Feasibility Study

Franz Josef Wastewater Treatment Plant Planning

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Prepared By

Penny Dyer / Grant Webby
Environmental Engineer / Technical Principal
(Hydraulic and River Engineering)

Reviewed By

John Crawford
Technical Principal - Wastewater

Opus International Consultants Ltd
Greymouth Office
23 High Street

PO Box 365, Greymouth 7840
New Zealand

Telephone: +64 3 769 9330
Facsimile: +64 3 768 7498

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Contents

Executive Summary

The Franz Josef Waste Water Treatment Plant (existing plant) consists of two oxidation ponds that discharge treated effluent into the Waiho River. The existing plant is owned and operated by Westland District Council (WDC).

There are two issues that are of considerable concern for the existing plant. The first issue is that, in the next 5 years, the Waiho River is likely to break out (i.e. overtop the river bank) in a significant flood through the oxidation ponds towards the Tartare Stream and inundate the ponds. Sediment material transported by the flood flows would be likely to fill the ponds, even if the river does not destroy the ponds. While this might not necessarily destroy the ponds (it would necessitate excavation of the ponds and rehabilitation of the inlet and outlet pipes to make them functional again), the ongoing sediment aggradation on the Waiho River fan could eventually allow permanent avulsion of the Waiho River through the pond site to the Tartare River. The ponds would therefore be lost to the river. The second issue is that the ponds are overloaded and are periodically discharging noncompliant water into the Waiho River.

Prior to WDC requesting this report, previous studies on the flood hazard potential of the Waiho River had focused only in general terms on the flood capacity of the various flood protection works on either side of the Waiho River fan, although they had probably also underestimated the flood capacity of these works. The studies had not specifically addressed the flood risk to the oxidation ponds. The 2012 West Coast Regional Council Report on future management of the Waiho River (Hall, 2012) did identify the high risk to the oxidation ponds but did not quantify this.

WDC engaged Opus to conduct a river analysis to determine the likely lifespan of the ponds and to provide a Net Present Value (NPV) cost analysis on the two options that WDC proposed for the plant. These two options were:

1. To leave the plant operating for the medium term, undertaking essential upgrades to improve discharge quality and building a new waste water treatment plant in approximately 10 years' time, or

2. To build a new waste water treatment plant (WWTP) as soon as possible in a different location that is not susceptible to river encroachment.

The results from the river engineering analysis predict that there is a very high chance that the Waiho River could inundate the ponds before 2019, and a reasonable chance that the ponds could be inundated before 2016. This is of course dependent on the actual storm events that affect the Waiho River Catchment in the next five years.

With this new information Option 1 appears to no longer be a feasible option. This report therefore only considers the NPV for Option 2. Possible upgrades and their capital costs for Option 1 have been included in this report in the event that West Coast Regional Council (WCRC) stipulates that the existing plants discharge water quality requires improvement whilst the new plant is built.

The most likely recommendation that we have provided for the existing plant to improve discharge water quality and lessen the capital costs required are:

- Desludging of the Ponds
- Modifying the outlet pipe to improve the diffusion of the discharge into the Waiho River
- Modifying the inlet pipes
- Modifying the existing wetland and baffled curtain system on Pond 2

If each of these upgrades are implemented a capital cost of approximately $600,000 will be required.

For the construction of the new WWTP the key factors to consider were:

- The new site location. Four potential locations have been identified by WDC. Each site will require slightly different capital and operational costs e.g. land purchase price, consenting approval, pumping of wastewater to the plant and discharge costs.
- The type of WWTP. In this report we have assumed that the plant will be an high rate treatment plant. Cost estimates have been provided for two types of reactors; an earthworks reactor and a concrete reactor.
- The type of disposal system. We have estimated capital costs for a high cost system (rapid infiltration disposal field) and a low cost system (disposal diffuser).

The median capital cost required for the new WWTP is $8.9 million (this cost is in current dollars). The 95th percentile capital costs for the new WWTP is $9.5 million. A NPV analysis for the first 30 years of the new WWTP's life estimated a median NPV of $23.25 million and a 95th percentile NPV of $24 million for the new WWTP.

As the river engineering analysis predicts that the existing plant will be inundated by the Waiho River within 5 years we recommend that WDC move forward with the planning and development of the new WWTP forthwith.

1 Introduction

1.1 Purpose

Westland District Council (WDC) has requested Opus International Consultants (Opus) to undertake an options assessment and a benefit/cost analysis for the upgrade/replacement of the Franz Josef Waste Water Treatment Plant (WWTP).

When WDC requested this report, there was a fundamental assumption that the flood risk to the oxidation ponds would not become significant for at least another 10 years. . During the course of this current investigation, the river engineering assessment carried out as part of it determined that this assumption was invalid. The results from the river engineering analysis predict that there is a very high chance that the Waiho River could inundate the ponds before 2019, and a reasonable chance that the ponds could be inundated before 2016. This is of course dependent on the actual storm events that affect the Waiho River Catchment in the next five years. The initial purpose of this report was then changed from an options assessment report to a costing analysis for a new WWTP.

This report also considers possible intermediate upgrades to the existing plant while waiting for the new plant to become operational. Capital costs for these upgrades have been included.

1.2 Background

Franz Josef WWTP is owned and operated by Westland District Council (WDC). At the commencement of the report WDC was concerned with two key issues impacting the existing WWTP. These are:

- The Waiho River bed is aggrading and migrating towards the oxidation ponds. There are therefore significant risks of outflanking and or inundation of the treatment plant by the river.
- The ponds are overloaded and are periodically discharging noncompliant effluent into the Waiho River. The West Coast Regional Council have issued abatement notices to the Council.

WDC is currently considering the following two options to address the above issues:

Option 1: Leave the plant operating for the medium term and undertake essential upgrades to improve the discharge quality. A new WWTP will be constructed in another location in say 10 years' time, or

Option 2: a new WWTP as soon as possible in a different location that is not susceptible to river encroachment.

A component of this current exercise has been to undertake a review of the likely levels of risk that the river poses to the existing treatment plant. This analysis[1] found that the expected life of the

[1] Refer section 3 below.

existing plant is likely to be less than 5 years. With this information, it is our opinion that Option 1 no longer seems like a viable solution.

WDC has provided Opus with reports that were produced by SKM (2009) and G2e (2013) that have extensively covered the topics of the capacity and noncompliance of the existing treatment plant and recommending potential upgrades to the plant to achieve compliance.

Figure 1: Franz Josef Oxidation Ponds

1.3 Scope

In this report we originally planned to undertake an options assessment to determine whether it would be more cost effective to upgrade the new ponds or construct a new WWTP immediately for WDC to achieve the required discharge compliance limits. During the course of this investigation the results from the river engineering assessment predicted that there is a high probability that the existing plant could be inundated within the next 5 years. With this in mind we determined that Option 1 was highly unlikely to be considered feasible due to the existing plant's short life expectancy. The initial purpose of this report was then changed from an options assessment report to a costing analysis for a new WWTP.

There is little remaining doubt that construction of a new treatment plant, away from the existing site, is required. A new WWTP will need to be constructed on a different site that is not susceptible to river encroachment. Four proposed locations for the new plant sites have been considered. The new WWTP will probably discharge into the Tartare Stream due to limited land availability for a land disposal option and the need to move sufficiently out of reach of the expanding Waiho River active alluvial fan.

We have conceptualised a new WWTP based on the assumption that stricter resource consent (RC) conditions than the ones currently in place for the existing ponds will be set by the West Coast Regional Council (WCRC) because of the smaller size of the receiving water. Apart from this and likely population growth figures, there has been little solid information on which to conceptualise a replacement plant and so sum assumptions are pure conjecture, based on our experience elsewhere.

A NPV analysis over the first 30 years of the new WWTP has been conducted. The NPV includes both capital and operational costs. Costs estimates have been compiled using information provided by suppliers, industry standards and estimates from recent WWTP construction reports. An inflation rate of 2.5% and a capital growth of 6.5% have been assumed.

During the initial stages of this report, prior to receiving the results from the river engineering analysis, a large number of potential upgrades were considered for the existing plant. With the results from the river engineering analysis predicting a high probability that the existing plant could be inundated within 5 years, we recognise that it may not be prudent to invest such a large injection of capital into the existing plant. For information, we have, however, included a capital cost estimate based on recommended upgrades

We have assumed that the plant will only receive wastewater loads from the Franz Josef Township sewerage main and from septic tank loads. The present and future wastewater flows and loads from the town have been estimated using the assumptions provided in the SKM and G2e reports. Septage information has been provided by Hibbs Drainage. We have made an assumption that stock trucks have their own disposal system and will not be discharging into the existing or new plants.

We have assumed that the existing wastewater mains are sufficiently sized.

2 Methodology

As part of this report the following steps have been undertaken:

- On the 25 March 2014 Opus staff (John Crawford and Grant Webby) attended a work shop with WDC to discuss potential options for the future of the ponds. A site visit to the ponds by John Crawford, Grant Webby and Petrina Cannell was conducted during the course of this visit.
- A river engineering analysis has been conducted on both the Waiho River and Tatare Stream to predict their future movements due to the ongoing sediment aggradation from their alluvial fan.
- An analysis has been conducted on the existing ponds to determine the costs required to upgrade the ponds to achieve the new RC conditions that have been proposed in Section 30.
- A 30 year NPV has been conducted to determine the costs involved in constructing and operating the new WWTP.

3 River Assessment

3.1 Assessment of Fluvial Risks to Existing WWTP

3.1.1 Location of Existing WWTP Relative to Waiho River Fan

The Waiho River below the State Highway 6 (SH6) Bridge crosses a classic alluvial fan, the fan being a conical-shaped surface upon which sediment material transported by river flows (particularly during floods) is deposited and subsequently reworked.

The aerial photo shown on the drawing of the Waiho River Flood Protection Scheme (sourced from GEMC (2008)) in Appendix A shows the extent of the upper part of the Waiho River alluvial fan. Geomorphological processes on the fan are influenced by the presence of lateral boundary constraints in the form of flood protection stopbanks along the left bank between the SH6 bridge and Canavan's Knob and below the Waiho Loop, and along the right bank between the SH6 bridge and the existing Franz Josef Village wastewater treatment plant (WWTP). These processes are also affected by the Waiho Loop (a terminal moraine relic left behind by historic glacial retreat) which severely constricts and forces the river to skirt around it at its south-western end, approximately 5.3km downstream of the SH6 bridge.

Because of the ongoing geomorphological processes affecting the alluvial fan, the active fan surface is monitored by means of regular cross-section surveys[2]. The location of the monitoring cross-sections is shown on the drawing in Appendix A.

The oxidation ponds forming the Franz Josef Village WWTP are sited along the right bank immediately adjacent to the Waiho River alluvial fan. They are located more precisely close to the right bank end point of cross-section 19. This end point lies opposite the prominent left bank feature known as Canavan's Knob, a remnant rock knob which sits in the middle of the wider alluvial fan (the left bank end points of both cross-sections 18 and 19 are located around the base of this particular knob).

Figure 2 shows a photograph of the right bank stopbank past the oxidation ponds as viewed looking downstream. The stopbank is armoured with heavy rock riprap material to provide erosion protection.

3.1.2 Geomorphic Context of Waiho River Alluvial Fan

SH6 crosses the Waiho River at the head of the alluvial fan about 5 km below the present terminus of the Franz Josef Glacier (Figure 3). The Callery River joins the Waiho River a few hundred metres above the SH6 bridge crossing.

The Waiho sub-catchment (77 km^2 in area) incorporates the Franz Josef Glacier (Figure 3) which covers about 18% of the catchment area (Davies and McSaveney, 2001). From 1985 to 2006, the glacier was generally advancing (McSaveney, 2013) but since then has been rapidly retreating.

[2] Bed levels at the SH6 bridge are also actively monitored by Opus International Consultants Opus) on behalf of the New Zealand Transport Agency (NZTA) with a much greater frequency than the alluvial fan cross-sections.

Figure 2: Rock armoured stopbank along right bank of Waiho River alluvial fan adjacent to oxidation ponds at Franz Josef Village WWTP

The Callery sub-catchment (92 km² in area) is slightly larger than the Waiho sub-catchment (Davies and McSaveney, 2001). It is also heavily ice-covered with three glaciated tributaries covering about 18% of the sub-catchment area which is similar to that for the Waiho sub-catchment. Below these glaciated tributaries the Callery sub-catchment is characterised by a 10 km long deeply incised gorge with very high steep sides prone to slope failure (Davies, 1997).

Between the Callery / Waiho confluence and the fan head at the SH6 bridge crossing, the river is confined by high uplift terraces on the right (north) bank and the glacier access road on the left (south) bank. Both the glacier access road on the left bank and the first 100 m upstream of the SH6 bridge on the right bank are protected by erosion protection works (rock spurs founded deep into the river bed). The abutments of the SH6 bridge constrict the river channel at the fan head.

Below the SH6 bridge, the active river bed widens out over an alluvial fan. The section of the fan down to the Waiho Loop (Figure 3), the remains of a terminal moraine feature from an historic glacial advance, is known as the upper fan (Davies and McSaveney, 2001). The river channel on the upper fan develops a braided form and frequently switches course rapidly under flood conditions. Stopbanks along both the left bank past the Holiday Park to Canavans Knob (a residual bed rock feature in the middle of the upper fan) and the right bank past the church, the heliport area, the Franz Josef Hotel and the village oxidation ponds significantly confine the angle of the fan apex. Canavans stopbank protects SH6 to the south of the Waiho River crossing from inundation under flood conditions across a wider part of the fan surface towards Docherty Creek (Figure 3).

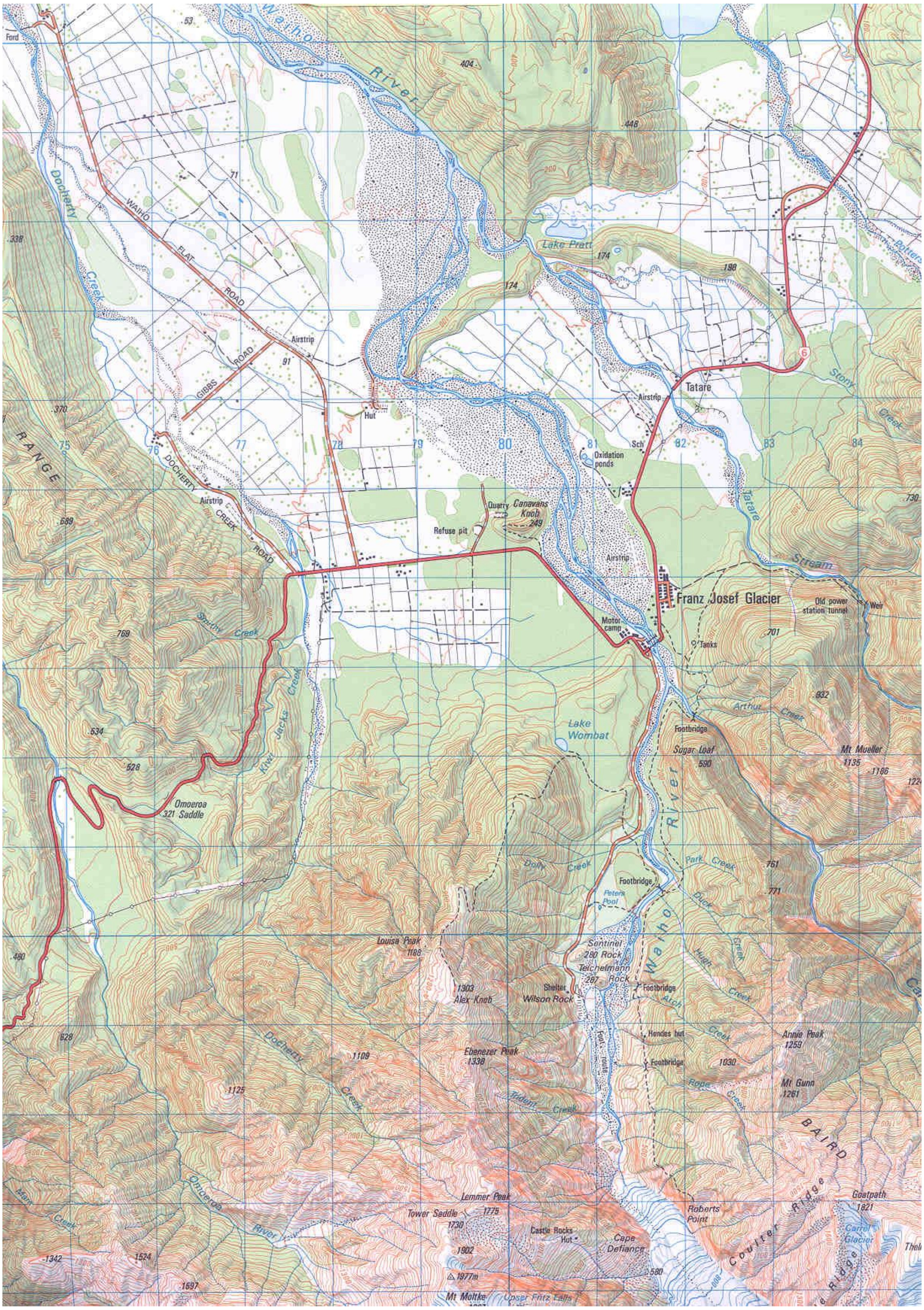

Figure 3: Extract from NZMS 260 series topographic map showing location of Franz Josef Glacier, Waiho River, Callery River, Franz Josef Village and WWTP

The section of fan below the Waiho Loop (Figure 3) is known as the lower fan. As the Waiho River passes the end of the Waiho Loop, it turns to the right in a north-easterly direction and flows down a steeper sub-fan before turning left in a north-westerly direction and joining the Tartare River which cuts through the Loop. From this confluence the Waiho River follows the north side of the valley for a distance of about 10.5 km to the sea. Where the Waiho River passes the end of the Waiho Loop, Eatwell's stopbank has been constructed to prevent breakout of the river across farmland and the Waiho Flats airstrip into Docherty Creek (McSaveney and Davies, 1998).

