil moisture levels expected and the effect this will have on operation of the course. The other SRI areas have been assumed to receive limited deficit irrigation of 1 mm/day which corresponds to approximately 250 m^3/d. Thus, there is a shortfall of 650 m^3/d to be disposed of at the current 2001 base flow. Some of this could be met by rapid infiltration for example. This shortfall increases to 923 m^3/d by year 2021.

A summary of the areas required and available for slow rate irrigation disposal of all the effluent except storm peaks is presented in Table 7. This assumes storage of two weeks of summer peak flow with base flow occurring for the remainder of the year. The areas shown in Table 7 are the total area required if year round slow rate irrigation disposal is used for all flows except large storm peaks. Flows induced by heavy and prolonged rainfall (such as would occur in a wetter than average rainfall year) would need to be disposed of by an alternative method such as by the existing ocean outfall. It should be noted that the golf course area is not available for winter disposal and so approximately 24 ha of additional land would be required from elsewhere. Table 7 clearly shows a significant shortfall in available irrigation land area beyond that required.

Table 7: Net Irrigation Area Required

Year	Area Required Winter (ha)	Area Required Summer (ha)	Area Available (ha)	
			Winter	Summer
2001	90	48	28.5	52.5
2011	102	56	28.5	52.5
2021	117	64	28.5	52.5

Notes: 1. *Area for 2011 approximated as average of 2001 and 2021 areas.*

8.1.1 Irrigation Application Rates

Limited infiltration field testing at two sites (IT1 and IT2 on Figure 1) has shown subsoil infiltration rates of 8 mm/hr at the Wainui Reserve, and 24 mm/hr at the Northern Gullies area. The Northern Gullies result is higher than expected and may have been influenced by pine tree roots. Topsoil at the Wainui Reserve showed a saturated infiltration rate of 30 mm/hr. The infiltration tests were undertaken during the site visit using a double ring infiltrometer and the results are appended (Appendix 2).

For SRI systems using surface spray sprinklers (pop-ups or fixed impact sprinklers) the rate of application would be in the order of 2 – 3 mm/hour to maximise evaporation and minimise potential for overland flow. This rate of irrigation is well below the saturated rate measured and hence is considered likely to prove acceptable.

8.1.2 Soils in the Irrigation Area

The soils in the irrigation area are all Raglan Clay Loam (Bruce, 1978) derived from volcanic ash, with some Horea sandy clay loam in part of the area. The Raglan clay loam typically consists of a dark brown friable clay loam topsoil over a subsoil of dark yellowish brown clay loam grading down to firm to very firm clay (Bruce, 1978). Bruce (1978) states that the surface drainage in these soils is rapid, however, internal drainage is impeded by the very firm clay subsoil. Due to its heavy texture the soil is prone to cracking in summer, when it is dried out, and water accumulation in winter. Overall, this means that the subsoils are of relatively low permeability and so, an appropriately low effluent application rate must be used, particularly in winter, for the irrigation system to work correctly.

A composite soil sample of multiple 75 mm soil cores was taken from the northern side of the Northern gullies area (forest block). Chemical analysis of the sample is appended (Appendix 4), and shows relatively low nutrient status, with very low phosphorus, low calcium, and low to medium levels of organic matter. Irrigation of the effluent would be beneficial to such a soil in terms of most nutrients. However, irrigation of effluent with relatively high sodium would require regular soil monitoring to ensure potential soil structure issues (such as decreases in permeability) did not arise. It is not expected that the Raglan effluent would contain excessive amounts of sodium but wastewater chemistry testing should be undertaken to confirm this. Soil chemistry over the other areas (golf course and remainder of reserve) has not been tested at present.

8.1.3 Slow Rate Irrigation Area Crop Species

The Working Group requested that the benefits of planting native tree or grass species on the slow rate irrigation land be investigated. In terms of calculating irrigation area capacity, the existing crops in the potential irrigation areas at the Wainui Reserve have been assumed to remain in their present state in any future irrigation development.

The majority of the irrigation area is grass pasture, which provides a good crop for irrigation as it has a relatively high evapotranspiration rate in comparison with some other potential crops. There are possibly some small gains to be made in increasing total crop water usage by planting a mixed crop of trees with open grass areas amongst the trees, but trees on their own are unlikely to provide greater water usage.

Native tree species could be planted but the growth rate of the larger species are often slower than some exotics and as such remove less nutrient and moisture from the soil. Overall, it is considered there would be little technical advantage in planting trees and/or shrubs (either native or exotic species) on the areas to be irrigated apart from some planting to form buffer zones to prevent spray drift during irrigation. However, there would clearly be an environmental and visual benefit, and possibly an economic benefit if the trees were later sold as millable crop.

8.1.4 Operational Issues

Operating an SRI system made up of the proposed areas and in a deficit regime will likely take the equivalent operational time of one full time worker based on about 6 hrs per day. There is a reasonable level of site work involved in daily operation of such a system, involving scheduling irrigation, measuring soil moisture and adjusting irrigation rates to match soil and weather conditions as well as daily maintenance and checks on the correct performance of the system. Many aspects of the system could eventually be automated at additional capital cost. However, there will always be a level of operator input required for such a system for onsite observation, monitoring and visual checking of the system whilst in operation. This will require at least a few hours once every two to three days. Initially at least, the operation of individual spray areas would be managed manually, until confidence is gained in the performance and response of the system as a whole, under varying climatic and soil moisture conditions.

8.1.5 Ponds and Flow Forms

The Working Group requested that the potential to include open water areas and flow forms within the golf course be considered and the following section discusses this.

Passing treated effluent through open water areas such as shallow ponds at the golf course could have a beneficial effect in terms of assisting effluent disposal by evaporation and soakage to ground. However, the negative side is that during periods of rainfall there would be a gain in water volume. On an annual basis rainfall exceeds evaporation, which means that open water surfaces produce a net increase of effluent volume to dispose of. Using the ponds as a treatment device to reduce organic load (BOD_5) is not considered to be a viable option as it has inherent problems of potential odour, human health implications from pathogens and mosquitoes, deterioration of water quality due to presence of bird life and nuisance growths.

Apart from these considerations is the intangible of negative perception by users of the golf course. Overall, it is considered that as a method of treatment, a pond or series of ponds at the golf course has no role. However, they could potentially perform a useful function of balancing the effluent flow over summer by providing temporary storage capacity for treated effluent prior to it being irrigated onto the golf course. The effluent would be treated at the STP to irrigable quality prior to pumping to the ponds for storage.

The quality in terms of concentrations of suspended solids and pathogenic bacteria would deteriorate within the ponds because of growth of algae and presence of birdlife.

8.2 Rapid Infiltration

Preliminary modelling of the potential daily volume of effluent that can be disposed of to rapid infiltration has been undertaken based on grain size analysis of a single sample taken from Ngarunui Beach, assumed groundwater depths and visually assessed beach profiles. All of these factors are subject to considerable variability and so the following results can only be treated as preliminary estimates.

The water level mound caused by rapid infiltration has been calculated using the Glover method (1964). The hydraulic conductivity (K) is estimated using the measured effective grain size of 0.11 mm (d_{10}) and applying Hazen's formula (for clean sand). The K value calculated using the above method is 1.4×10^{-5} m/s. Assuming an effective aquifer thickness of 30 m (a conservative thickness), the transmissivity is 4.2×10^{-4} m^2/s.

Applying the Glover method, the maximum application rate over a 150 m long rapid infiltration area was found to be 400 m^3/d. This application rate causes a 2 m groundwater level rise beneath the rapid infiltration basins. This is the maximum water level rise that can be tolerated, as the depth to water level below the basins is assumed to be about 2 m (assuming the pre infiltration water level is at the high tide level). The modelling has been based on rapid infiltration basins located in the existing depressions in foredunes spaced over a beach length of 150 m.

If effluent break out was to occur the expected break out point would be at the toe line of sand dunes about 45 m from the rapid infiltration basin. However, conservative modelling parameters have been used and it is considered unlikely breakout will occur.

It is expected the seepage zone where the effluent will enter the ocean is about the mid tide mark. Because of the relatively high (good) quality of the effluent disposed of to the rapid infiltration basins and the long travel time through the beach sands, it is not expected there will be any adverse public health impact or adverse environmental effect from the effluent disposed of via the rapid infiltration system. However, it must be reiterated that further detailed field evaluation and modelling work will be required to verify the rapid infiltration capacity of the dunes.

8.2.1 Rapid Infiltration Capacity

Because of the preliminary nature of the rapid infiltration modelling work undertaken to date, a conservative approach has been taken when assessing the rapid infiltration capacity of the dunes. Taking a conservative approach it appears that disposal of 400 m^3/d via the rapid infiltration system is possible.

Taking a less conservative approach the rapid infiltration system may well have potential to dispose of the total average winter flow of 1173 m^3/d in year 2021. Storm peaks would be treated and discharged via the ocean outfall, but summer peak dry weather flows would be stored at the STP and discharged at an average flow rate through the rapid infiltration system.

9.0 Treatment Options

9.1 Upgrading of Existing Sewage Treatment Plant

Options for upgrading the existing treatment plant have been investigated based on utilising as much of the existing plant as possible and using the existing site. From information provided by WDC, there is about 12 ha of land available excluding the pond nearest the road (pond 2) which appears to be adequate to upgrade the plant.

It is understood that of the two existing oxidation ponds, pond 2 will be demolished and the area returned to its natural mangrove/wetland state.

In looking at the upgrading options, we have assumed that the existing primary oxidation pond will remain on its present footprint but not necessarily in its present form.

The following two options have been investigated at a desktop feasibility level. Both options will prove satisfactory to handle the predicted flow and organic load through to year 2021.

9.1.1 Option 1 – Oxidation Pond System

This involves constructing a new oxidation pond upstream of the existing primary oxidation pond. The new pond will have a surface area of 1.9 ha and be about 1.5 m deep. The existing primary oxidation pond would be converted into three cells of equal volume using HDPE curtains.

Raw sewage would be diverted into the inlet of the new oxidation pond which would be lined with an impermeable HDPE liner to minimise leakage. The new oxidation pond would operate in that mode during periods of base flow. During the summer peak and peak holiday periods the pond would use mechanical aeration to boost its treatment capacity. The mechanical aerators would be similar to the aerator currently installed at the STP. Four aerators each of 3.7 kW would be installed in the new oxidation pond. At predicted peak flow, effluent would have a retention time in this pond of 10.6 days. After treatment, effluent would flow by gravity pipeline through to the inlet of the existing primary oxidation pond.

This first cell within the existing primary oxidation pond would act as an oxidation pond during base flow periods but at times of peak flow, such as holidays, would have mechanical aeration operating in similar mode to the new oxidation pond. Retention time in this cell would be about 2.4 days at peak flow.

The second cell within that pond would also be aerated and would have the same retention time.

The third cell would simply be a settling cell or maturation pond.

To undertake this upgrading of the treatment plant, additional pipework would need to be installed, the new oxidation pond would need to be constructed and existing oxidation pond 1 would need to be sub-divided and its wave band improved to withstand the erosion effects of the aeration. Aerators would also need to be installed with one aerator each of 3.7 kW capacity installed in cells 1 and 2 of the existing oxidation pond. Effluent from the settling cell would undergo further treatment depending on which disposal option is under consideration and this is discussed further in the following sections.

9.1.2 Option 2 – Aerated Lagoon/Oxidation Pond System

This would comprise constructing a new aerated lagoon and a new oxidation pond upstream of the existing oxidation pond no. 1. The aerated lagoon would be 2.5 m deep and have an effective volume of 12,500 m^3. The oxidation pond would be 1.2 m deep and have an effective volume of 6,000 m^3. A new pipeline would transfer the raw sewage into the inlet of the aerated lagoon where it would be treated. Effluent would then flow into the new oxidation pond and then from there out via a new pipeline to the existing number 1 oxidation pond. Number 1 oxidation pond would be segmented into three cells of equal volume as for Option 1 above.

Both the aerated lagoon and a new oxidation pond would be lined with an HDPE liner to minimise any leakage. The segmentation of the existing oxidation pond would be undertaken using HDPE curtains installed across the pond to effectively form impermeable barriers. In year 2021, the aerated lagoon would be required to provide mechanical aeration to the raw sewage on a daily basis and this would have an annual power cost of approximately $9,000.

Over summer, the aeration would be increased by running the aerators for a longer number of hours to handle the peak organic load. There would be 4 aerators installed in the aerated lagoon. The new oxidation pond would have 1 aerator installed and this would only be used during periods of peak loading. The first segmented cell in existing oxidation pond 1 would have one aerator, which would only be used during peak loading, the second cell would require no aeration and the third cell would be a settling or maturation cell.

In terms of capital costs, this option is about $110,000 cheaper than Option 1, but does require operating the aerators more frequently and on an annual basis with a power cost of about $9,000.

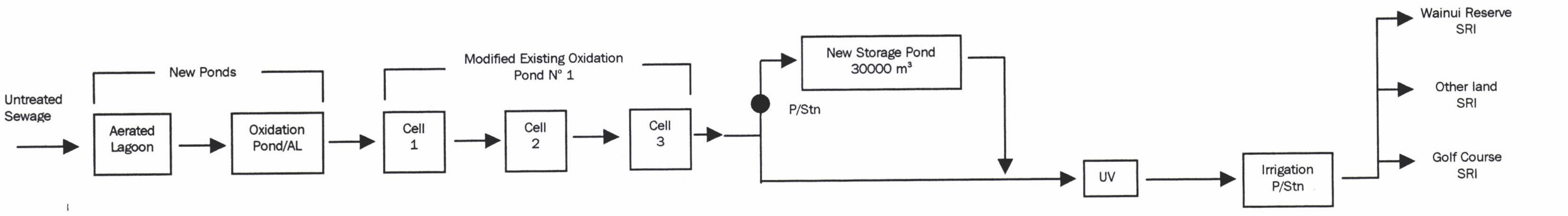
Further treatment would be undertaken downstream of these treatment ponds dependent on the disposal option and this is discussed further in the following sections.