The Tartare River (Figure 3) drains a small catchment with an area of about 3 km^2 to the north of the Callery catchment (McSaveney and Davies, 1998). It flows out from the mountains about 2 km to the north-east from where the Waiho emerges at the SH6 bridge crossing and then follows its own incised fan to the cut in the Waiho Loop. The bed of the Tatare is lower than that of the Waiho above the Loop.

3.1.3 Sediment Inputs to Waiho River Fan

Sediment is supplied to the river system from both the Waiho and Callery sub-catchments.

The recent retreating behaviour of the Franz Josef Glacier means that the glacier will be injecting a significant volume of sediment material into the Waiho River system although occasional rockfalls also deposit large volumes of material directly onto the valley floor (Mosley, 1983). The bed of the Waiho River immediately below the glacier is about 0.5 km wide over a distance of 2.5 km which provides substantial storage for sediment delivered by the sub-glacial drainage system (Davies and McSaveney, 2001). However, below Sentinel Rock, the river bed narrows significantly and provides no storage capacity for sediment material. The reach between Sentinel Rock and the Waiho River / Callery River confluence therefore functions as a sediment transfer reach.

The narrow Callery River Gorge also provides no storage for eroded sediment and similarly functions as a sediment transfer conduit to supply sediment material to the Waiho River fan.

Hall (2012) describes a range of sediment delivery sources for the Callery Gorge including:

- rainfall-induced landslides, landslide dam formation and subsequent failure due to flood overtopping
- glacial derived material
- gully erosion material from other non-glaciated tributaries
- debris flows
- shallow slab avalanches from the steep upper sides of the gorge
- major seismic-induced mass movements (due to the proximity of the major Alpine Fault and other splinter faults)

Hall (2012) comments further on the timescale and rate of sediment input from the Callery Gorge into the Waiho River system as follows:

> *The steep, rock lined, confined nature of the Callery Gorge means that whilst sediment inputs occur into the main channel routinely and in an episodic manner, the residence*

time in the gorge is short i.e. with the possible exception of major landslides, floods and freshes subsequent to the sediment entering the gorge, sediment entering the gorge will pass through and be discharged into the Waiho River below the Waiho–Callery River confluence in a relatively short time frame.

In considering the nature of the sediment sources identified above, it is opined that a variety of temporal supply mechanisms will be operating simultaneously at any one time.

This situation means that sediment movement through particularly the Callery River, which subsequently feeds into the Waiho River, might over time manifest as a reasonably steady base flow of sediment input to the Waiho River below the confluence but none the less it will actually comprise a series of episodic sediment injections from upstream. From time to time significant sediment injections will occur and these will give rise to fluctuations in sediment loads above the base level of sediment throughput just described.

Like the Callery River Gorge and the reach of the Waiho River between Sentinel Rock and the Waiho / Callery Confluence, the next very short reach of the Waiho River down to the State Highway 6 Bridge also acts in a geomorphic sense as a sediment transfer reach. Sediment delivered to this reach from upstream by the Callery and the Waiho Rivers passes through the reach to the upper fan downstream. Depending on the balance between the rate of sediment supply and the sediment transport capacity of the water flow through this reach, the river bed in this reach may be either aggrading, degrading or remaining at an approximately constant slope.

The bed is very active in this transfer reach. It is constantly changing over time with bed levels varying and the primary braid channels switching position.

Evidence for the active nature of the bed at the bridge crossing is given by:

- the stage (water level) plots shown in Appendix A of Webby and Waugh (2003) (these plots show stage measured over time by the NIWA recorder at the bridge) with periodic upwards shifts; and
- the plot of mean bed levels measured at the bridge on a regular basis for NZTA.

Figure 4 shows the latter plot of mean bed levels. From this, sudden episodic upward shifts in mean bed level are seen to have occurred in March 1999, between March 2009 and March 2010 and January 2011. These sudden episodic shifts reflect the injection of significant sediment volume pulses onto the head of the Waiho River fan by large flood events.

3.1.4 Observations and Interpretations of Behaviour of Waiho River Fan

In recent years, a range of interpretations has been put forward regarding geomorphological trends in behaviour of the Waiho River fan based on the evidence available. These were debated at a workshop held as part of an International Gravel-Bed Rivers Symposium at Franz Josef in September 2000.

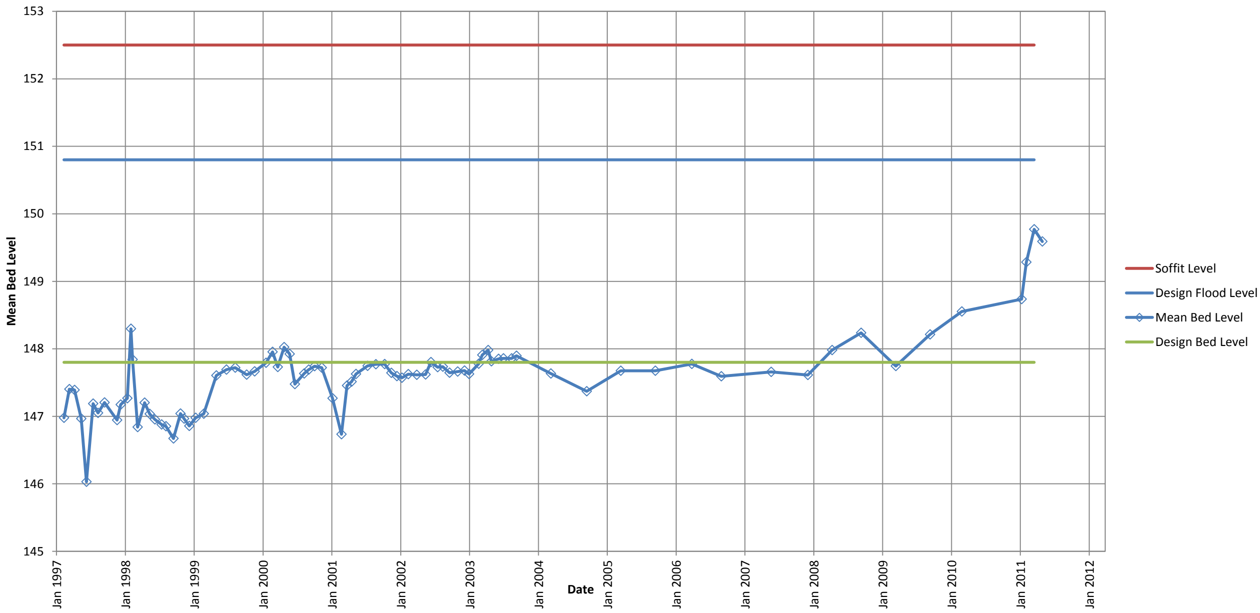

Figure 4: Mean bed levels in Waiho River at site of State Highway 6 Bridge Crossing (1997-2012)

Davies and McSaveney (2001), reiterating their earlier interpretation in McSaveney and Davies (1998), hypothesised that the fan is in an equilibrium state (with the sediment transport capability matching the sediment supply) but that, since human intervention, the effect of continued confinement of the upper fan by stopbanks on both sides has been to induce fan aggradation. However, Williams (2001), in a discussion of Davies and McSaveney's (2001) paper, offered an alternative view that the fan is not in an equilibrium state and that the stopbanks may simply be influencing a natural trend of aggradation by restricting the deposition of transported sediment into adjacent areas across the wider fan surface. Mike Church, one of the overseas experts who made a presentation at a public meeting on the problems of the Waiho River as part of the International Gravel-Bed Rivers Symposium Workshop, also questioned the hypothesis of the fan being in an equilibrium state (Rouse *et al*, 2001).

It is difficult to ascertain the correct interpretation of actual fan behaviour as the effects of stopbank confinement of the fan and the natural fan-building process are impossible to separate out from the available field evidence.

Since the International Gravel-Bed Rivers Symposium Workshop was held in 2000, more intensive monitoring in the form of cross-section surveys of the Waiho River fan has taken place. The evidence obtained from these surveys is evaluated and discussed in the next section.

3.1.5 Recent Quantitative Trends of Bed Levels on Waiho River Fan

In this section, the monitoring data from the January 2002, June 2008 and March 2011 cross-section surveys are re-examined in greater detail compared to the analysis by Hall (2012). However we have not re-analysed the data. In our re-examination we have re-used the mean bed level and sediment volume data calculated by Hall (2012) and presented in Table 1 of Appendix 5 of his report.

The purpose of our re-examination of the monitoring data was to discern more precise bed level trends on the upper part of the fan in order to extrapolate possible future bed levels. We have focussed particularly on that part of the upper fan between cross-section 15 (where the fan first starts to expand past the heliport) and cross-section 22 (the pinch point at the Waiho Loop) (refer to the drawing in Appendix A).

Appendix B presents graphs of the March 2011 cross-sections over-plotted on the June 2008 cross-sections for cross-sections 15 – 22. There are a number of clear trends evident from these cross-sections over-plots:

(a) There is a distinct gradient across the fan from left to right on cross-sections 16-22, with the left side higher than the right side, i.e. the fan surface is tilted from left to right between these cross-sections.

(b) On cross-sections 16-18, there appear to be distinctive braid channels down the left side of the fan, even though bed levels are higher.

(c) In contrast, on cross-sections 19-21, the predominant braid channels occur down the right side of the fan.

(d) The left side of the fan on most cross-sections is higher than the crest level of the right bank stopbank (or top of bank) level (cross-sections 16, 17, 19, 20, 21).

(e) Bed levels are distinctly higher in March 2011 compared to those in June 2008.

Figure 5 and Figure 6, which show aerial photographs of the upper point of the fan in January 2011 and October 2013, confirm points (b) and (c) above (refer to the drawing in Appendix for the location of the various cross-sections). In fact the aerial photograph from October 2013 (Figure 6) indicates that the switch in location of the left side braid channels over to the right side is even more accentuated than it was in January 2011. This could very well be correlated with the sharp upward shift in the mean river bed level at the site of SH6 Bridge Crossing seen in Figure 4.

Figure 7 shows mean bed levels down the Waiho River fan as calculated by Hall (2012). Mean bed levels at cross-sections 16–21 are above the top levels of the bank on the right side of the fan. This trend is even more accentuated in Figure 8 which shows maximum bed levels on the left side of each cross-section (excluding bank top levels) down the fan. While the mean bed levels are typically in order of 1-2 m higher than right bank top levels, the left side maximum bed levels are as much as 3-4 m higher than the right bank top levels.

Figure 5: Aerial photograph of upper part of Waiho River fan (January 2011) (sourced from NZ Aerial Mapping)

Figure 6: Aerial photograph of upper part of Waiho River fan (October 2013) (sourced from Google Earth)

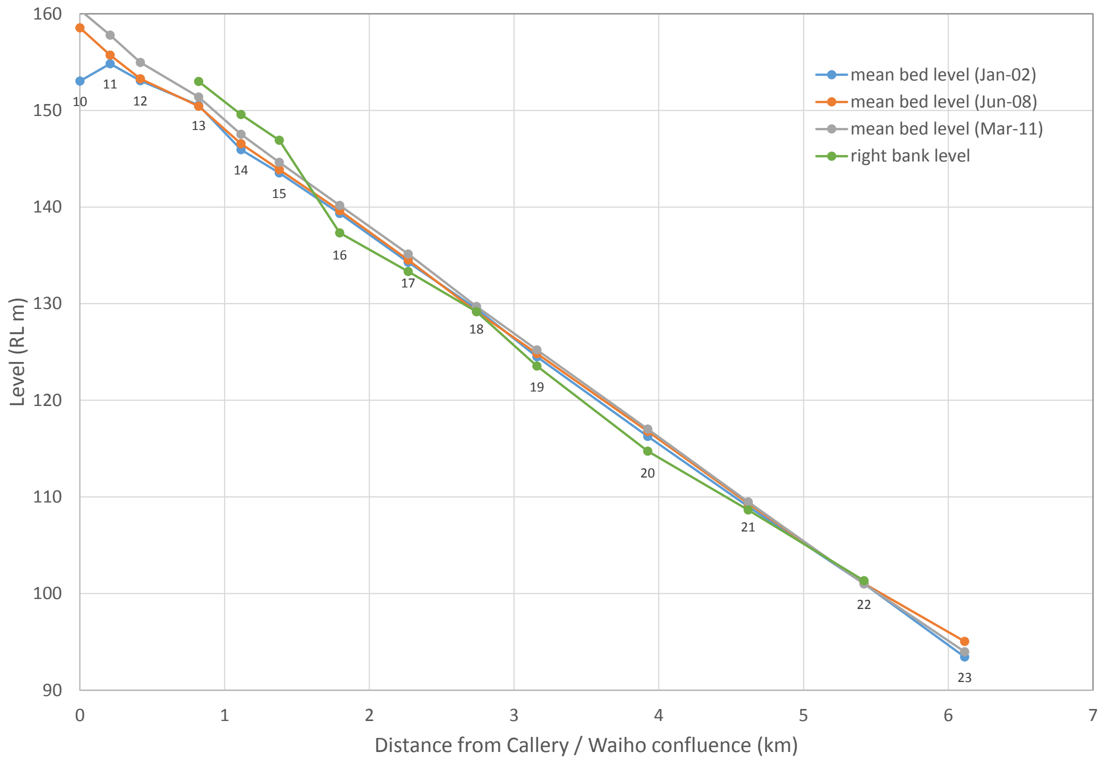

Figure 7: Mean bed levels down Waiho River fan between 2002 and 2011 (cross-section numbers attached to each data point)

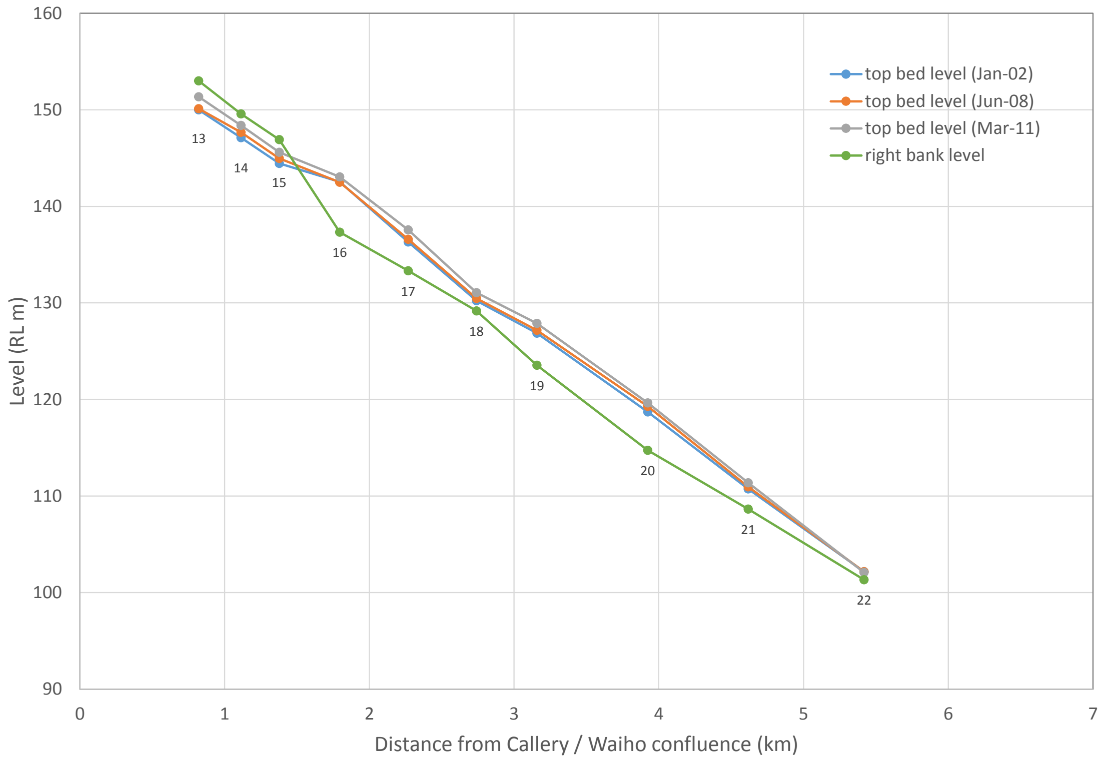

Figure 8: Top bed levels down Waiho River fan between 2002 and 2011 (cross-section numbers attached to each data point)

Figure 9 shows the average bed level change per year down the fan computed from the data presented by Hall (2012). The data shows two distinct trends:

(a) from January 2002 to June 2008, the fan surface between cross-sections 15-21 showed an approximately uniform increase of 0.050 m/yr; and

(b) from June 2008 to March 2011, the fan surface between cross-sections 15 and 22 showed an approximately linear decrease in average bed level change from 0.275 m/yr to 0 m/yr.

The latter trend represents a distinctly greater aggradation volume across the fan surface.

In both cases the average bed level change at cross-section 22, the pinch point on the Waiho Loop, was very close to 0 m/yr.

Figure 10 shows the cumulative bed volume gain from sediment aggradation down the fan between the three surveys. A positive gradient on the curves in this graph is indicative of aggradation while a negative gradient is indicative of degradation. The whole of the fan surface was aggrading between January 2002 and June 2008 with more than half of the aggradation occurring below cross-section 21. In contrast the fan surface was aggrading at a much faster rate between June 2008 and March 2011, but below cross-section 21, degrading. Figure 11 translates the cumulative bed volume data in Figure 10 into cumulative bed volume change values per year. The shorter inter-survey period from June 2008 to March 2011 means that cumulative bed volume change per year was substantially greater than in the previous January 2002 to June 2008 inter-survey period. The shape of the 2008-2011 curves in Figure 10 and Figure 11 is suggestive of a large sediment pulse moving down the upper fan.