9.1.3 Preferred Treatment Option

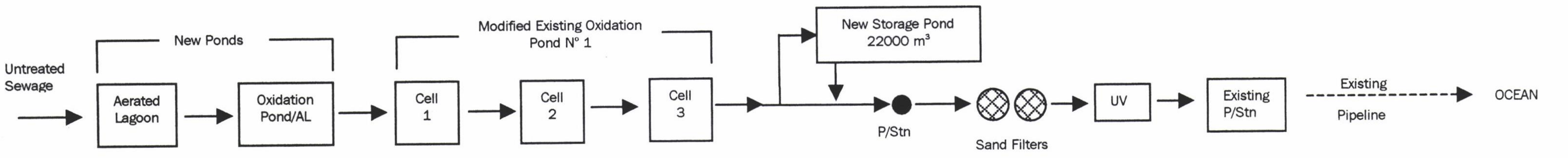
For the purposes of this report, Option 2 has been used for preparing the capital costs of the different treatment and disposal option. This option has a slightly lower expected capital cost than Option 1 although to all intents and purposes there is very little between them. Option 2 still provides good operational control and flexibility although is a little more labour intensive and expensive to operate than Option 1.

9.2 Treatment Upgrades Required for Various Disposal Options

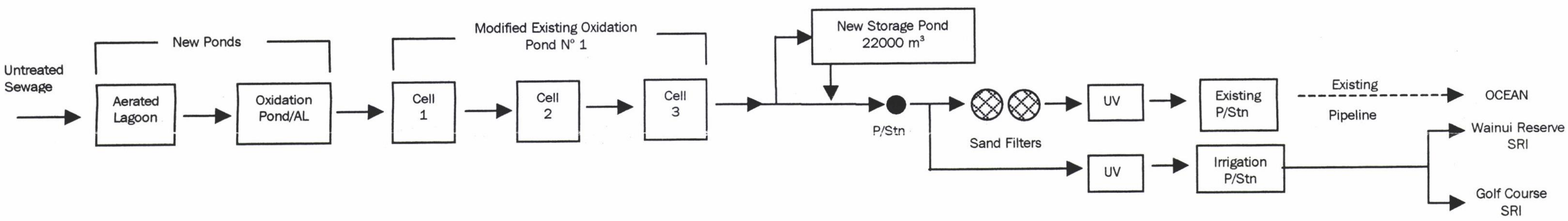
The following sections describe the treatment unit processes required for the various disposal options.



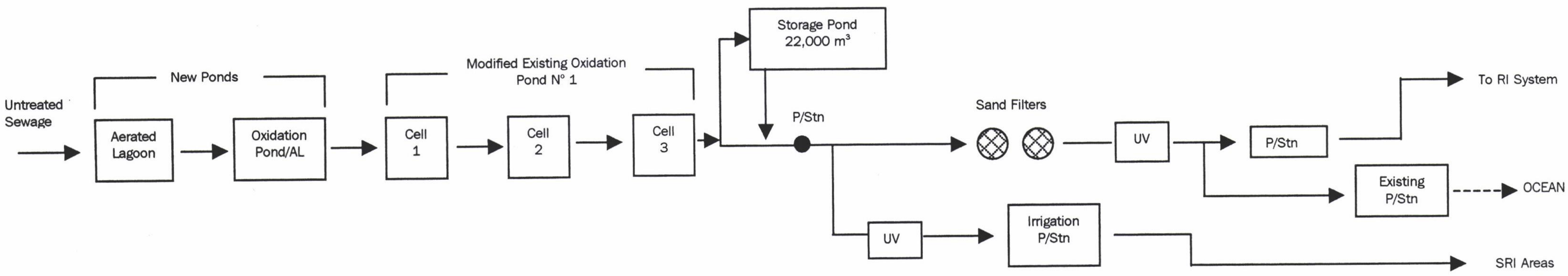
A) STAND ALONE SRI Treatment and Disposal Unit



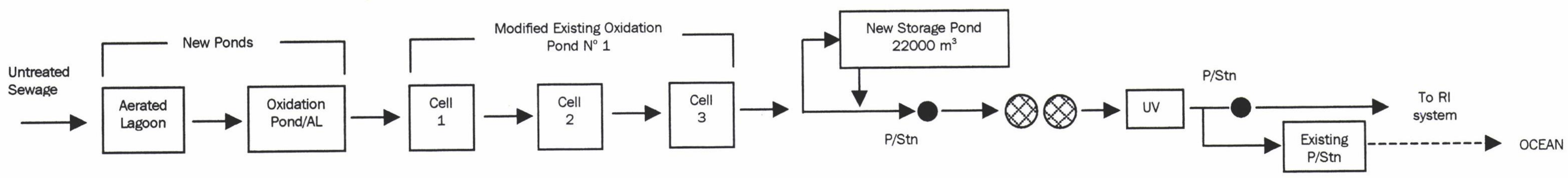
B) STAND ALONE OCEAN DISCHARGE Treatment and Disposal Unit Processes



C) SRI + OCEAN DISCHARGE Treatment and Disposal Unit Processes



D) SRI + RI + OCEAN DISCHARGE Treatment and Disposal Unit Processes



E) RI + OCEAN DISCHARGE Treatment and Disposal Unit Processes

NB. The diagrams are provisional and may alter at the time of detailed design.

FIGURE 3: UNIT PROCESS FLOW DIAGRAMS

9.2.1 Stand Alone Slow Rate Irrigation

The effluent quality required for stand alone slow rate irrigation is as shown in Table 2. To achieve this quality the unit processes as shown in diagram A of Figure 3 would be used.

The treatment plant up to the end of cell 3 has already been described in Option 2 Section 9.1.2 above.

A new storage pond with a volume of 30,000 m^3 would be constructed to store some of the peak summer flow and to also account for periods when rainfall or wet ground conditions make irrigation impractical. Effluent would be temporarily stored within this pond for later irrigation. The pond would include an impermeable HDPE liner to minimise any leakage. The liner would also act as a waveband to prevent erosion of the earth embankment due to wave action. No filtering of the effluent would be required. All effluent would be required to pass through a UV disinfection unit to reduce the concentration of pathogens to an acceptable level prior to irrigation. This will minimise the public health implications of irrigating to areas such as the golf course and Wainui Reserve.

The effluent would then pass by gravity to the irrigation pump station, where it would be pumped via separate rising mains either to Wainui Reserve slow rate irrigation area, the golf course and an additional irrigation area of 64.5 ha at a location as yet unknown.

The pumpstation and pipelines transmitting this effluent would be sized to handle the peak irrigation requirement in accordance with the deficit irrigation regime discussed earlier.

The total irrigation area required for stand alone slow rate irrigation would be 117 ha as shown in Table 7. This would require use of an additional net area of about 64.5 ha of land for irrigation over and above that currently available at the Wainui Reserve and Golf Course. At present the precise location of this additional land is unknown. However, there is farmland in the vicinity of the STP that would likely be suitable if a long term agreement could be reached with the land owner to apply the effluent to their land. For the purposes of preparing preliminary cost estimates for the Stand Alone Slow Rate Irrigation option, it has been assumed that the additional irrigation land will be located within 1 km of the STP. The capital cost of the Stand Alone SRI option has been estimated at about $7.1 M excluding any land purchase costs.