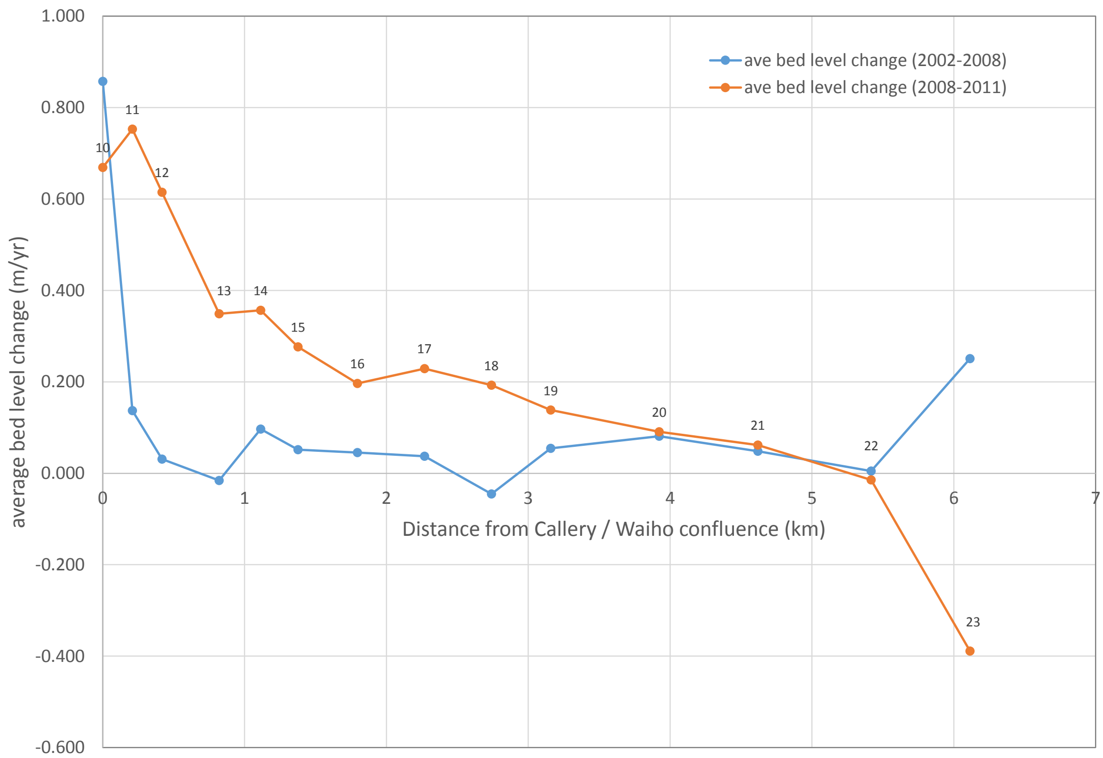

Figure 9: Average bed level change per year on Waiho River fan between 2002 and 2011 (cross-section numbers attached to each data point)

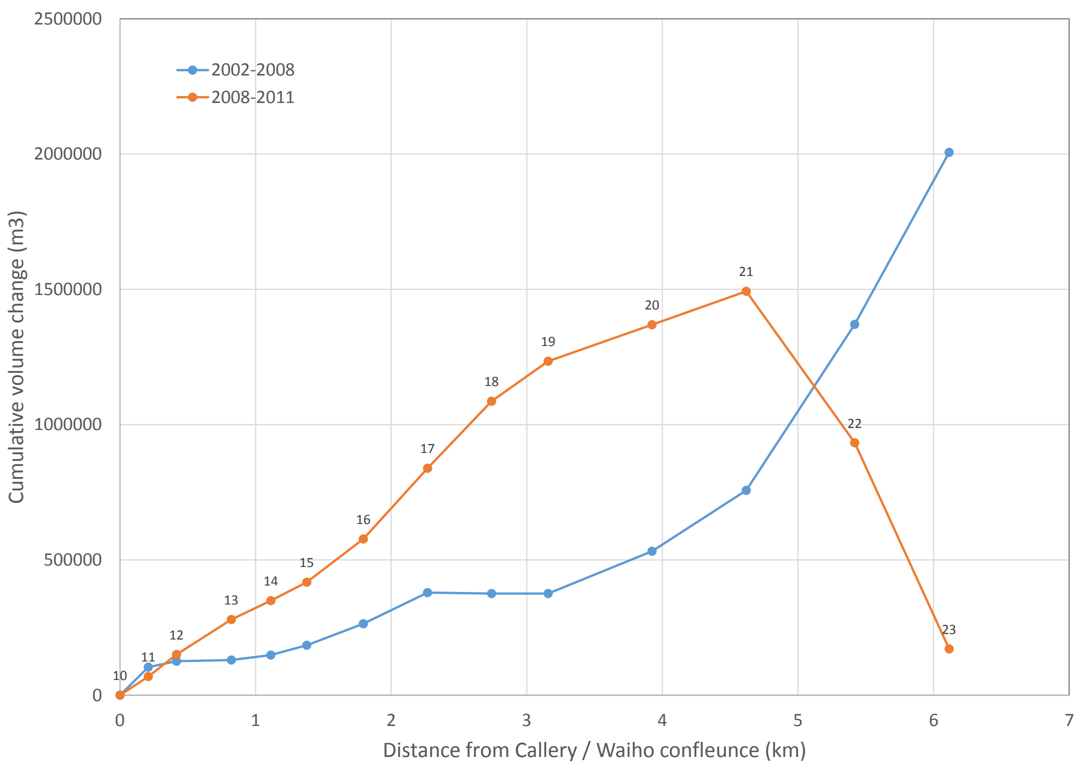

Figure 10: Cumulative bed volume change down Waiho River fan between 2002 and 2011 (cross-section numbers attached to each data point)

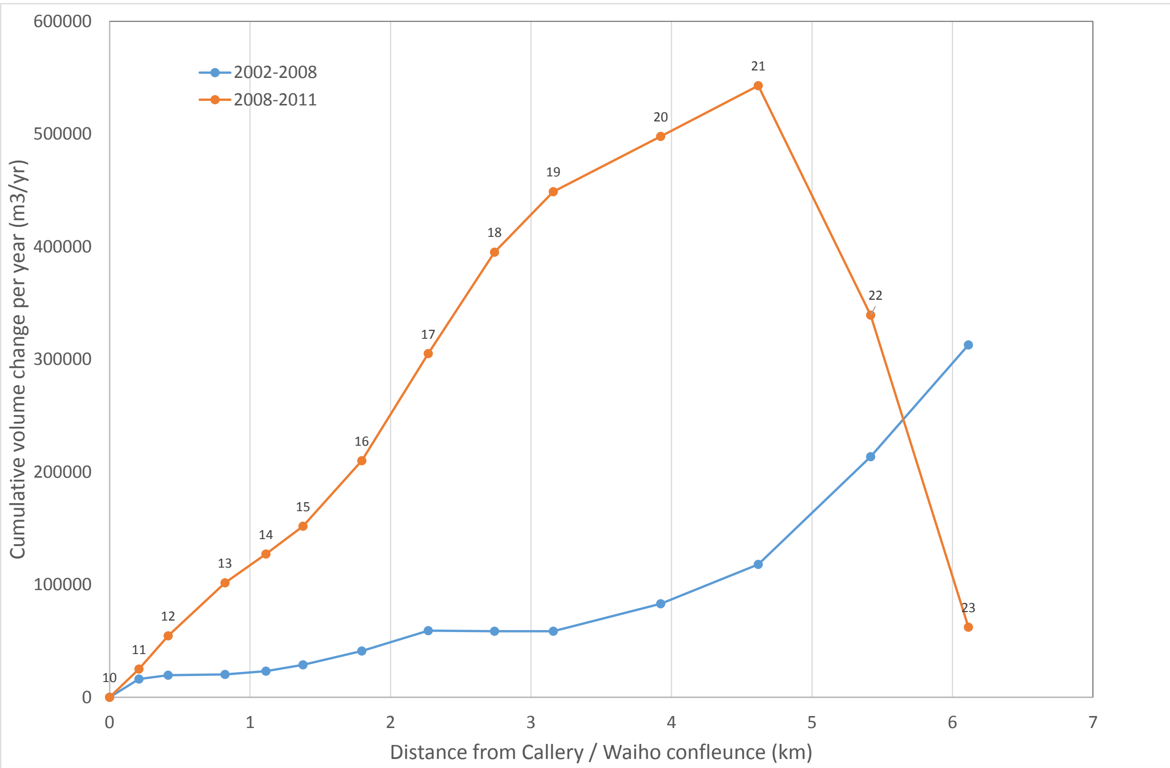

Figure 11: Cumulative bed volume change per year down Waiho River fan between 2002 and 2011 (cross-section numbers attached to each data point)

3.1.6 Fluvial Related Risks to Existing WWTP

In the light of the observed bed level trends on the upper part of the Waiho River fan, the location of the existing Franz Josef Village WWTP along the right side of the fan between cross-sections 18 and 19 makes the oxidation ponds extremely vulnerable to damage by large floods.

Damage could occur in either of two ways:

(a) flood-induced lateral erosion of the right bank adjacent to or immediately upstream of the ponds; or

(b) flood overtopping and channel avulsion.

Scenario (a) is considered less likely to occur than scenario (b). While the right bank adjacent to the oxidation ponds is heavily armoured with rock rip-rap as seen in Figure 2, there appears to be a gap in the bank protection between the start of this heavy rock armour and the end of the rock snub groynes installed further upstream. This gap, just upstream of the first oxidation pond, makes the right bank vulnerable to lateral erosion of a breach through to this pond.

However, the risk of flood overtopping and channel avulsion is considered to be a far greater threat to the oxidation ponds. Hall (2012) has already articulated this risk and identified potential overflow points where the right bank is lower immediately to the south (upstream) of the first oxidation pond (the access track running along the top of the right bank is markedly lower at this point and rises up as it goes past the two ponds), and further to the north (downstream) of the second pond. The risk arises from a number of factors: the low bank levels, the tilted shape of the fan directing flood flows towards the right side of the fan, and the significant aggradation trend on the fan surface with increasing bed levels. Figure 12and Figure 13 illustrate the effects of rising bed levels across the fan surface on existing right bank protection measures.

If the right bank immediately upstream of the first oxidation pond was overtopped, flood flows would flow northwards through the oxidation ponds and head across country towards the Tartare River. Suspended sediment and gravel bed material transported by the flood flows would fill the ponds. While this may only be a temporary and episodic occurrence initially, ultimately with ongoing deposition of sediment material on the fan surface, it could lead to permanent avulsion of river flows through the oxidation pond site into the Tatare River.

Hall (2012) rates the potential for avulsion towards the Tatare River either at this location or further downstream of the oxidation ponds as very high in the near future.

3.1.7 Quantification of Risks to WWTP

As part of his review of future management options for the Waiho River, Hall (2012) carried out a flood frequency analysis of the Whataroa River at SH6 bridge gauging station flow record and scaled the flood estimates based on a regional flood frequency approach to determine flood estimates for the Waiho River. The flood frequency analysis used a Generalised Extreme Value (GEV) EV1 frequency distribution. The results of the analysis are presented in Table 2 of his report.

Table 1 below extends the data presented by Hall (2012) to much higher frequencies and corrects the mean annual flood value given in his data table (the corrected mean annual flood value of 1446 m^3/s very closely matches the equivalent value obtained by Webby and Waugh (2003) using the same flood estimation approach but based on a Log Pearson 3 frequency distribution).

Figure 12: Partial submergence of snub groyne structures along right bank of Waiho River providing bank erosion protection upstream of Franz Josef Village WWTP

Figure 13: Partial submergence of toe of existing rock armour along embankment protecting oxidation ponds comprising Franz Josef Village WWTP

Table 1: Flood frequency estimates for Waiho River at SH6 Bridge (based on a GEV EV1 type frequency distribution)

Average Recurrence Interval (years)	Annual Exceedance Probability	Flow (m^3/s)
1.001	0.999	1156
1.01	0.990	1203
1.1	0.909	1279
1.2	0.833	1312
1.4	0.714	1353
1.6	0.625	1382
2	0.500	1422
2.33 (mean annual)	0.429	1446
2.7	0.370	1468
3	0.333	1484
4	0.250	1523
5	0.200	1550
10	0.100	1640
20	0.050	1730
50	0.020	1820
100	0.010	1910

GEMC (2008 and 2011) have used the cross-section data for the Waiho River fan to construct a one-dimensional computational hydraulic model in order to establish design flood levels down the fan for various left and right bank stopbanks. There are a number of criticisms of this approach:

(a) A one-dimensional model treats each cross-section as representing a single thread channel and does not account for flood flows across the alluvial fan surface being distributed amongst several different braid channels. For example, cross-sections 15 and 16 (refer to the cross-section over-plots in Appendix B) almost certainly have several braid channels across their entire width which convey flood flows as indicated by Figure 5 and Figure 6, and the aerial photo underlying the drawing in Appendix B.

(b) GEMC (2008 and 2011) assessed design flood levels for Waiho River fan based on Manning's n channel roughness values in the range of 0.030-0.040 (predominantly 0.037). As noted by Hall (2012), these hydraulic roughness values for a steep gravel bed river channel in flood are unrealistically low. They are more likely to be in the range 0.050 and 0.060 due to the effects of entrainment and transport of coarse bed material and "*the formation, translation, destruction and reformation of gravel bars etc.*" by flood flows.

(c) Even though GEMC (2008 and 2011) undertook sensitivity testing of their predicted design flood levels with increased channel roughness values (predominantly 0.044), their design flood levels are likely to be underestimates of actual flood levels.

Notwithstanding these criticisms, we have constructed a simple one-dimensional hydraulic model of the Waiho River fan between cross-sections 15 and 22 to estimate flood levels down the fan for different sized floods and for future bed level projections. The future bed level projections were based on the two average mean bed level change per year trends inferred previously in Section 3.1.5 from Figure 9:

(a) lower bound estimate – mean annual bed level change of 0.050 m/yr between cross-sections 15 and 21
(b) upper bound estimate – linear decrease in mean bed level change from 0.275 m/yr at cross-section 15 to 0.0 m/yr at cross-section 22.

In the case of both lower and upper bound bed level projections, the mean bed level at cross-section 22 (the pinch point on the Waiho Loop – refer to the drawing in Appendix A) was assumed to remain constant over time even though the braid channels shift around (refer to the cross-section over-plot in Appendix B).

For the purpose of the flood levels calculations, we first had to estimate March 2014 bed levels by projecting out from the last surveyed cross-sections in March 2011. Because the amount of sediment aggradation on the fan has been so dramatic in recent years, we only considered the effect of bed levels out to March 2019.

Figure 14and Figure 15 show stage/discharge rating curves at cross-sections 18 and 19 respectively for the following bed level scenarios for the Waiho River fan:

- Existing bed levels (March 2011)
- March 2014 bed levels – lower bound estimate based on scenario (a) above
- March 2019 bed levels – lower bound estimate based on scenario (a) above
- March 2014 bed levels – upper bound estimate based on scenario (b) above
- March 2019 bed levels – upper bound estimate based on scenario (b) above

The stage/discharge rating curves for cross-section 18 in Figure 14 are almost certainly overestimated as they do not reflect the presence of braid channels along the left side of the fan conveying a substantial portion of the total flow for each flood flow value.

The stage/discharge rating curves in Figure 15 for cross-section 19 are considered more realistic as cross-section 19 is predominantly triangular-shaped with the bulk of flood flows forced to flow along the right bank of the fan past this cross-section.

Hall (2012 has reviewed the design flood level estimates for the right bank by GEMC (2011) and determined that the freeboard is marginal along the right bank at cross-sections 18 for a 5% (1 in 20) annual exceedance probability (AEP) flood of about 1820m^3/s. The freeboard was also marginal downstream of the armoured bank around the perimeter of the oxidation ponds for cross-section 19 (refer to long-section G in Appendix B of GEMC (2011)).

With a more realistic Manning's n value of 0.06 for the "channel" down the alluvial fan, the freeboard for a 5% AEP flood would be even more marginal. We have assumed therefore that this flow value was the threshold for flood breakout in March 2011.