9.2.2 Stand Alone Rapid Infiltration

Based on the limited amount of investigation undertaken to date, it appears that Rapid infiltration as a stand alone option may be feasible but only if peak storm flows are discharged via the existing ocean discharge treatment. Peak summer dry weather flows would be temporarily stored for later discharge. This is not a true 'Stand Alone' option because there would still be some limited discharge via the ocean discharge but this would likely be only a small proportion. Effectively the treatment process would be the same as shown in diagram E in Figure 3 with all effluent being treated to a high quality prior to discharge. The estimated capital cost of this option is $4.13 M.

9.2.3 Stand Alone Ocean Discharge

The option to dispose of all effluent via the existing ocean outfall has been included in this report for comparative purposes. This option is not favoured by the Working Group and is in conflict with their objectives.

All effluent would be treated to the high quality shown in Table 2. The treatment system would be the same as that discussed in Option 2 above. Additional treatment processes would be required as shown in diagram B of Figure 3.

A pumpstation would be constructed downstream of cell 3, which would feed effluent into the sand filters. These sand filters will remove algae and biological material to achieve the 10 g/m^3 suspended solids concentration that is required for discharge. Effluent from the sand filters will then be passed through a UV disinfection unit to remove pathogenic bacteria down to low concentrations. This effluent will then pass by gravity to the existing pumpstation where it will be pumped down the existing pipeline to the ocean discharge.

Whilst this option is not preferred by the Working Group, it does have some inherent positive features, such as the ability to more easily accommodate periods of rainfall. For example, this system is not controlled by wet weather or wet ground conditions, whereas any system relying on land disposal alone is governed by these factors. This means that rainfall does not stop discharge of effluent to ocean whereas wet ground condition will prevent discharge of effluent via slow rate irrigation.

The stand alone ocean discharge option includes a storage pond which would temporarily balance out the summer peak flows and thereby allow the unit processes downstream of cell 3 to be sized for slightly more than the average flow with peak flow being balanced through the storage pond. This will likely be cheaper in terms of capital cost and running cost than if the downstream unit processes were sized to handle the peak flow, which only occurs for about 2 weeks per year.

The capital cost of the ocean stand alone option would be approximately $2.5 M excluding any costs associated with upgrading the existing outfall pumpstation and pipeline. WDC have advised that the present 200 mm diameter pipeline is likely to be too small to handle storm events within the Consent discharge time limits and will need to be upgraded at a cost of about $0.7 M. This makes the cost of this option about $3.2 M.

9.2.4 Stand Alone Discharge to Receiving Streams

The option of stand alone discharge to a receiving stream is not considered to be viable, given that there are no receiving waters in the vicinity of the existing treatment plant that would not be heavily impacted by the discharge of well treated effluent. For example, unless nutrients (nitrogen and phosphorus) were removed to very low concentrations discharge of well treated effluent into the receiving stream adjacent to the existing treatment plant would likely have consequential adverse effects on the inner harbour estuary in terms of leading to undesirable biological growths.

In addition, discharge of well treated effluent to a receiving water is not considered to be acceptable by the Working Group. For these reasons, this option has not been considered further.

9.2.5 Combination Slow Rate Irrigation + Ocean Discharge

This combination would use slow rate irrigation as the disposal means during summer, up to the capacity of the 52.5 ha area that is available for irrigation as discussed in Section 8. Any flow over and above what the irrigation system can accommodate, would be discharged via the existing ocean outfall after treatment. For year 2001 during the winter period, this means that on a daily basis approximately 650 m^3 is discharged via the ocean outfall plus whatever rainfall events occur. For year 2001 during the summer peak period, all flow over a five month period is discharged to slow rate irrigation, except for large storm peaks.

In the year 2021 during winter, 923 m^3/day plus rainfall induced peak events are discharged via the ocean outfall and during summer the daily volume can be accommodated for a three month period during December, January and February without any discharge to the ocean apart from any storm flow induced peaks.

The treatment process would use the pond system outlined in Option 2 above. The treatment process is shown in diagram C in Figure 3. Downstream of cell 3 a new storage pond would be included with a capacity of about 22,000 m^3 to balance out the peak summer flow and provide additional storage when ground or weather conditions mean that irrigation is not feasible. A pumpstation would then transfer the treated effluent to a UV disinfection unit and from there to the irrigation pump station where it would be pumped to the Wainui Reserve irrigation area and the Golf Course irrigation area.

During times when discharge to the ocean is required, additional treatment would comprise sand filtering followed by UV disinfection prior to effluent being discharged into the existing pumpstation where it would be pumped to the existing ocean outfall.

The approximate capital cost of this option including construction of the irrigation pumpstation, irrigation pipelines to the Wainui and Golf Course irrigation areas and the irrigation systems would be $5.05 M.

This $5.05 M would provide a system that would be capable of handling the predicted flows through to year 2021. This capital cost could be deferred by in the order of $0.25 M if construction of the capital works was staged so that capacity of the unit processes at the outset was matched to handle the flow in year 2011, rather than 2021. At year 2011 upgrading of the unit processes would be required.

9.2.6 Combination Slow Rate Irrigation + Rapid Infiltration + Ocean Discharge

This option is more complex than the previous options because it adds the additional disposal mechanism of rapid infiltration into the equation which has implications from both the treatment plant points of view. Additional unit processes must be added to the treatment system to achieve the required effluent quality suitable for rapid infiltration. This will increase the operating cost and will make the operation of the treatment plant more complex. The treatment plant process diagram is shown in diagram D in Figure 3.

The approximate capital cost of this option is $5.95 M. There is a potential deferral in the short term of between $0.3 M and $0.5 M on the capital cost if the system is initially constructed to handle the flow in year 2011. This money would need to be spent at year 2011 to increase the ultimate capacity to handle the flow for 2021.

9.2.7 Combination Rapid Infiltration + Ocean Discharge

The system would use a combination of rapid infiltration to sand dunes on Ngarunui Beach (of 400 m^3/d) plus discharge to the ocean of treated effluent to handle the surplus flow beyond the capacity of the rapid infiltration system as well as flow from peak rainfall events.

In the year 2001 this would mean that there would be a volume of approximately 500 m^3/d during winter and about 750 m^3/d during summer discharged via the ocean outfall with a constant flow of 400 m^3/d discharged by rapid infiltration.

In the year 2021 during winter 773 m^3/d plus rainfall peaks would be discharged via the ocean outfall with 400 m^3/d discharged by rapid infiltration and in summer during the two week peak period approximately 2,038 m^3/d would be discharged via the ocean outfall with 400 m^3/d being discharged via the rapid infiltration system. The treatment process is less complex than combination other options and is shown in diagram E of Figure 3.