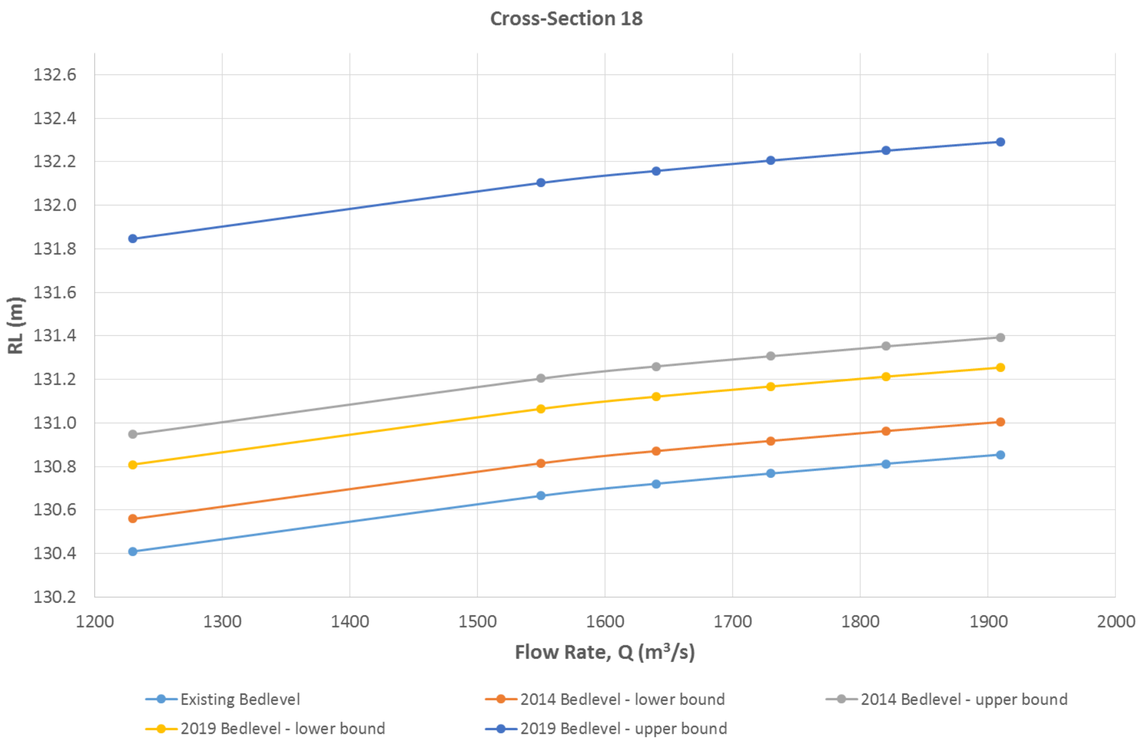

Figure 14: Estimated stage / discharge rating curves for cross-section 18 from 2014-2019

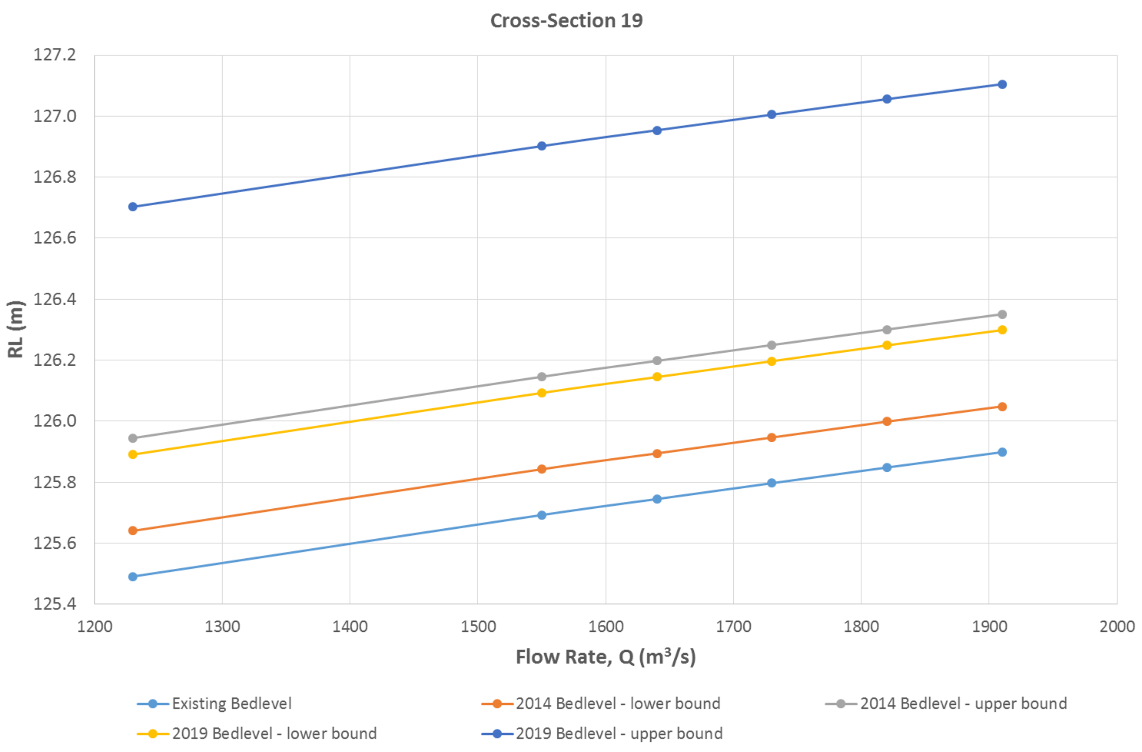

Figure 15: Estimated stage / discharge rating curves for cross-section 19 from 2014-2019

We have used the rating curves plotted in Figure 15 for cross-section 19 to determine at what flow the equivalent threshold flood level is reached for each of the lower and upper bound bed level projections to March 2014 initially, and then to March 2019. We have then linearly interpolated additional flow threshold values for each year between 2014 and 2019 and translated these flow values into annual exceedance probability values using the data from Table 1. Table 2 and Table 3 summarise the results of this process for the lower and upper bound sediment aggradation estimates for the alluvial fan.

Table 2: Estimated annual exceedance probabilities for right bank flood breakout through Franz Josef Village WWTP for lower bound sediment aggradation scenario on alluvial fan (2014-2019).

Year	Flow Threshold (m^3/s)	Average Recurrence Interval (years)	Annual Exceedance Probability
2011	1820	20	0.050
2014	1550	5	0.200
2015	1480	3	0.333
2016	1410	1.9	0.526
2017	1340	1.25	0.741
2018	1270	1.09	0.917
2019	1200	1.01	0.999

Table 3: Estimated annual exceedance probabilities for right bank flood breakout through Franz Josef Village WWTP for upper bound sediment aggradation scenario on alluvial fan (2014–2019)

Year	Flow Threshold (m^3/s)	Average Recurrence Interval (years)	Annual Exceedance Probability
2011	1820	20	0.050
2014	1150	1.001	0.999
2015			1.000
2016			1.000
2017			1.000
2018			1.000
2019			1.000

It is noted that the upper bound sediment aggradation scenario based on average bed level changes per year between June 2008 and January 2011 implies that the annual probability of occurrence of a right bank flood breakout is almost 1 for this current year (March 2014). As this did not occur in the recent November to March flood season, it suggests that this sediment aggradation scenario is probably not realistic for a long-term trend.

This is supported by evidence presented previously. We have previously noted that the appearance of the cumulative sediment volume curves for the June 2008 to March 2011 period in Figure 10and

Figure 11 is suggestive of a large sediment pulse moving down the upper part of the fan. The very significant aggradation that occurred in this period appears to be well correlated with the significant shifts in mean bed level seen at the site of the SH6 Waiho River bridge between January 2009 and January 2012 and in January 2011 seen in Figure 4.

Over the period January 2002 to June 2008 the mean bed level at the SH6 Bridge remained relatively constant so that the lower bound sediment aggradation trend on the alluvial fan is probably more indicative of the longer term trend. However the potential for an episodic sediment volume injection cannot be discounted which means that there is a risk that the lower bound estimates of annual exceedance probability for flood breakout could understate the true values likely to be experienced over the next five years.

Table 2 indicates that the annual exceedance probability for right bank flood breakout through the Franz Josef Village WWTP increases from 0.20 in March 2014 to 0.99 in March 2019. In other words the potential for flood breakout through the WWTP is an almost certainty by 2018-2019.

The implication for Westland District Council is that Council needs to proceed with planning for, designing and constructing a new WWTP on a different site as a matter of urgency.

3.1.8 Conclusions

The Waiho River alluvial fan has experienced accelerated aggradation of gravel bed material since January 2002, and particularly between June 2008 and March 2011.

The ongoing sediment aggradation trend is manifested by continually increasing bed levels across the width of the fan.

These continually increasing bed levels are superimposed on bed profiles with a distinct left to right tilt between cross-sections 16 and 21.

The left to right downward bed tilt trend is characterised by left side bed levels that are several metres higher than right side bed levels and also higher than right bank top levels.

The shape of the alluvial fan surface effectively directs braid channels on the fan surface below the head of the fan at the SH6 bridge towards the right bank immediately upstream of the oxidation ponds at the Franz Josef Village WWTP, and thereafter to hug the right bank till the Waiho Loop is approached.

The combination of a tilted alluvial fan surface, ongoing sediment aggradation and left side bed levels higher than right side bed levels and bank levels makes the right bank in the vicinity of the oxidation ponds highly vulnerable to flood breakout through the ponds to the Tartare River, with permanent avulsion of the river ultimately possible.

Hall (2012) has assessed the right bank freeboard along the right bank upstream and downstream of the oxidation ponds to be marginal for a 5% AEP flood of about 1820m^3/s for the March 2011 alluvial fan profile.

Using this flow threshold as a starting point and the January 2002 to June 2008 aggradation trend as indicative of the long-term trend, we have estimated that the annual probability of occurrence of flood breakout would increase from 0.20 in March 2014 to 0.99 in March 2019.

This implies that Westland District Council should be planning for designing and constructing a new WWTP facility on a different site as a matter of urgency.

3.2 Assessment of Fluvial Risks from Tartare River

3.2.1 Fluvial Risks from Tartare River

The most suitable location for an alternative WWTP facility has been identified to the northeast of Franz Josef Village on open ground on the left bank flood plain of the Tartare River (Figure 16). The treated effluent from the WWTP could be either discharged to ground via an infiltration trench system or discharged directed to the river through a diffuser.

Figure 16: View of Tartare River and floodplain looking upstream from SH6 bridge

The Tartare River has a much smaller catchment than the Waiho River and has no major sediment aggradation problems like the Waiho. Figure 17 shows a photo of the SH6 Tartare River Bridge in which the pile cap on each bridge pier is slightly exposed above river bed level. This suggests there has been a slight degradational trend of the river bed since the bridge was first constructed.

The only fluvial risks associated with the alternative WWTP site are:

- the potential for flood inundation if the plant is sited to close to the river
- localised bank erosion around any effluent discharge point

The first risk can be mitigated by siting the new WWTP well away from the river. The second risk can be mitigated by appropriate siting of any outlet structure and armouring the bank in the vicinity with rock riprap material to prevent the occurrence of lateral bank erosion.

Figure 17: View of bridge pier foundations on existing SH6 bridge crossing of Tartare River

The potential for permanent avulsion of the Waiho River into the Tartare River in the near future is considered a very real possibility. This would occur well downstream of the alternative WWTP so the latter site would be unaffected by this eventually.

3.2.2 Conclusion

The fluvial risks associated with the alternative WWTP site of the left bank floodplain of the Tartare River are minimal and easily mitigated.

3.3 Franz Josef Wastewater Discharges

Wastewater generation in Franz Josef Township includes contributions from residents, transient workers, overnight visitors and day visitors. The greatest wastewater demand occurs in the peak tourism period between January and March. Septage loads from hotels, residential properties and DOC huts are currently also discharged directly into the ponds.

Until recently the Franz Josef WWTP has had no inlet or outlet flow meters and therefore there is very limited historical data available on the flows to and from the plant. In January 2014 the recently installed outlet flow meter at the ponds recorded a total discharge of 4,478 m^3 (144m^3/d or approximately 600 persons equivalent population). This is very significantly less than the town's recorded potable water use of 20,951 m^3 in the same month. The water consumption appears reasonable in relation to the population numbers at that time of the year. This leads us to conclude that the outlet flow meter on the ponds may not be installed correctly.

Due to the lack of reliable recorded flow data we have used the flow and load figures estimated in the SKM report. SKM's estimates agree more closely with the water consumption figures minus and allowance for use outside plumbing systems. The key assumptions used in the SKM report were:

- Day visitors use 60 L/day of potable water, overnight visitors use 240 L/d.
- Day visitors produce 37.5 g BOD/day; overnight visitors produce 75 g BOD/ day.
- There is 1 worker to 4 visitors, and 50% of workers are seasonal.
- Visitor numbers were obtained from the GCDMP (2009).
- Franz Josef population figure were obtained from Statistics NZ Current and Projected Population (2006).
- A low growth rate (1.3%p.a) for the resident population, workers and visitors has been assumed until 2016, then a medium growth rate (3.4%p.a) has been used. The Glacier Country Destination Management Plan (GDCMP) issued in April 2009 considered a low visitor growth rate of 1.3%p.a. most likely for the region. To allow for the potential for tourism to increase strongly in the future SKM considered it prudent, in terms of assessing wastewater infrastructure, to allow for GDCMP's estimated median growth rate of 3.4%p.a. after 2016.
- The majority of visitors arrive in the peak months of January, February and March.

The Council has no specific data on the septage, DOC hut or camper van discharges into the existing plant. SKM and G2e excluded these flows in their reports. We have consulted with Craig Neiman from Hibbs Drainage who is the main septic tank service provider for the Franz Josef area. Craig services the Franz Josef area 3 times per year and estimated that he discharges approximately 30 m^3 of septage each visit into the ponds. As we have no sampling data for the septage loads we have used typical septic waste concentrations based on average septic load concentrations in NZ.

For the purpose of this report Opus have made the following assumptions:

- Average day wastewater flow from the town has been calculated as 20% higher than the ADWF
- New rising mains have been sized using twice the ADWF
- Discharge disposal systems have been sized for flows 50% higher than the ADWF flow, with flows in excess going to the Tartare Stream via a pipe and diffuser.
- The disinfection system has been sized assuming a 3hr PWWF event + the base ADWF

Table 4 shows the loads and flow estimates based on the information provided and derived:

Table 4: Projected Wastewater and Septage Flows and BOD Loads

	2006	2011	2016	2021	2026	2031	2036	2041	2044
Population Equivalent	2,006	2,127	2,256	2,611	3,028	3,510	4,088	4,761	5,233
ADWF[1] (m^3/day)	409	434	460	533	618	716	834	972	1,068
1.2 x ADWF (m^3/day)	491	520	552	640	742	859	1001	1166	1281
1.5 x ADWF (m^3/day)	613	650	690	799	927	1074	1251	1457	1601
PWWF[1] (m^3/day)	1,227	1,301	1,380	1,599	1,854	2,147	2,502	2,914	3,202
PWWF 3hr storm (m^3/hr)	290	307	326	378	438	507	591	688	756
BOD[1] (kg/day)	160	170	180	209	242	281	328	382	420
Septage Loads[2] (m^3)	30	30	30	30	30	30	30	30	30
Septage BOD[2] (kg)	210	210	210	210	210	210	210	210	210

[1]Estimated flow and loads discharged into the ponds via the wastewater main from Franz Josef Township

[2]Estimated flow based on HIBBs Drainage estimate of 30 m^3 septage/visit discharged into the ponds over a 2 day period 3 times per year

[3]Estimated Assuming 3 hr PWWF event, subtracting the usual ADWF flow

4 Option 1: Upgrading Existing WWTP

4.1 Description

At the inception of this report Option 1 was proposed as an upgrade of the existing plant to achieve compliance with the RC discharge concentrations and to make other basic improvements such as the proper handling of screenings and septage. Recognising that the Waiho River was likely to inundate or outflank the plant, this would have been considered an interim phase of works, to be followed by construction of a new WWTP (details outlined in Option 2) prior to Waiho River flooding becoming a critical threat. Our initial thinking, prior to any river analysis, was that this time frame could be in 10 years' time.

As discussed in Section 1.3 the results from the river engineering assessment predict that there is a high probability that the existing plant will be inundated by the Waiho River within 5 years. Option 1 is therefore no longer considered a feasible Option.

In the event that West Coast Regional Council (WCRC) stipulates that the existing plant discharge water quality must be improved whilst the new plant is built, projected costs (per G2e) for potential upgrade have been reiterated. We have obtained pricing for other preferred upgrade requirements (which are available if required) but have currently not stated these due to the, now understood, criticality of a change of site.

The following, Section 4, discussion is applicable should an interim plant improvement programme be enforced between the report date and implementation of a new treatment plant. Discussions will be required with regional council to confirm / negotiate minimum upgrade requirements.

4.2 G2e and SKM Proposed Upgrades

When G2e assessed the existing plant there were no flow meters measuring the flows into and out of the ponds. G2e was therefore required to base their analysis on the loads and flows that were estimated in the SKM report. These estimates were based on population and visitor numbers and equivalent concentrations and flows.

The G2e report found that the existing plant is operating on the brink of its maximum treatment capacity for the estimated 2013 load (800 PE in summer). RC compliance limits for BOD_5 and Am-N are only expected to be achieved during times of ideal weather and loading conditions. The report also noted that the plant's average inflow of 500 m^3/day is close to the RC maximum discharge allowance of 600 m^3/day. We can expect that the plant routinely discharges over the compliance limit and this will worsen as the population increases.

The flow and load estimates from the SKM report did not include load data for any additional loads, such as septic tanker discharges, camper van loads and DOC hut wastes. The G2e report acknowledges that as these additional loads would be very concentrated and add significant waste loads to the plant this lack of data and information reduces the effectiveness of the reports analysis.

G2e recommended that before any long-term upgrade strategy for the ponds is confirmed the following data should be collected:

- *24 hr flow meters should be installed at the plant's inlet and outlet and on-going flow records should be compiled* – An outlet flow meter has already been installed since the G2e report was released. The readings from the flow meter are suspect and it may require calibration.

- *An intensive load-monitoring period should be implemented to monitor the contaminants to the plant for at least one month during the four seasonal periods* – This has not yet been implemented. It is critical for this to be completed soon.

- *A data collection and sampling program should also be implemented for all waste loads entering the plant in addition of what comes in through the inflow pipe e.g. septage or DOC hut discharges* – This has not yet been implemented. It is critical for this to be completed soon.

In our opinion, these three recommendations should be implemented as a matter of urgency. The information received from these steps will be critical for the determination of developmental parameters of a new WWTP.