The treatment pond system would comprise unit processes as outlined in Option 2. Downstream of these ponds there would be a pumpstation supplying effluent to sand filters which would remove suspended solids down to a quality of 10 g/m^3 prior to UV disinfection and then discharge via the existing ocean outfall. A side stream of the sand filtered effluent at the rate of 400 m^3/d would be pumped to the rapid infiltration system. UV disinfection of the effluent that is to be rapid infiltrated, would also be undertaken prior to disposal to ensure that public health is not compromised in the event of effluent coming into human contact and to ensure edible shellfish quality is not compromised. Effluent will be further treated by natural processes as it passes through the sands before reaching the ocean.

The approximate capital cost of this treatment and disposal option is estimated at $3.31 M. This assumes that there is no upgrading of the existing outfall pumpstation and pipeline required.

10.0 Capital and Operation and Maintenance Costs

Approximate capital costs have been prepared for the various treatment and disposal options discussed above to upgrade the existing treatment plant and construct the disposal system based on accommodating the predicted sewage flow in year 2021. These costs make no allowance for surveying, consultation or attainment of Resource Consents, WDC costs (administration/finance etc), legal costs or GST.

Also no cost allowance has been made for removing the secondary oxidation pond and returning the land to its original condition. Similarly no cost has been allowed for purchase of land for the treatment plant should extra land be required.

The capital costs discussed do include a 15% allowance for engineering design and construction supervision and also a 15% allowance on total costs as a contingency sum. The accuracy of all costs have been determined to within an estimated ±35%, being based only on a desk top study, with no specific field details utilised in their determination (such as geotechnical evaluation, hydrogeological modelling or detailed engineering study).

Approximate annual Operational and Maintenance (O and M) costs have been calculated based on the following percentage assumptions which are typical for capital works:

- 5% of electrical and mechanical costs
- 2.5% of total pump station costs
- 1% of irrigation capital costs, civil works and reticulation
- 5% of treatment capital costs
- Power at $0.15/kWhr
- Operator Salary $60,000/yr/operator

A single operator is assumed for the treatment plant, who is also available for some disposal system work. The total number of full time equivalent operators is shown in Table 8 for the various options:

Table 8: Full Time Operators	
Option	**Number of Operators**
Stand Alone SRI	2
Stand Alone RI	1
Stand Alone Ocean	1
SRI and Ocean	2
SRI and RI and Ocean	2
RI and Ocean	1

The total O + M costs have been calculated based on a system suitable for year 2021 flows but do not include any allowance for deferred capital for upgrading or replacement works.

The estimated capital and annual O and M costs are summarised in Table 9.

Table 9: Capital Costs and O and M Costs		
Option	**Capital Cost ($M) Excl land purchase**	**Annual O and M ($M)**
Stand Alone SRI	7.10	0.45
Stand Alone RI	4.13	0.25
Stand Alone Ocean	3.20	0.20
Combination SRI + Ocean	5.05	0.31
Combination SRI + RI + Ocean	5.95	0.34
Combination RI + Ocean	3.31	0.22

The capital cost estimates presented in Table 9 vary widely for the different options. The Stand Alone Ocean option is the cheapest in terms of both capital and O and M costs. The $3.2 M capital cost essentially reflects the cost of upgrading the existing STP to treat the predicted flow in year 2021 to the high quality required by the 1998 Resource Consent and to upgrade the existing outfall pipeline.

An interesting comparison is the Stand Alone RI with the Combination RI + Ocean. For the combination option, the rapid infiltration system is used to dispose of 400 m^3/d in year 2021 with the surplus volume being discharged to the ocean.

The Stand Alone RI system discharges 1173 m^3/d in year 2021 with a limited discharge to the ocean to accommodate peak storm flows, but only costs an additional $0.8 M to construct and an additional $30,000 per year to operate.

11.0 Assessment of Other Relevant Issues

In addition to an economic analysis of the various options, other relevant issues have been considered in order to provide a comprehensive assessment of the options. A summary table has been prepared to compare the options under consideration under six criteria, as defined below:

- "Reliability" = Security of proposed system against failure
- "Consents" = Likelihood of obtaining consents without lengthy appeal process
- "Environment" = Ability of the system to provide adequate long term environmental and public health protection
- "Operation" = Difficulty and complexity of operation from a technical viewpoint
- "Social" = Public perception and concerns
- "Cultural" = Iwi perception and concerns

Table 10 provides a subjective evaluation of the performance of the various options relative to these issues within each category. Whilst it is recognised that the evaluation is purely subjective, it does provide an alternative method of ranking the options which can be compared with the economic assessment. Each option is rated against the others by assigning an unweighted value to it. The lower the value, the more favourable the option.

The 'consents' ranking has not been included in the Sum of Rating because there is already a consent in place which permits discharge to ocean.

Table 10: Evaluation Matrix

	Reliability	Consents[1]	Environment	Operation	Social	Cultural	Sum of Rating (excl. Consents)
Stand Alone SRI	6	1	1	6	1	1	15
Stand Alone RI	3	2	5	3	2	2	15
Stand Alone Ocean	1	-	6	1	6	6	20
SRI + Ocean	4	4	2	4	4	3	17
SRI + RI + Ocean	5	3	3	5	3	4	20
RI + Ocean	2	5	4	2	5	5	18

Notes: 1. Consent already granted for Stand Alone Ocean

Based on the assessment in Table 10, the options involving disposal by application solely to land (Stand Alone SRI and Stand Alone RI) rank as the most favourable. However, the SRI option ranks worst in terms of reliability and operational complexity but scores well on the other criteria. Overall, Stand Alone RI appears to be a more 'middle of the road' option with no extremes at either end of the ranking scale.

12.0 Summary and Recommendations

Based on the work undertaken as part of this study, the slow rate irrigation land disposal sites identified do not contain sufficient land area to dispose of the predicted annual sewage flow in the year 2021. Additional land could probably be sourced, but the capital and O and M costs of this option are very high.

The Stand Alone Ocean option is cheapest in terms of both capital and O and M costs, but is not favoured by the Working Group. In addition, whilst it is a technically simple option, it ranks badly in the Evaluation Matrix. Of the combination options, RI + Ocean scores well on cost and middle of the range in terms of the Evaluation Matrix. This option appears to represent a reasonable compromise between cost and other non-economic factors and merits further investigation. Overall, the Stand Alone RI option ranks highest taking into account all factors, but has a higher capital and O and M cost than the combination RI + Ocean option.

The following course of action is recommended:

- Undertake further detailed engineering investigation of the STP upgrade works and refine upgrading costs;
- Undertake further detailed engineering, hydrogeological and environmental investigation of the Stand Alone RI and combination RI + Ocean options to further refine the feasibility and associated costs.
- Prepare a public information document with the options outlined, including the refined costs and consult with public as to their preferred option.

PDP is in a position to assist WDC and the Working Group with the action steps outlined above.

Report Prepared by:

Andrew Sussex, Rob A. Docherty

FINAL 15/06/01

13.0 References

Bruce J. (1978), *Soils of Part Raglan County, South Auckland, New Zealand*, Soil Bureau DSIR, Wellington.

Glover, R. E (1964), *Groundwater Movement*, Engin. Monogr., Vol 31, US Bureau

of Reclamation, 67pp.