4.3 Existing WWTP Compliance Limits

The existing Franz Josef WWTP operates under a 35 year Resource Consent which is valid until 2036. Since 2006 sampling of the pond discharges reveal that the concentrations of BOD, SS, NH_4-N and faecal coliform rarely meet their required consent conditions. SKM considered the RC conditions to be too stringent for a two-pond system and argued that the consent conditions could be relaxed. Table 5 shows the existing and proposed RC conditions that the median annual concentrations are not to exceed:

Table 5: Existing and Proposed RC annual median consent conditions

Parameter	Unit	Existing	Proposed
Biochemical Oxygen Demand	mg/L	30	40
Suspended Solids	mg/L	30	60
Ammonia Nitrogen	mg/L	15	20
Faecal Coliforms	cfu/100mL	10,000	75,000

The RC also notes that the total daily volume of wastewater shall not exceed 600 m^3. As discussed in section 4.1 we expect this flow to be exceeded on a regular basis.

4.4 Existing Plant Upgrades

4.4.1 Upgrading Options

G2e and SKM have provided extensive recommendations for the upgrading of the existing WWTP. These suggestions include constructing additional ponds, installing submerged and surface aerators and desludging the ponds. To implement all of these recommended upgrades a large injection of capital would be required, however we recognize that as the existing plant life is limited to a predicted maximum of 5 years such a large investment may not be prudent.

The following potential upgrades for the ponds have been ordered from the likely most effective in terms of both cost and treatment to the least effective. These upgrades should improve the discharge quality for the plant. These recommendations are based on the assumption that the

West Coast Regional Council will grant approval to change the existing RC conditions to those outlined in section 5.1.

Desludging the Ponds

Accumulated sludge in oxidation ponds reduces the effectiveness of the ponds and their ability to treat wastewater loads. We recommend that a sludge and pond survey is conducted on the ponds and that the ponds are then desludged.

Desludging costs would be considerable and affected by the amount of desludging that is required. Previous estimates from the G2e report indicate that there could be around 2,500m^3 of accumulated sludge to be removed. A sludge survey of the ponds would be required to determine how much sludge would need to be removed to improve the treatment capacity of the ponds.

Modification of Outlet Exfiltration Gallery

Reconstruction of the rock based disposal diffusion system to ensure coloured effluent flow does not exit at the river bed surface but rather dissipates out and down to join the subsurface flows. We note that such a system is difficult to maintain when there is any significant silt flow in the river. Water velocity essentially drops to zero in the void spaces between the rocks, allowing silt to rapidly settle out and fill the voids. However, potentially, a heavily armoured gallery, with a strong geotextile separator layer above the gallery proper, has a chance of success for a sufficient length of time.

This operation would involve construction within the Waiho River.

Modification of Existing Wetland System and Baffle Curtains

The existing wetland system installed upon Pond 2 appears to be having a minimal effect on the water quality treatment of the plant's effluent. From observation of the site we believe that the existing layout of the wetland system probably allows the inlet flow to bypass the wetlands and travel almost directly to the outlet due to ineffective baffle curtains. This could be tested with some reasonably basic dye testing. We recommend that the wetland rows are extended to full pond width, and curtain integrity improved, to minimise this short circuiting effect and to provide a greater treatment capacity efficacy in terms of TSS and FC reduction in Pond 2. Additional baffle curtain may be required to be installed within the ponds.

Modification of Inlet Pipework – Inexpensive Option

The existing inlet pipe extends several metres into Pond 1 and discharges the effluent directly towards the centre of the pond. This promotes short-circuiting and an overall reduction of treatment as some of the flow will short-circuit towards the pond outlet.

A simple modification of the inlet pipework and the possible installation of a non-mechanical screen could reduce the issue of short circuiting of flow within Pond 1 and remove solid matter before it is discharged into the ponds. However, this would also increase operational inputs and may not be justifiable

Disinfection of Effluent

With the current pond system the faecal coliform discharge concentrations are regularly above the existing and proposed RC conditions. The installation of a UV disinfection system at the pond outlet would allow for additional treatment of the outlet flow and would provide an effective, if somewhat costly, solution to achieve the RC FC conditions.

UV Disinfection performance is affected by solid particles and dissolved organic material in the effluent. Because of the typically elevated solids levels in pond effluent, a UV disinfection system treating this effluent would not typically be effective at reliably delivering much more than 1 log_{10} inactivation of faecal indicator bacteria.

Installation of a Septage Receiving Facility

Approximately 30 m^3 of septage is released into the ponds over a 2 day period three times a year. These septage loads are discharged directly into the western corner of Pond 1. These loads are highly concentrated and will therefore likely to be incompletely treated before reaching Pond 2, resulting in the plant's overall discharge standards not being met.

We recommend that, at least, a rudimentary septage receiving facility is installed. This receiving facility would feed the septage loads to the ponds in a small steady dose over a longer period of time. The septage facility could either be an underground tank that has the septage pumped to the ponds, or an aboveground tank with a gravity feed pipeline to the ponds. Both tanks may require some type of mixing unit installed.

Surface Aerators

One of the main capacity limitations of the treatment plant lies in the plant's reliance on the sole use of wind action and algae to provide oxygen to the pond. The installation of two 5.5kW surface brush aerators in Pond 1 would significantly increase the treatment capacity of the ponds by providing supplementary aeration and more complete mixing of the first pond.

Modification of Inlet Pipework – Expensive Option

The treatment plant influent is currently entirely unscreened. This already makes the treatment system unsightly. If mechanical devices were to be installed, it would be necessary to install screening. The inlet pipework would be modified to allow the inlet flow to pass through a stainless steel inlet chamber which will be equipped with a mechanical screen. This screening system would remove solid matter within the wastewater before it is discharged into the ponds, improving the performance of the ponds.

4.4.2 Recommended Upgrades (if pond use was to be continued)

The desludging of the ponds and the installation of a disposal diffuser system both require significant capital costs, however both of these upgrades require once-off capital costs and do not require the installation of a power supply to the ponds or continuing operational costs. Overall the treatment and compliance benefits that would be provided by these upgrades make them a more efficient and cost beneficial upgrade for WDC to implement.

The modification of wetlands, curtains and inlet pipes (non-powered option) should improve the circulation of the effluent water in the ponds and therefore improve the treatment capacity of the ponds. These three upgrades would probably be less effective than the desludging of the ponds, however they would be low cost methods to implement and would require minimal operational costs.

If the septage storage facility can be installed with a gravity feed pipe discharging septage to the ponds and no electricity is required for pumping we would recommend that this option is implemented. If electricity for a pump is required this system would probably have limited value for the associated potential gains to water quality.

The surface aerators, UV System, mechanical screen and septage receiving facility are all expensive options and would require the development of additional infrastructure (power lines, transformer, MCC units) to the ponds. Although they would all be effective in increasing the treatment process of the plant, these may not be considered suitable for the small timeframe that they would be operational.

The UV system and the mechanical screen could be relocated to the new WWTP once the new plant was to become operational, however they run the risk of being washed away, being severely damaged by flooding from the Waiho River or being of sub-optimal configuration for the new plant. Relocation of these items would also require additional capital costs.

4.4.3 Cost Estimates for the Upgrades to the Existing Plant

A breakdown of the capital cost estimate for the upgrades to the existing ponds can be seen in Appendix C. The cost estimate was developed by applying likely minimum and maximum values to the quantities and rates to the relevant items and running an @Risk, Monte Carlo simulation to calculate the statistically likely costs for the total of the work.

The preliminary cost investigation estimated that a median total capital costs of $600,000 and a 95th percentile costs of $750,000, would be required to implement the upgrades to the existing pond recommended in Section 4.4.2. This price estimate includes the sludge survey, desludging costs, modifying the outlet and inlet pipes (inexpensive option), wetland system and baffle curtains.

An estimated median capital cost of $480,000 and a 95th percentile capital costs of $500,000 would be required to implement the other upgrades outlined in Section 4.4. This capital cost includes the $40,000 costs required to provide the existing site with 25kva of power. This does not include the costs to relocate the UV system and the Inlet Screen to the new WWTP.

The estimates make provision for design, management of the works and for contractual preliminary and general costs. Contingencies are provided for by the Monte Carlo analysis on each item.

5 Option 2: Building a New WWTP

5.1 Description

In Option 2 the existing plant is to be abandoned and a new high rate treatment plant is to be built in a different (safer) location as soon as possible. In determining a cost profile for this option, we have assumed that the new plant will be operating by 2017 at the earliest. The existing plant will continue to operate until the new WWTP is running. For the purpose of the cost analysis of the new WWTP we have assumed that the existing plant will not require any upgrades.

As we are still in the conceptual phases of the plant design we have considered a range of for unit processer and construction types in order to build up a realistic picture of the range of likely capital cost. These are:

- Whether the plant will discharge into a rapid infiltration disposal field or to the Tartare River through a disposal diffuser.
- Whether the plant will have an earth based reactor or a concrete based reactor.
- Which site the new WWTP will be constructed on. This impacts the land purchase values, the subdivision and resource consent costs and the costs of installing support infrastructure such as electrical supply, rising mains and wastewater pump stations.

5.2 Proposed WWTP Development Timeline

As indicated in Section 3.1.8 the existing plant may be inundated by the Waiho River anytime within the next 5 years. We strongly recommend that WDC begin to initiate the planning and development of the new WWTP forthwith. An indicative timeline schedule for the new WWTP is shown in Appendix D. We estimate that it could take WDC approximately 2 years from the commissioning of a conceptual plant design until the plant is operational. This would include strong caveats on finance availability and planning provisions. The timeline we have evaluated includes:

- Conceptual Design – 2 months
- Consenting Approvals for Disposal, land use and designation – 8 months
- Detail Design – 3 months
- Procurement, Tendering – 6 weeks
- Procurement, Evaluation and approvals – 6 weeks
- Construction – 12 months
- Commissioning of plant – 2 months

5.3 New WWTP Site

We estimate that the new WWTP will need a land size of approximately 1.5 ha for the plant buildings and treatment processes. If the plant has a disposal diffuser system we estimate that it will require an additional $150m^2$ and the final stage of a rapid infiltration disposal field would require an additional 1.6ha. We have identified four potential sites for the new WWTP. The locations of these four sites can be seen on Figure 18 and the advantages and disadvantage of each site is outlined in Table 6.

Table 6: New WWTP Potential Development Sites

Site No #	Property	Advantage	Disadvantage
1	South side of the Callery Holdings Land	- Close to existing wastewater main and power lines - Not located near any hotels or residences	- Vulnerable to movement of Tartare Stream - Will require pump station
2	North side of the Callery Holdings Land	- Close to existing power lines - Not located near any hotels or residence	- Will require pump station
3	DOC Land	- Not located near any hotels or residence - Provides a land purchase option near the Tartare Stream	- Will require DOC permission and additional permits - Will require pump station
4	Fulton Land	- May require less pipework constructions - Will not require a pump station - If discharges into the Waiho River may require less strict consent conditions than the other 3 sites which will discharge to the Tartare St	- Could receive complaints from nearby Top 10 Holiday Park and school - May require additional power lines to be constructed - Quite far from the Tartare Stream so may discharge into Waiho River. This leaves the disposal system vulnerable to the movements of the Waiho River.

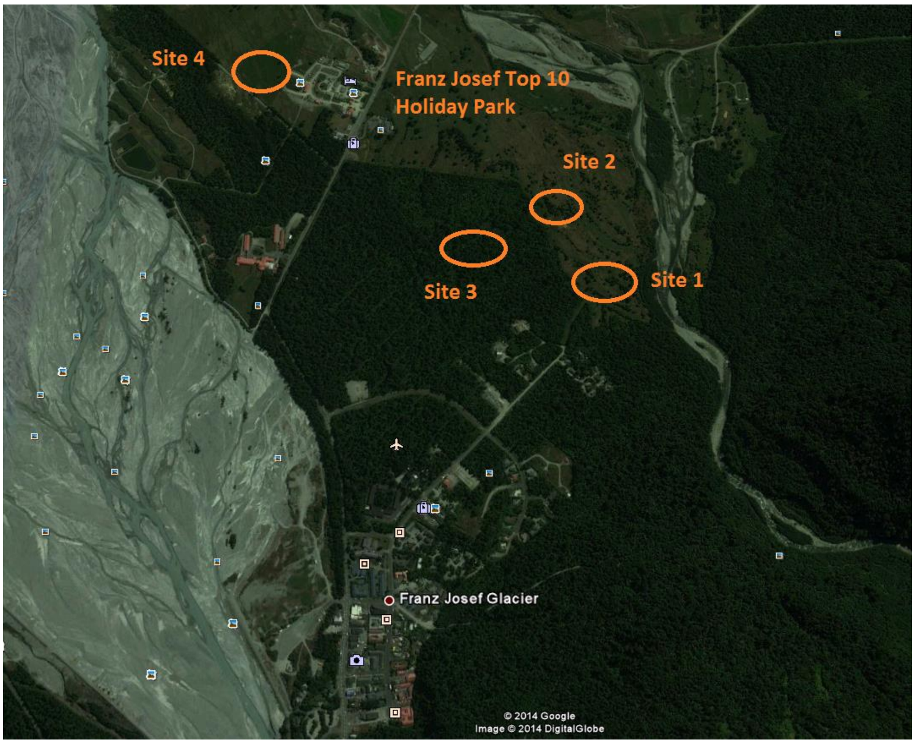

Figure 18: Potential Sites for new WWTP

5.4 Compliance Limits

We have assumed that the RC compliance limits for the new WWTP will be lower than the existing limits. We have therefore performed our preliminary cost estimate on our new WWTP using the following discharge compliance limits:

Table 7: New WWTP Plant 90th Percentile Discharge Limits

Parameter	Unit	Proposed
Biochemical Oxygen Demand	mg/L	20
Suspended Solids	mg/L	20
Ammonia Nitrogen	mg/L	2.5
Faecal Coliforms	cfu/100mL	200

5.5 Infrastructure Requirements

5.5.1 Wastewater Mains and Pump Stations

If the WWTP is constructed on Sites 1, 2 or 3, WDC preference is for the wastewater flow from the town to be diverted from the existing gravity main that conveys the flow from the town to the ponds to a new gravity main that would be constructed along SH6. This could transport the flow to a new pumping station, located near Top Ten, which would pump the wastewater to the new WWTP via a new rising main. Figure 19 shows the proposed pipeline route to the new WWTP. Cost estimates have been provided for the new gravity main, rising main and pump station.

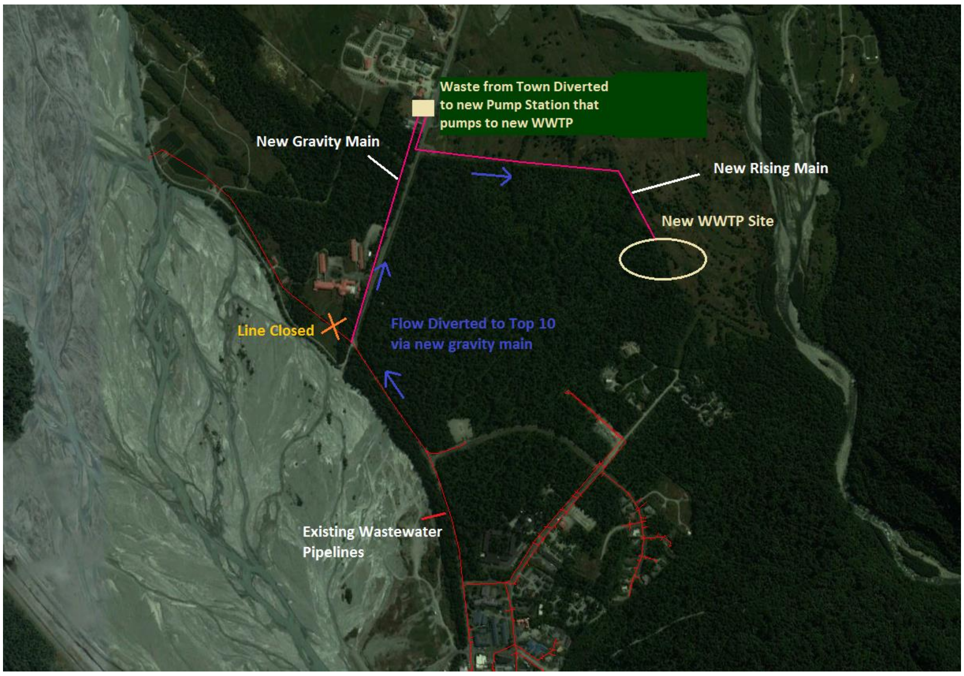

Figure 19: New Wastewater Mains and Pump Stations for Sites 1, 2 and 3

Alternatively the existing pump station (shown on Figure 20) could potentially be used to pump some of the town's wastewater directly to the new WWTP. The remaining wastewater would be routed down the same route as in Figure 19, however a smaller pump station would be required. For the purpose of this report the route determined on Figure 19 will be used, however we recommend that once the exact location of the new plant site is determined WDC analyses, in detail, which route would be more cost effective.

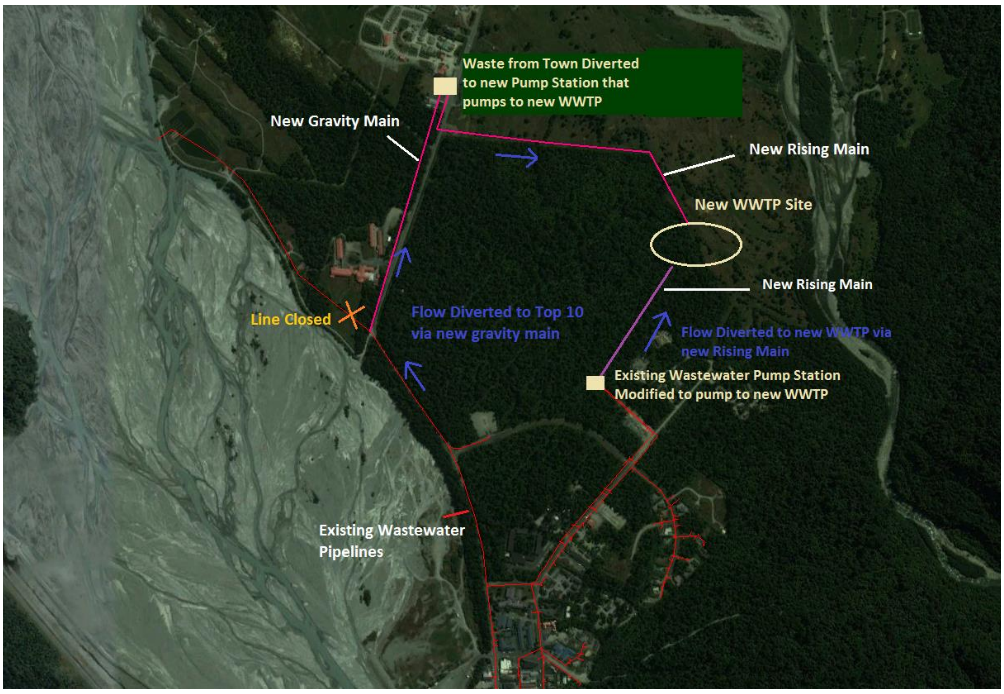

Figure 20: New Wastewater Mains and Pump Station with Modified Existing Pump Station for Sites 1, 2 and 3

If Site 4 is selected then the new gravity main would turn left from SH6 at the Top 10 Holiday Park and flow to the new WWTP site behind Top 10. The entire pipeline route would travel downhill and therefore would not require a pump station (Figure 21).

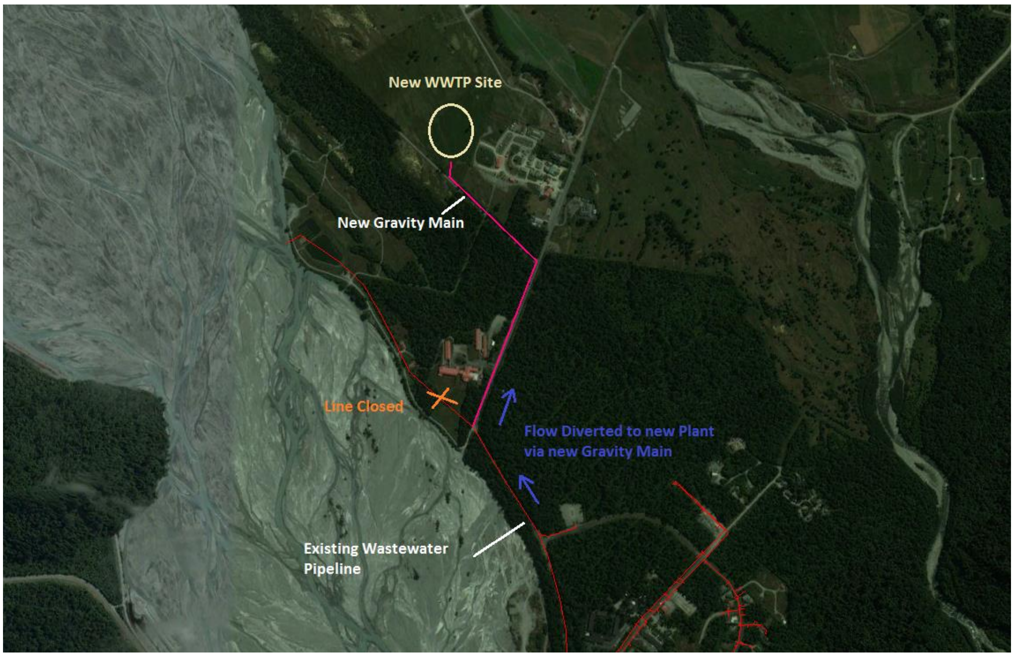

Figure 21: New Wastewater Mains for Site 4

5.5.2 Road Access

There appears to an unformed road that could provide for access to Sites 1, 2 and 3 off the end of State Highway 6. The legal status of this road is uncertain. It would not provide for the full length of access required. This road would require proper formation to allow for construction traffic and the passage of the septage trucks and Council service vehicles.

An access road to Site 4 does not exist and would probably travel along the southern end of the Top 10 holiday park.

The cost of constructing a road to access the site has been included in the capital cost estimate for the new WWTP. A low cost site access option (unsealed road) and a high cost site access option (road has a 2 coat seal and kerbs) have been analysed. Cost required to maintain the roads has not been included in the NPV analysis.

5.5.3 Power Line

Sites 1, 2 and 3 are all located next to an existing high voltage power line. The West Power (Electronet) power company provided an estimate of $25,000 to provide this site with the required 320kva of power. Site 4 was located away from any nearby power poles and will require $37,000 to connect this site to power. A further $15,000 would be required to supply 15kva to the new SH6 waste water pump station required for sites 1, 2 and 3.

These estimates from Electronet do not cover any easement or consents which may be required, nor any internal wiring or metering of any site.

5.6 New Plant Costs

Cost estimates for the purchase of the WWTP equipment were achieved with a combination of supplier cost estimates and cost estimates from similar sized plants. Power law, flow based scaling, has been used where appropriate.

For the purpose of compiling a preliminary cost estimate the following assumption have been made:

- Sites 1, 2, and 3 have all been assumed to be in roughly the same location
- The disposal system has been configured for the Site 1, 2 and 3 location only. Site 4 would require further investigation into a suitable location for disposal.
- That, for the eventuality of disposal to land, the underlying alluvial fan material can sustain soakage rates for secondary treated, tertiary filtered effluent of 100mm per day.
- The UV system will be installed in 2 stages and will be designed for the PWWF assuming a 3 hour storm event plus the base ADWF flow. Stage 1 will be designed for the 2024 flow (411m^3/hr) and will require 1 channel with 24 lamps. Stage 2 has been designed for the 2044 flow (750m^3/hr) and will require the addition of another channel with 24 lamps.

5.6.1 Capital Expenditure

5.6.1.1 Plant Construction Costs

The option of building a new plant presents a range of capital costs that are associated with the construction of a complete plant, including the cost of a new delivery pump station and a rising main to the site.

The treatment plant proper has been conceptualised to include the following:

- Mechanical screening and vortex grit removal arrangement for the inlet works,
- Anoxic and aerated reactors with a recirculation pump,
- Clarifier (batch reactor technology would also be a real option)
- Return mixed liquor (RAS) and mixed liquor wasting (WAS) pumping systems,
- Solids management system consisting; mixed WAS tank, Thickening / decanting tank, dewatering centrifuge
- Tertiary filtration,
- Effluent disinfection system,
- Motor control centre (MCC),
- Programmable Logic Controller (PLC) based control system,
- A building to house MCC and control, air blowers and sludge dewatering, and
- An effluent disposal system

Two different treatment plant reactor options have been considered for our cost benefit analysis; a plastic lined earth based reactor and a concrete reactor. The concrete reactor is more expensive, however it is a smaller reactor which will require less space and will have a significantly longer life before a major overhaul is required.

5.6.1.2 Land Purchase Costs

As discussed in Section 5.1 four sites are being considered for the new WWTP. Land purchase prices for large allotments of land were provided by the Council. As we assume the WWTP will require approximately 1.5ha of land for the plant, with a possible 1.6ha required for the disposal field.

WDC provided Opus with land valuations for the four sites. These land valuations were for properties between 13ha and 125ha. The purchase price for the required land has been estimated by breaking down the land prices to per hectare cost. For a low cost estimate we added an additional 30% to the per hectare cost, and for the high cost estimate we added 100%. We decided to add the conservative high cost estimate due to the limited number of suitable potential plant sites and because some of the landowners are on difficult terms with WDC, which may negatively affect price negotiations. We have assumed costs for subdivision fees and resource consent fees, of which the DoC land will have the highest.

5.6.1.3 Disposal System

Two disposal systems have been considered for this report; a disposal field system and a disposal diffuser system.

The disposal field system is the high cost option, however it would be highly effective in dispersing the discharge flow from the treatment plant. We have sized the disposal system using 1.5 times the ADWF flow. The disposal system would be constructed in stages, however it would be preferential if WDC purchased the land required for the 2044 flow when purchasing the land for the new WWTP. In the NPV analysis we have assumed the disposal field is installed in three stages; for the 2021 flow (0.8 ha), the 2031 flow (1.1ha) and the 2044 flow (1.6ha).

For the preliminary sizing of the disposal field we have assumed that the land area for the effluent discharge will be free draining. The percolation rates may vary between 40 and 200 mm/day, however we have assumed 100 mm/day for our calculations. Costing has been based on a Low Pressure Effluent Distribution (LPED) system. This type of system works well in soils with a high percolation rate, and soil analysis of the discharge area will be required before the system is constructed.

The LPED system comprises of a low pressure pipe with discharge orifices along its length, that is nested inside a slotted drain pipe (refer to Figure 22). The low pressure pipe evenly distributes effluent along the entire length of the drain pipe. The effluent in the drain pipe then passes into the trench aggregate prior to entering the surrounding soil.

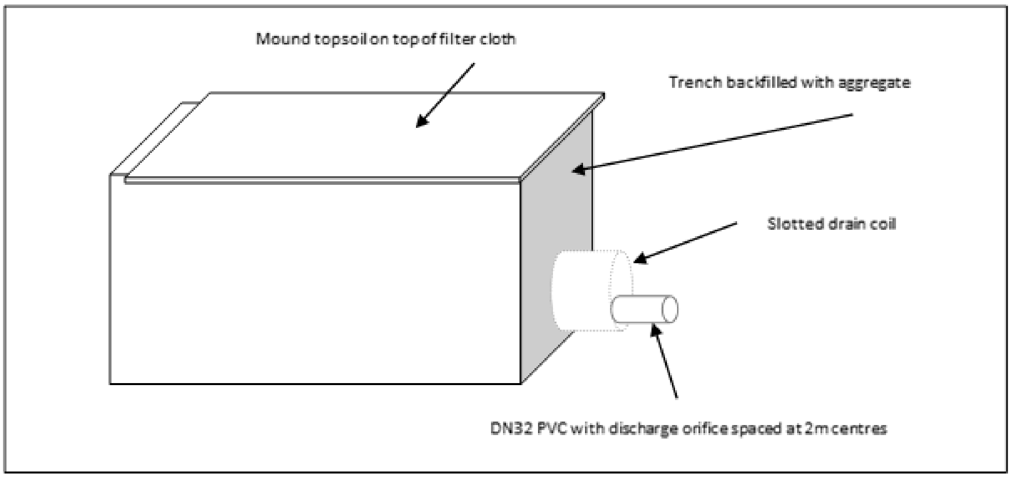

Figure 22: Diagram of the LPED system showing nested pipe arrangement

Installing a river edge disposal diffuser system would be the low cost option. We have assumed the diffuser system will discharge into the Tartare Stream. To construct the diffuser system an area approximately 2m deep and 4m wide would need to be excavated. The length of the diffuser would need to be determined after further consultation. 1500mm of 300mm diameter rocks would be laid out on top of a geotextile lining. Another geotextile lining would be installed over the rock layer and the existing ground surface would be reinstated. A 200mm diameter pipeline would discharge treated water from the new WWTP into the rock layer.

Large boulders (approx 1.5m diameter) would be installed at the outlet of the diffuser system to prevent erosion and stop the smaller rocks from being washed away.

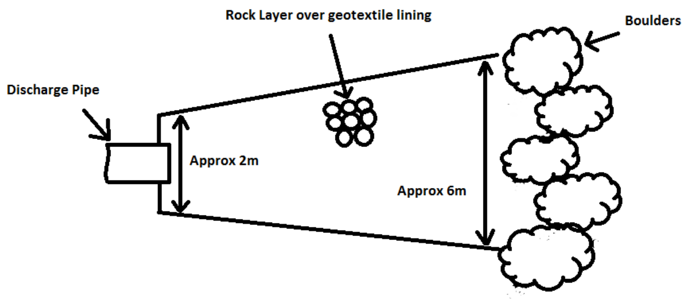

Figure 23: Estimated requirements for Disposal Diffuser System

Figure 24: Example Rock Diffuser Discharge System under Construction (River at Right)

5.6.2 Operational Expenditure

The general makeup of the operational cost included in the cost benefit analysis for the new WWTP are:

- The annual power consumption costs associated with operation of the treatment plant and the pumping cost from the existing reticulation through a new 1045m long rising main (for sites 1, 2 and 3 only).
- Labour costs comprising of two operators working four hours a day.
- Chemicals required for alkalinity replenishment.
- Supplementary rbCOD for nitrogen removal and polymer for sludge dewatering.
- Screenings, grit and sludge transportation and disposal.
- It has been assumed that 24 UV lamps will require replacement every year until 2031, when the UV system will be upgraded and 48 lamps will require replacement.
- Average flow rates (1.2 x ADWF) have been used for any operational costs dependant on the wastewater inflows into the plant.
- Annual maintenance costs have been applied based on 3% of the assessed mechanical and electrical portion of the plant cost.
- Annual inflation has been applied to cost rates and
- A cost of capital of 6.5% has been applied as the Present Value (PV) discount factor.

5.7 Capital Cost Analysis for the new WWTP

A breakdown of the capital cost estimate for the construction of the new WWTP and associated pipework and pump station construction can be seen in Appendix E. The cost estimate was achieved by applying minimum and maximum values to the quantities and rates to individual items when relevant. An @Risk simulation was run to calculate the statistically likely costs for each item. This methodology allows for the uncertainties in quantities and prices for individual items.

To estimate the capital cost for the new WWTP three key costing variables that were considered were:

- The location of the new site
- The type of reactor
- The type of effluent disposal system
- The sludge disposal method

The cost investigation found that the high and low cost options were:

- High Cost: Site 1, concrete reactor, disposal field system
- Low Cost: Site 4, earth works reactor, disposal diffuser system

The median capital cost required for the new delivery system, WWTP and disposal system is $8.9 million (this cost is in current dollars). The estimated 95th percentile capital is $9.5 million.

The type of disposal system was the most significant variable for the cost estimate. Constructing the new WWTP on site 4 does reduce the pumping and site access costs to the new WWTP, and this will also reduce the operational costs for the plant. However, Site 4 is located quite far from the Tartare Stream and this may significantly increase capital cost.

5.8 Operational Cost Analysis for the new WWTP

An NPV analysis for the first 30 years of the new WWTP's life estimated a median NPV of $23.25 million and a 95th percentile NPV of $24 million for the new WWTP.

The NPV included the following capital costs:

- The upgrade of the UV system in 2024
- The upgrade of the disposal field in 2030
- The addition of a second clarifier in 2031
- A mechanical renewal allowance at year 20 (2037) as we can assume that pumps, centrifuges etc. will be worn and will require significant overhaul or replacement
- The upgrade of the disposal field in 2041 to increase capacity.

Significant operational costs were found to be the power consumption of the plant and the workforce costs. The purchase of chemicals and the costs involved with the disposal of sludge were also significant.

6 Discussion and Summary

Results from the river engineering analysis estimate that the annual probability of occurrence of flood breakout of the Waiho River to engulf the existing plant will increase from 0.20 in March 2014 to 0.99 in March 2019. This implies that Westland District Council should be planning for designing and constructing a new WWTP facility on a different site as a matter of urgency.

WDC will need to consider if it is prudent to invest in upgrades to the existing plant which is at risk of failing any time within the next 5 years. However if planning for the new WWTP commences immediately it will most likely be at least another 2 years before the new plant is operational. This means that the existing plant will continue to discharge non-compliant water into the Waiho River over the course of this time. Possibly non-powered upgrades such as the dredging of the ponds and the installation of an outlet diffuser system will be required to improve discharge water quality. If investment in a new plant is underway, a solution may be able to be negotiated with regional Council to avoid 'sunk cost' upgrades in the short term at the existing site.

For the new WWTP the proposed sites 1, 2 and 3 will have similar costs. Site 4 will require less pumping of the towns wastewater and will require less pipeline construction however it is located quite far from the Tartare St and this may affect the disposal costs. Site 4 is still located on the Waiho 'fan' (retaining some residual inundation risk) and is also located close to a popular Top 10 Holiday Park and a school. These circumstances may adversely affect WDC's ability to achieve approval to construct in this location.