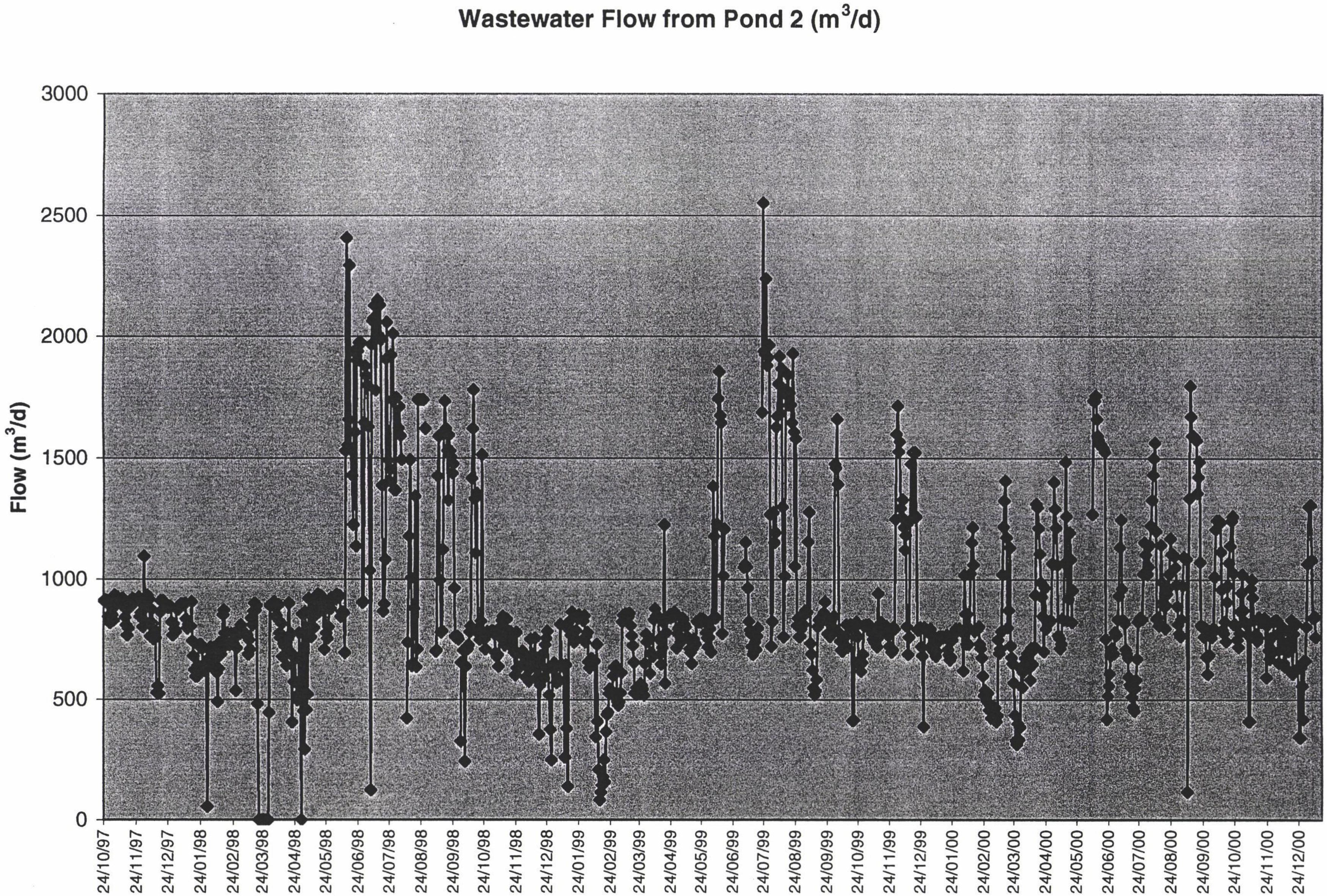
Hauber G. (1995), "Wastewater Treatment in N.Z. Evaluation of 1992/93 Performance Data – ORGD", *Water and Wastes in New Zealand*, May 1995.

V.K. Consulting Ltd (1998), *Raglan Wastewater Treatment and Disposal System Upgrade Treatment Options on Reduced Footprint*. Options Report.

Watt J. McIndoe I. Green S et al, (1997), *Crop Water Requirements for Irrigation in the Waikato Region*, Landcare Research Report prepared for Environment Waikato.