Prior to constructing the WWTP WDC will need to determine:

- The site for the WWTP
- The disposal method for the plant
- The Resource Compliance Conditions which will be set by WCRC
- How the town effluent will be conveyed to the WWTP
- Ability to finance the work

7 References

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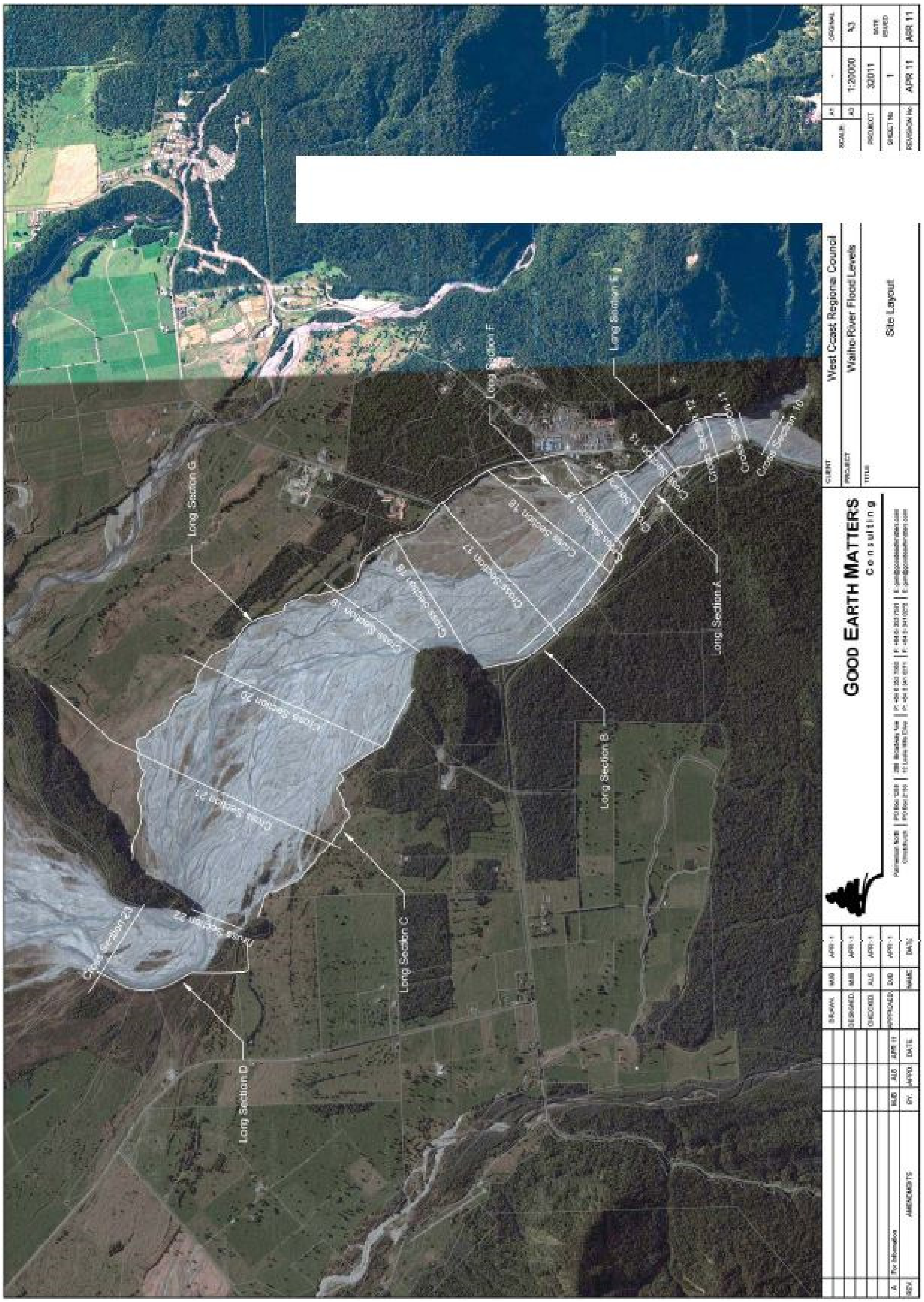
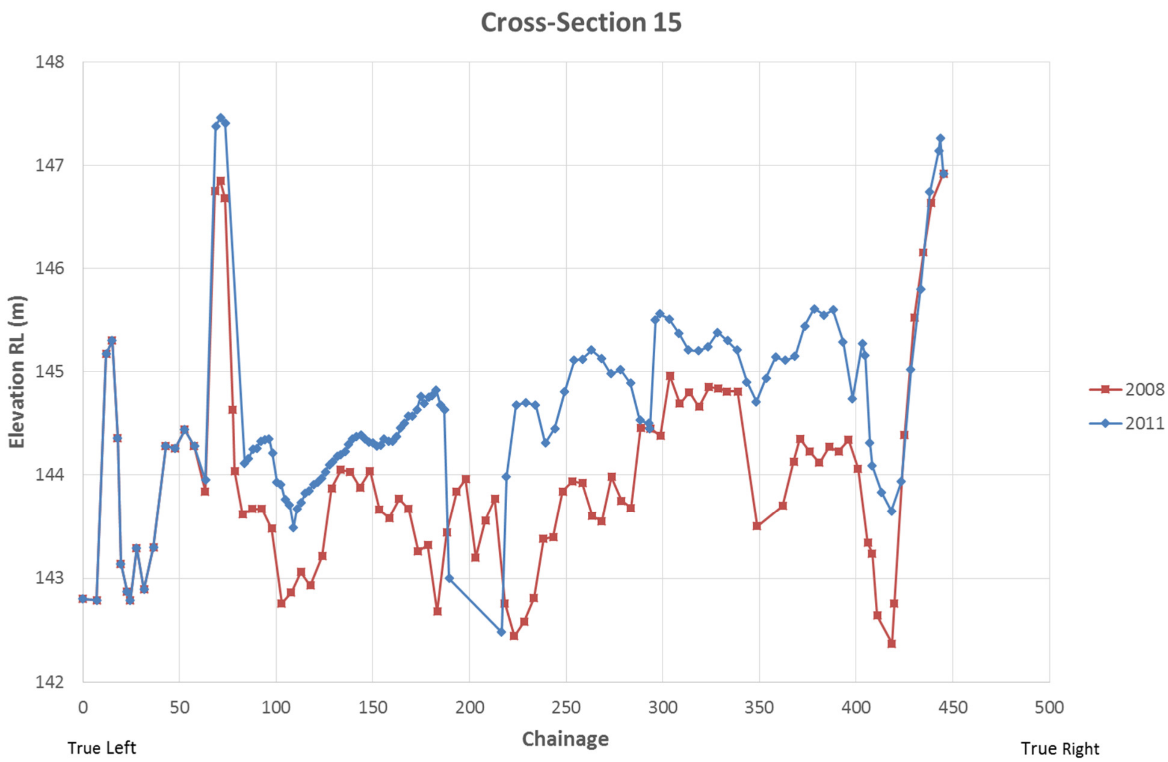
Appendix A

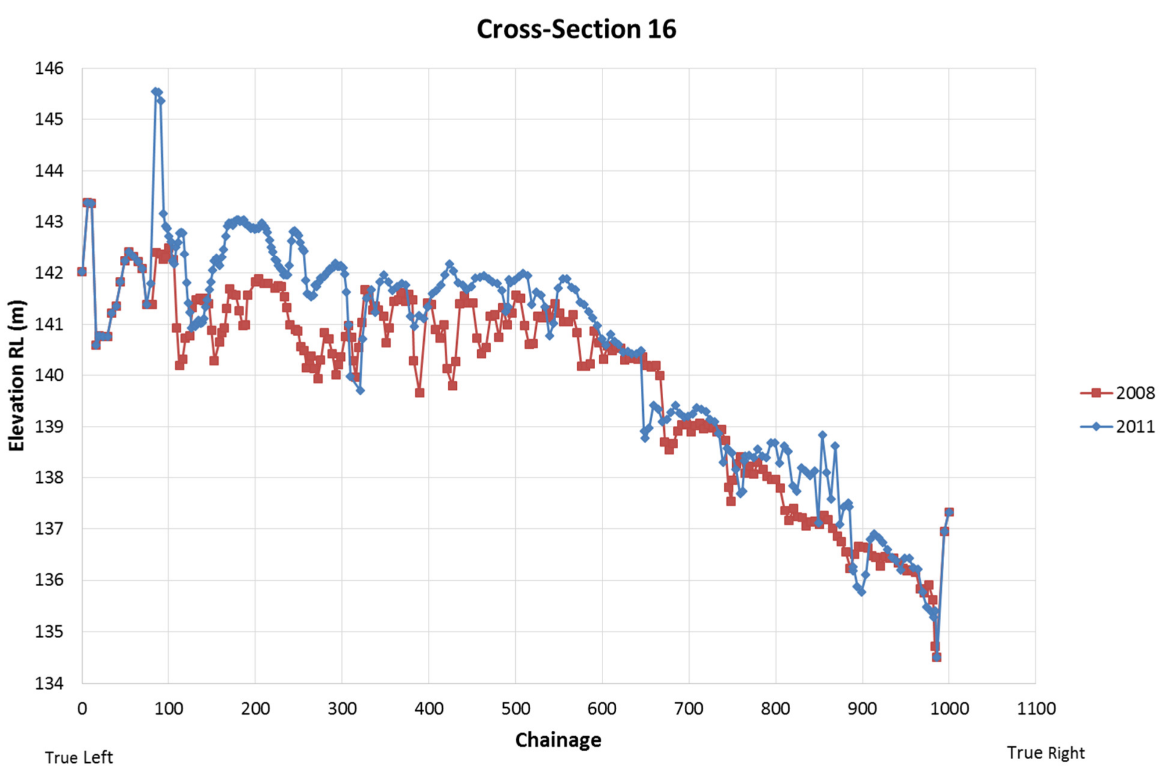
Location of Monitoring Cross-Sections on Waiho Alluvial Fan


West Coast Regional Council
Waiho River Flood Levels
Site Layout
GOOD EARTH MATTERS
Consulting
Long Section A
Long Section B
Long Section C
Long Section D
Long Section F
Long Section G
Cross Section 20
Cross Section 21
Cross Section 22
Cross Section 23
Cross Section 10
Cross Section 11
Cross Section 12
APR 11

Appendix B

2008 and 2011 Monitoring Cross-Sections on Waiho Alluvial Fan


Cross-Section 15
Elevation RL (m)
148
147
146
145
144
143
142
0
50
100
150
200
250
300
350
400
450
500
Chainage
True Left
True Right
2008
2011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Elevation RL (m)
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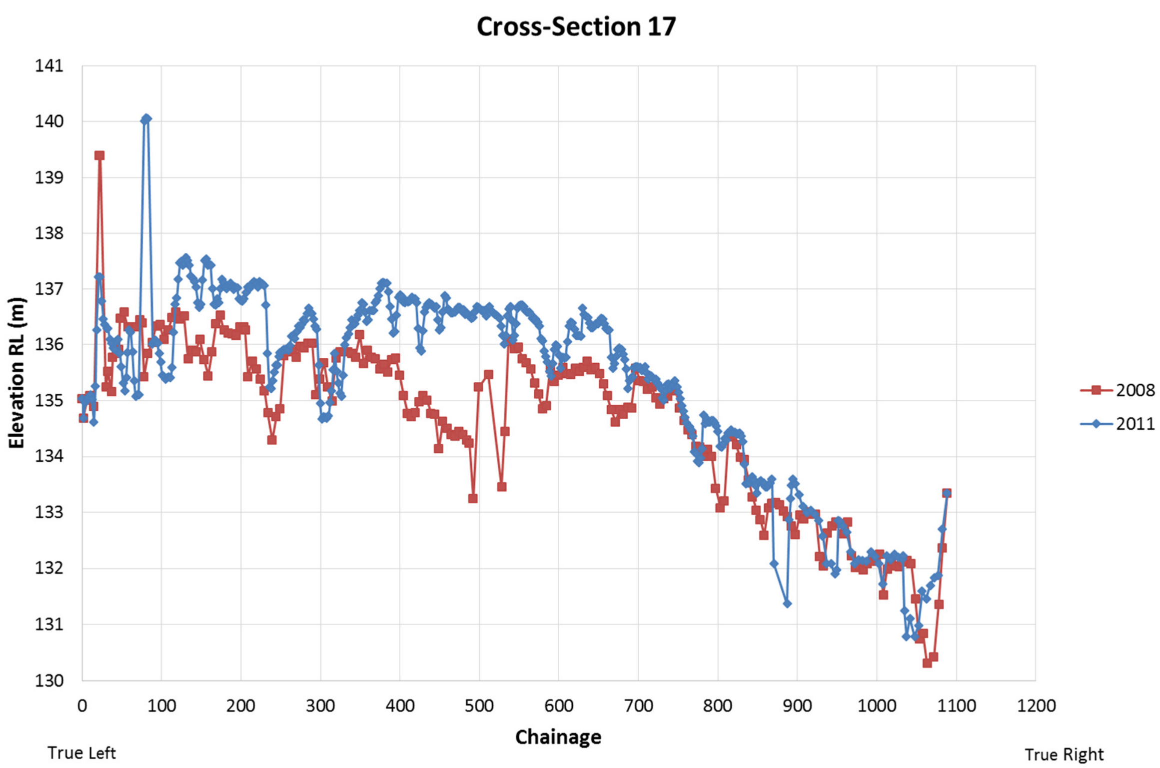
145
144
143
142
141
140
139
138
137
136
135
134
0
100
200
300
400
500
600
700
800
900
1000
1100
Chainage
True Left
True Right
2008
2011


Cross-Section 17
Elevation RL (m)
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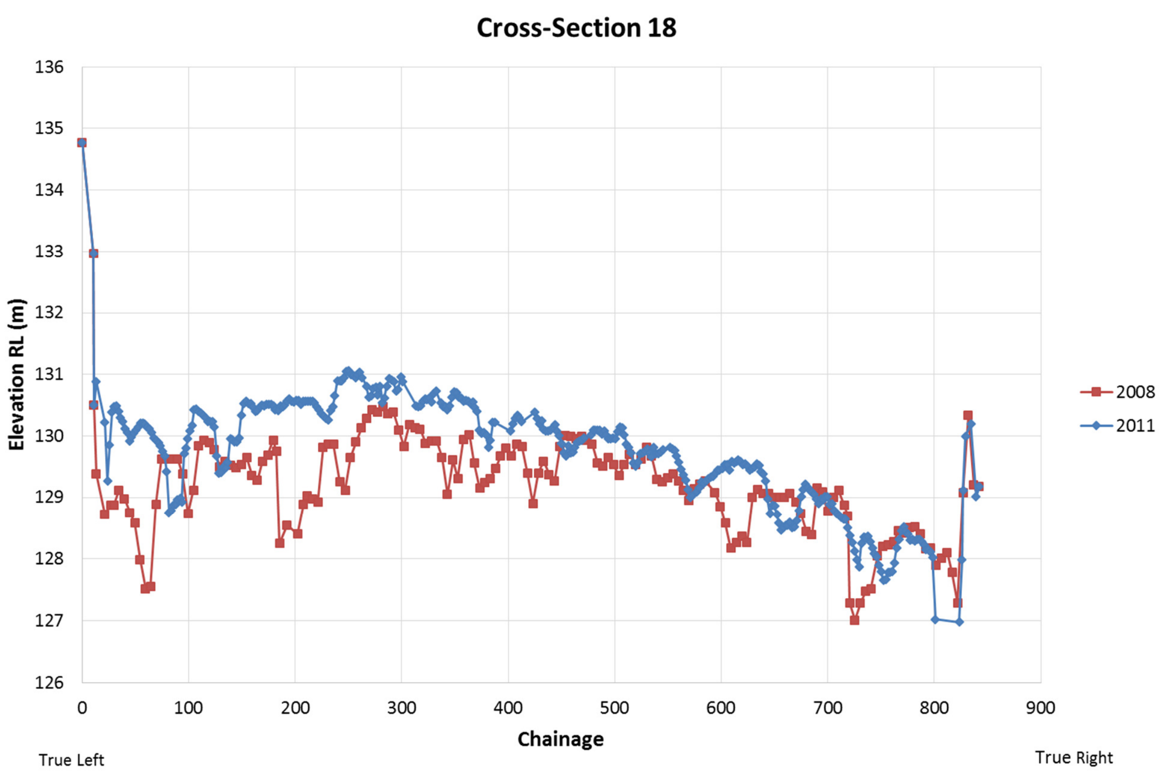
140
139
138
137
136
135
134
133
132
131
130
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
Chainage
True Left
True Right
2008
2011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Elevation RL (m)
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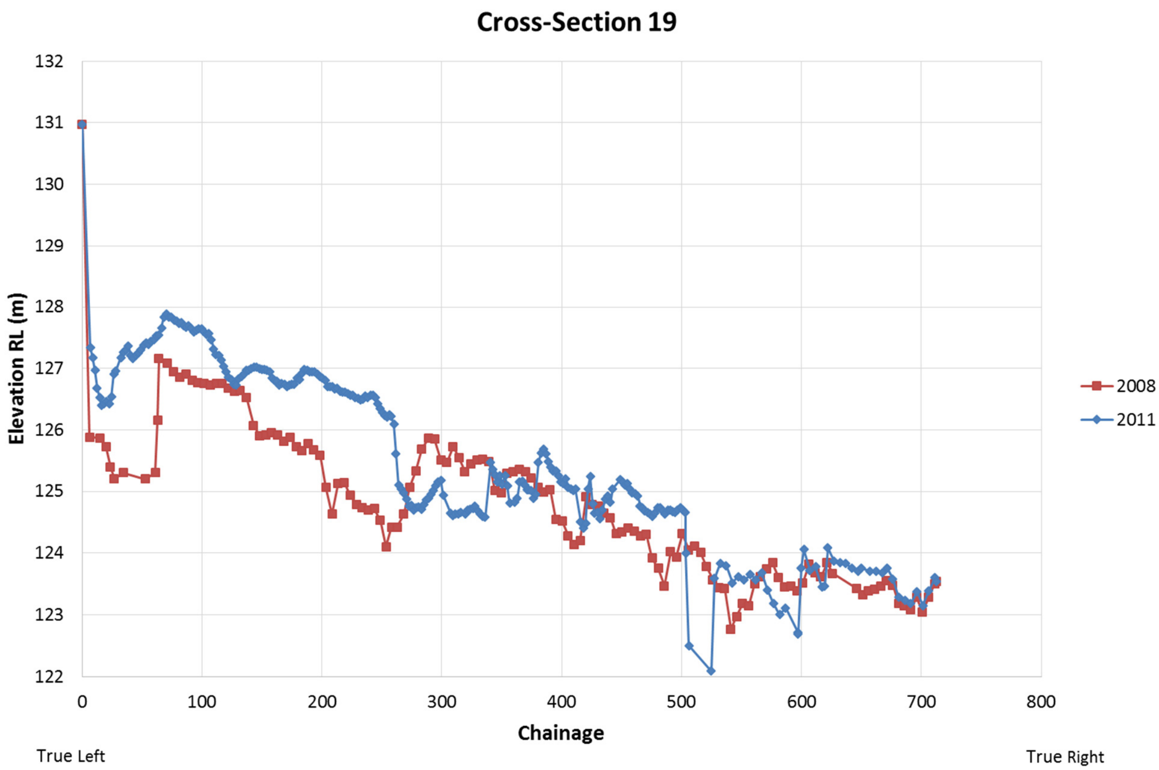
134
133
132
131
130
129
128
127
126
0
100
200
300
400
500
600
700
800
900
Chainage
True Left
True Right
2008
2011