Appendix 1

Wastewater Flows


Wastewater Flow from Pond 2 (m3/d)
Flow (m3/d)
3000
2500
2000
1500
1000
500
0
24/10/97
24/11/97
24/12/97
24/01/98
24/02/98
24/03/98
24/04/98
24/05/98
24/06/98
24/07/98
24/08/98
24/09/98
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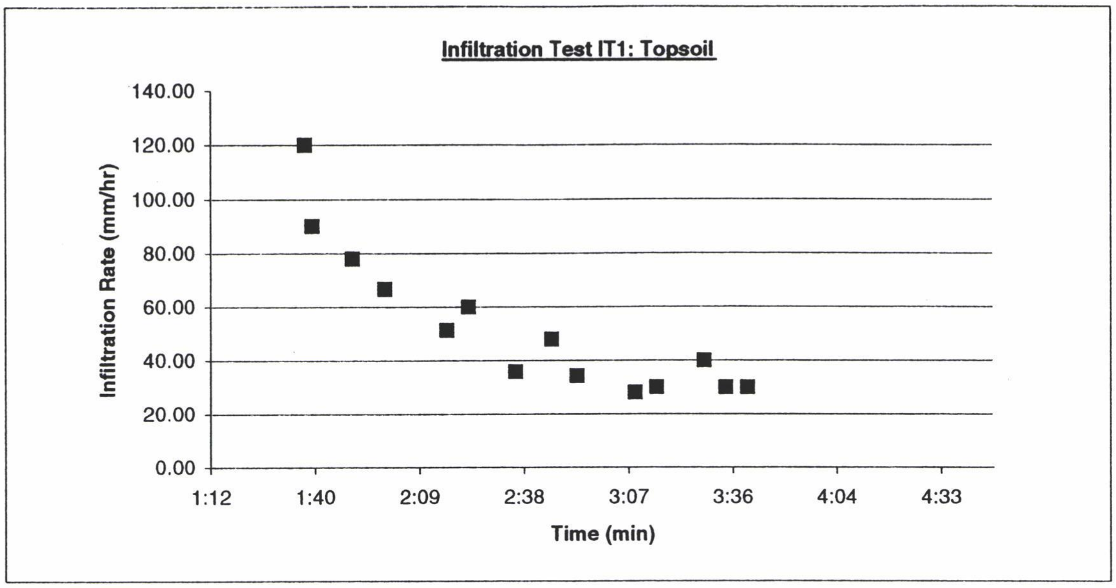
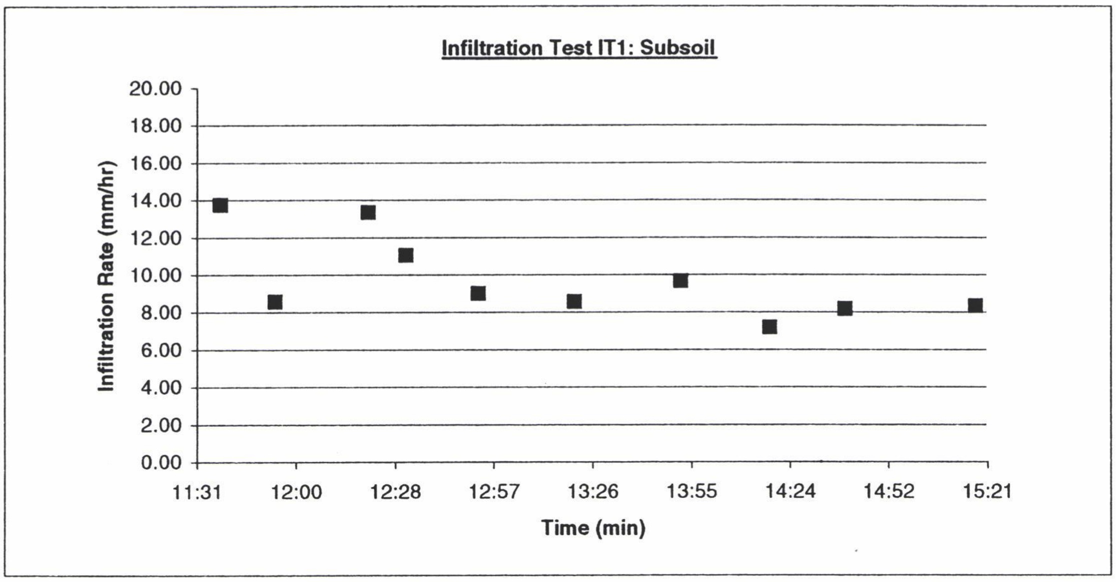
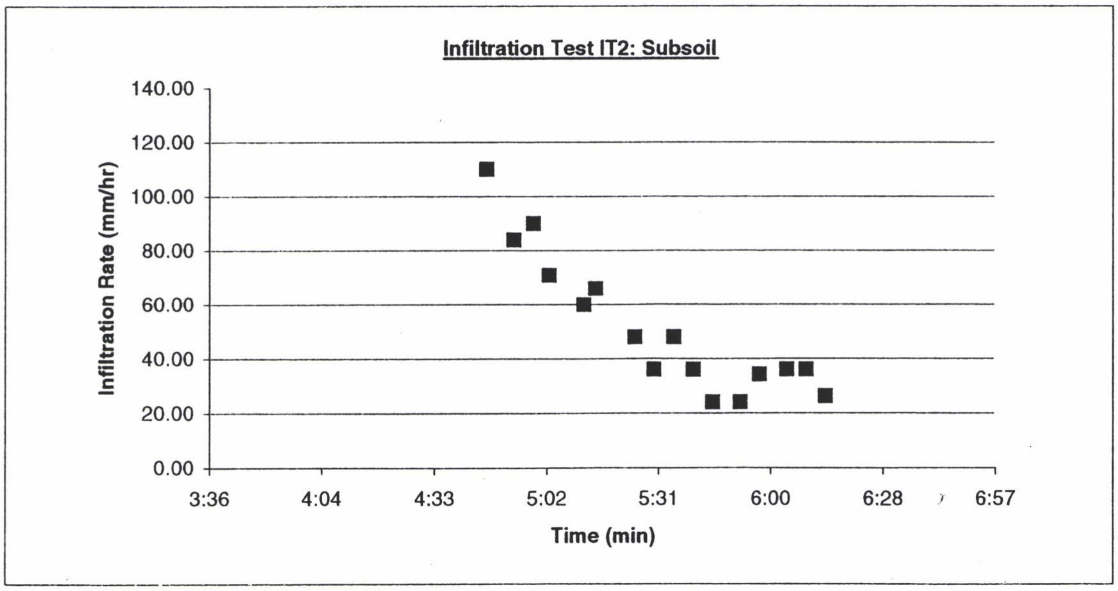
24/04/00
24/05/00
24/06/00
24/07/00
24/08/00
24/09/00
24/10/00
24/11/00
24/12/00

Appendix 2
Infiltration Test Results







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Appendix 3

Grading Curve Results

ENVIROLAB GEOTEST
132 Vincent St
PO Box 5585
Auckland
Ph (09) 300 9005
Fax (09) 300 9300

PARTICLE SIZE DISTRIBUTION

Job Name: Raglan AJ815
Client: Pattle Delamore Partners Ltd
Date: 27 April 2001

Job No.: 6006108/089
Tested By: J. Gisel
Checked By: I. Otene

Test Pit No.: -
Sample No.: S3
Depth (m): -

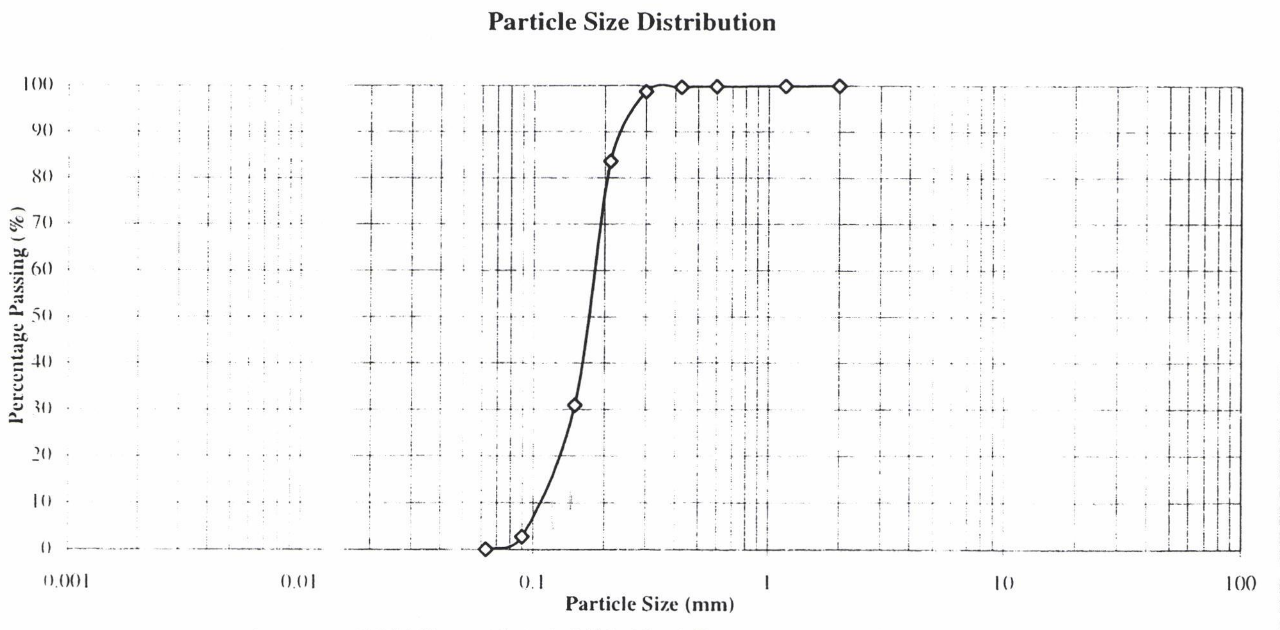
Sample Type: Bulk
History: As Received
Report No: 1126L

Sample Description: Dark brownish grey fine to medium SAND; moist, non-plastic.

Test Standard: NZS 4402:1986, Test 2.8.2

Coarse & Intermediate Fraction		Fine Fraction	
Sieve Size	% Passing	Sieve Size	% Passing
75mm		2mm	100
63mm		1.18mm	100
53mm		600μm	100
37.5mm		425μm	100
26.5mm		300μm	99
19mm		212μm	84
13.2mm		150μm	31
9.5mm		90μm	3
6.7mm		63μm	0
4.75mm		<63μm*	0

*Mass passing 0.063mm obtained by difference



Authorised Signatory:......................
A.Pickard

ENVIROLAB
GEOTEST

Pattle Delamore Partners Ltd
PO Box 9528
NEWMARKET

27 April 2001
Our Ref: 6000000/089
L1:47401-JDG14L03.DOC

Attention: Andrew Sussex

RECEIVED
01 MAY 2001

Dear Sir

Raglan AJ815 Dry Grading

Please find attached the results from the dry grading test (Report No. 1126L) carried out on sample S3 for the Raglan AJ815 project. I have also included a copy of our current testing prices if you require any testing in the future.

Thank you for the opportunity to be involved with this project and if you have any queries please contact the undersigned.

Yours faithfully
Envirolab Geotest Ltd

Justin Gisel

Direct Dial: +64-9-300 9005
Email: jgisel@beca.co.nz
JDG:krj

Appendix 4

Soil Chemistry Results

Hill Laboratories

RJ Hill Laboratories Limited

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Private Bag 3205,
Hamilton, New Zealand

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Facsimile:
+64 (7) 858-2001

Email:
mail@hill-labs.co.nz
Internet:
www.hill-labs.co.nz

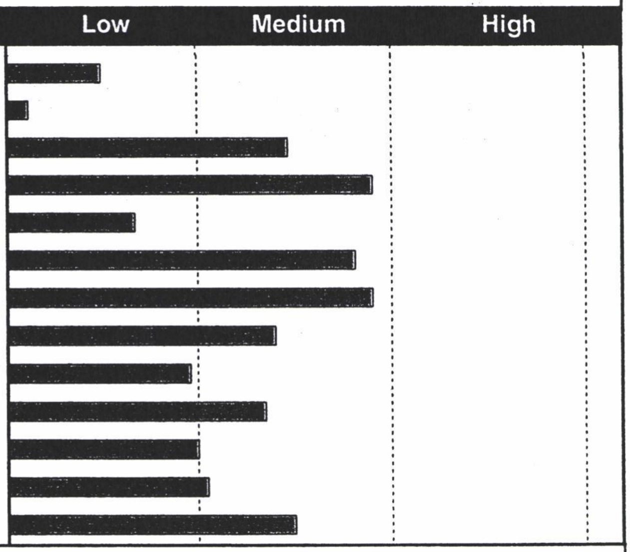
ANALYSIS RESULTS

Client: Pattle Delamore & Partners
Address: P O Box 9528
Newmarket
AUCKLAND

Laboratory No.: 33107/01
Received: 12-Apr-2001
Despatched: 18-Apr-2001
Order No.:
Submitted By: A Sussex
Job Comment: Project AJ815

Sample Name: Forest Block
Sample Type: SOIL General, Outdoor (S10)

Analysis		Level Found	Medium Range
pH		5.4	5.8 – 6.5
Olsen P	(ug/ml)	6	20 – 30
P Retention	(%)	44	30 – 60
Potassium	(me/100g)	0.77	0.50 – 0.80
Calcium	(me/100g)	4.7	6.0 – 12.0
Magnesium	(me/100g)	2.62	1.00 – 3.00
Sodium	(me/100g)	0.28	0.10 – 0.30
CEC	(me/100g)	17.2	12.0 – 25.0
Base Saturation	(%)	49	50 – 85
Volume Weight	(g/ml)	0.74	0.60 – 1.00
K/Mg Ratio		0.3	0.3 – 1.0
Organic Matter	(%)	7.5	7.0 – 17.0
Total Nitrogen	(%)	0.45	0.30 – 0.60



Base Saturation Data	%K 4.5	%Ca 28	%Mg 15.3	%Na 1.6
MAF Cation Units	K 12	Ca 4	Mg 44	Na 10

The above nutrient graph compares the levels found with reference interpretation levels. NOTE: It is important that the correct sample type be assigned, and that the recommended sampling procedure has been followed. R J Hill Laboratories Limited do not accept any responsibility for the resulting use of this information.

No Laboratory Comments

RECEIVED
20 APR 2001

Hill Laboratories

RJ Hill Laboratories Limited

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1 Clyde Street
Private Bag 3205,
Hamilton, New Zealand

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ANALYSIS METHODS

Client: Pattle Delamore & Partners
Address: P O Box 9528
Newmarket
AUCKLAND

Laboratory No.: 33107
Received: 12-Apr-2001
Despatched: 18-Apr-2001
Order No.:
Submitted By: A Sussex
Job Comment: Project AJ815

The following table gives a brief description of the analysis methods used for this job. The COV (coefficient of variation) gives a measure of precision and is sometimes referred to as the Relative Standard Deviation, ie the standard deviation expressed as a percentage of the absolute value.

For further details and explanations, please contact the laboratory.
These samples were collected by yourselves (or your agent) and analysed as received at this laboratory.

Test	Extraction/Digestion	Determination	COV(%)
Soil Samples			
Sample Preparation	Air dried at 35°C for two days (residual moisture typically 4%) and crushed to pass through a 2 mm screen. Tests performed and reported on this basis.		
Volume Weight	None	The weight/volume ratio of dried, ground soil	2
pH	1:2 (v/v) soil:water slurry	Potentiometrically using a pH electrode	1
Phosphorus	Olsen extraction	Molybdenum Blue colorimetry	6
Extractable Cations	1M Neutral ammonium acetate extraction	ICP-OES.	4
Cation Exchange Capacity	None	Summation of extractable cations (K, Ca, Mg, Na) and the acidity determined from the change in pH of the cation extraction solution	4
Total Base Saturation	None	Calculated from Extractable Cations and Cation Exchange Capacity	4
Organic Matter	None	Dumas combustion. Organic Carbon is converted to Organic Matter using a factor of 1.72.	5
Total Nitrogen	None	Dumas combustion	10
Phosphate Retention	Equilibration with 1000 mg/L P solution	Molybdo-vanadate colorimetry	2
ASC	Equilibration with 0.02M potassium phosphate solution	ICP-OES	2

365

This laboratory is accredited by International Accreditation New Zealand (previously known as TELARC). The tests reported herein have been performed in accordance with its terms of accreditation. Accreditation does not apply to comments and interpretations. The 'normal range' levels and the subsequent bar graph interpretations are specifically excluded. This report may not be reproduced, except in full, without the written consent of the signatory.

Signatory: Tony Kay