Cross-Section 19
132
131
130
129
128
127
126
125
124
123
122
Elevation RL (m)
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200
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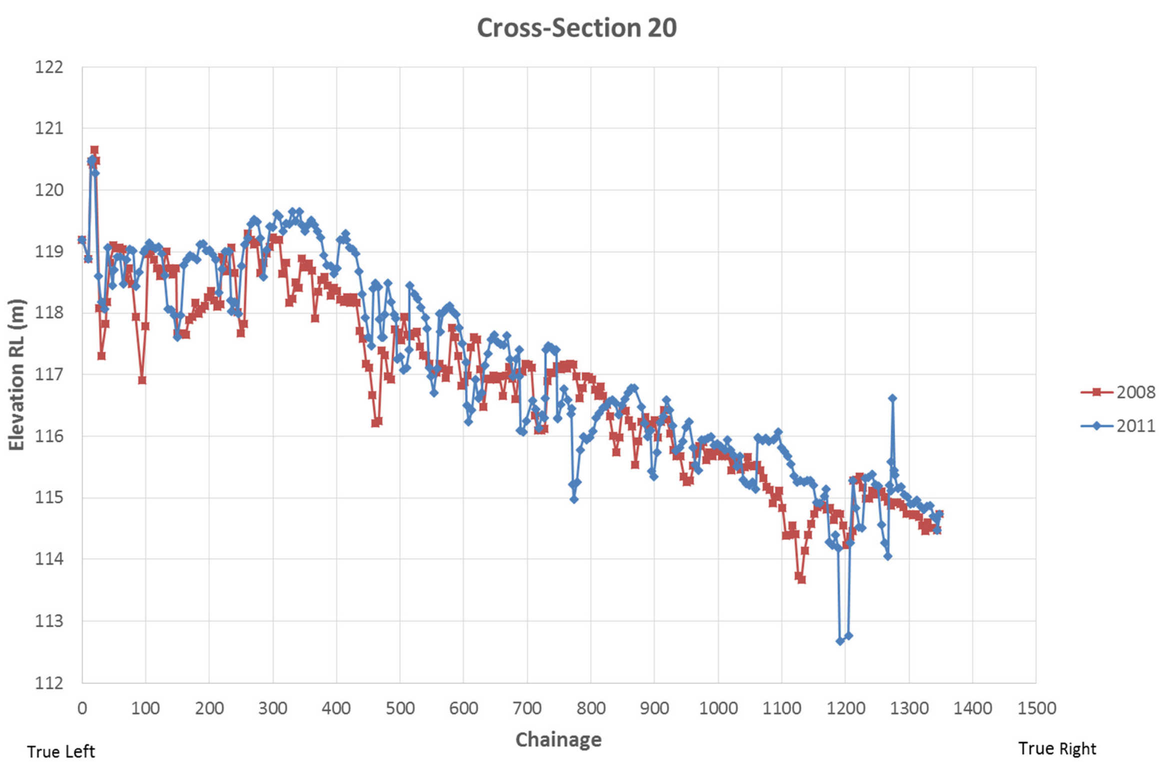
400
500
600
700
800
Chainage
2008
2011
True Left
True Right


Cross-Section 20
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121
120
119
118
117
116
115
114
113
112
Elevation RL (m)
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100
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400
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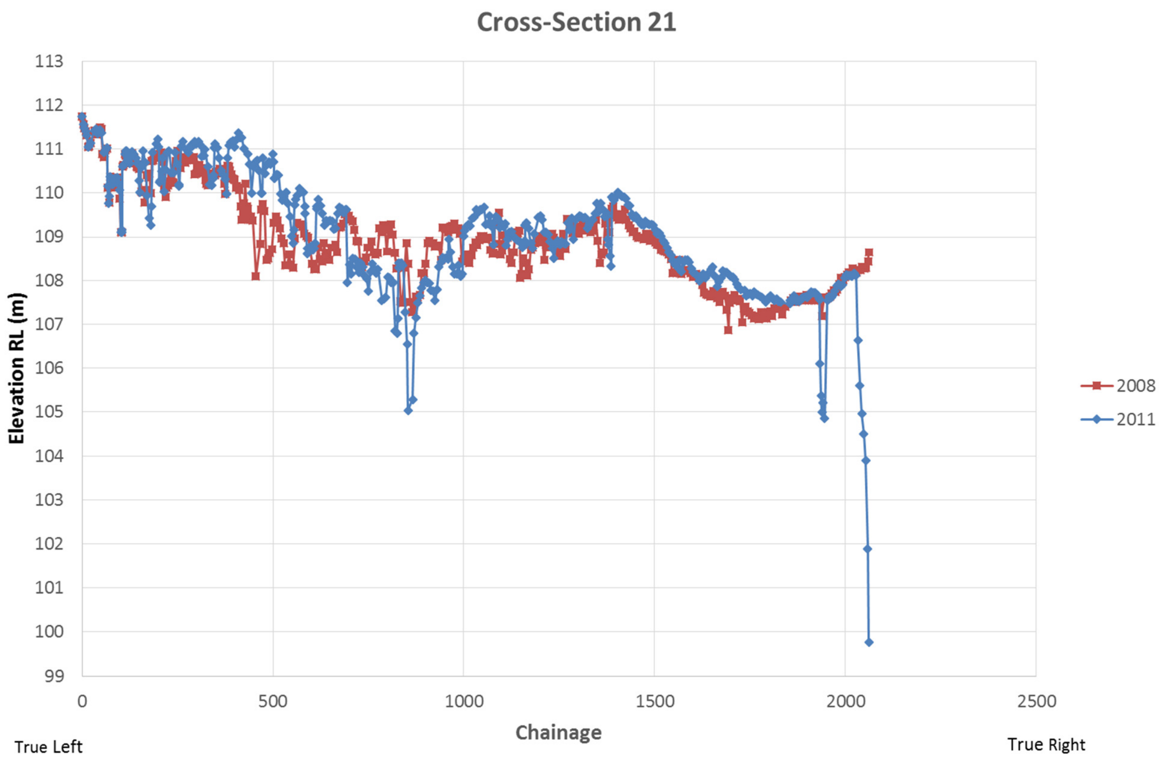
600
700
800
900
1000
1100
1200
1300
1400
1500
Chainage
2008
2011
True Left
True Right


Cross-Section 21
Elevation RL (m)
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112
111
110
109
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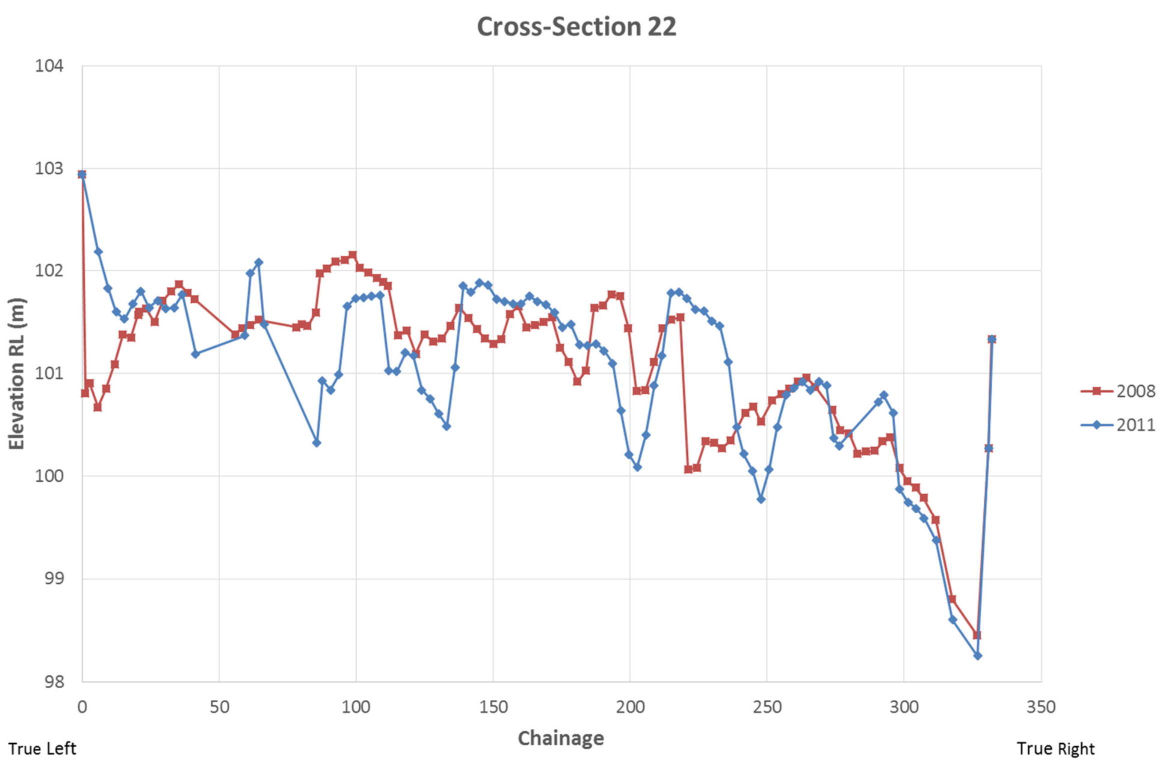
0
500
1000
1500
2000
2500
Chainage
True Left
True Right
2008
2011


Cross-Section 22
Elevation RL (m)
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103
102
101
100
99
98
0
50
100
150
200
250
300
350
Chainage
True Left
True Right
2008
2011

Appendix C

Capital Cost Estimates for the Upgrade of the Existing Plant

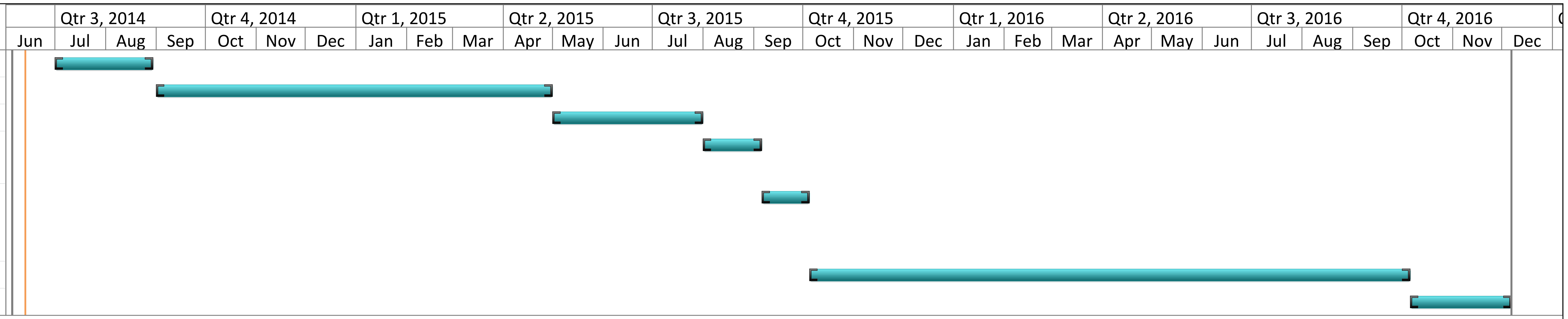
Updated 13 June 2014
Franz Josef WWTP Upgrade
Option 1: Upgrading Existing Oxidation Ponds

Item	*Description*	*Unit*	*Qty (min)*	*Qty (ave)*	*Qty (max)*	*Rate (min)*	*Rate (ave)*	*Rate (max)*	*Estimated Cost*	*Qty*	*Rate*	*Amount*
1	**Pond Desludging**											
1.1	Sludge Survey	LS		1		$4,000	$6,000	$10,000	$6,000		$6,333	$6,333
1.2	Desludging	m3	2000	2500	3000	$70	$120	$190	$300,000	$2,500	$123	$308,333
2	**Modification of Pond Flows**											
2.2	New Curtains and Anchoring	LS	200	250	300	$200	$220	$250	$55,000	$250	$222	$55,417
2.3	Modification of Inlet Pipework	LS		1		$12,000	$15,000	$25,000	$15,000		$16,167	$16,167
2.4	Modificationn of Outlet Pipe - Gabion Overflow Structure	LS		1		$15,000	$17,000	$25,000	$17,000		$18,000	$18,000
2.5	Modification of Existing Outlet Flow Meter	LS		1		$1,500	$2,000	$3,000	$2,000		$2,083	$2,083
2.6	Wetland Relocation and Modification	LS		1		$50,000	$60,000	$70,000	$60,000		$60,000	$60,000
	Subtotal								$455,000			$466,333
3	**Design and Supervision**	LS		1		10%	15%	20%	$68,250		15%	$69,950
4	**Preliminary and General**	LS		1		10%	15%	20%	$68,250		15%	$69,950
	CAPEX TOTAL								**$591,500**			**$606,233**
5	**Provisional Items**											
5.1	Installation of Inlet Works with washing and dewatering, mechanical screening	LS		1		$130,000	$139,000	$150,000	$139,000		$139,333	$139,333
5.2	Purchase of 5.5KW.hr Brush Surface Aerator	ea		2		$35,000	$45,000	$50,000	$90,000		$44,167	$88,333
5.3	Delivery and Installation of 5.5KW.hr Brush Surface Aerator	LS		1		$18,500	$20,000	$25,000	$20,000		$20,583	$20,583
5.4	Purchase, Supply and Installation of UV Channel Type	LS		1		$150,000	$165,700	$180,000	$165,700		$165,467	$165,467
5.5	MCC	LS		1		$10,000	$15,000	$25,000	$15,000		$15,833	$15,833
5.6	Telemetry	LS		4		$2,000	$2,500	$3,000	$10,000			$10,000
5.7	Cost to supply 25kva power to site	LS		1		$40,000	$45,000	$60,000	$45,000			$45,000
	Additional CAPEX TOTAL	LS							**$484,700**			**$484,550**
	Overall CAPEX TOTAL											**$1,090,783**

Appendix D

Timeline Schedule for the Development of the new WWTP

ID	Task Name	Start	Finish
1	Conceptual Design	1/07/14	29/08/14
2	Consent Applications	1/09/14	30/04/15
3	Detail Design	1/05/15	31/07/15
4	Procurement - Tendering	1/08/15	5/09/15
5	Procurement - Evaluation and Approvals	6/09/15	4/10/15
6	Construction of Plant	5/10/15	5/10/16
7	Commissioning	6/10/16	6/12/16



Appendix E

Capital Cost Estimates for the new WWTP

BUDGET COSTS ESTIMATE Updated 13 June 2014
Description: Franz Josef new WWTP

Item	*Description*	*Unit*	*Qty (min)*	*Qty*	*Qty (max)*	*Rate (min)*	*Rate (ave)*	*Rate (max)*	*Estimated Cost*	*Qty (using @Risk)*	*Rate (using @Risk)*	*Subtotal*	*Amount*
1	**Wastewater Mains and Pumping**												
1.1	Supply and Install New PE100 DN180 PN6.3 Rising Main	$/m	0	1045	1400	$225	$242	$290	$252,890	930	$247		$229,927
1.2	Supply and Install New PVC100 DN180 PN6.3 Gravity Main	$/m	600	900	1180	$225	$242	$290	$217,800	897	$247		$221,685.89
1.3	Pump Station, Supply, Build and Commission	LS	0	1	1	$122,582	$136,203	$272,405	$136,203		$156,633		$156,632.96
2	**Plant Construction**												
	Earth works to form new activated sludge reactor	LS		1		$126,913	$141,014	$183,318	$141,014		$145,715	$145,715	
	Concrete 1620m3 reactor	LS		1		$360,000	$400,000	$520,000	$400,000		$413,333	$413,333.33	
2.1	Reactor Costs	LS		1		$140,000	$360,000	$520,000	$360,000		$350,000		$350,000
2.3	New Activated Sludge Reactor Mechanical	LS		1		$328,628	$365,142	$604,685	$365,142.05		$398,980		$398,980.12
2.4	New Clarifier and Associated Items	LS		1		$634,424	$704,916	$916,391	$704,916		$728,413		$728,413
2.5	Ras Pumps and Associated Items	LS		1		$76,827	$85,364	$110,973	$85,364		$88,209		$88,209.08
2.6	Works Associated with the Alkalinity dosing	LS		1		$45,034	$50,038	$65,050	$50,038		$51,706		$51,706
2.7	Screen and Grit Separaration	LS		1		$266,292	$295,880	$384,644	$295,880		$305,743		$305,743
2.8	Dewatering Facility	LS		1		$437,040	$485,600	$631,280	$485,600		$501,787		$501,787
2.9	WAS Pumps and Rising Main	LS		1		$40,000	$50,000	$65,000	$50,000		$50,833		$50,833
2.10	Supply and Installation of Disk Filter	LS	0	1	1	$156,600	$174,000	$226,200	$174,000	1	$179,800		$149,833.33
2.11	Purchase, Supply and Installation of UV Channel Type	LS		1		$149,130	$165,700	$215,410	$165,700		$171,223		$171,223
2.12	MCC & Electrical	LS		1		200000	$250,000	$425,000	$250,000		$270,833		$270,833
2.13	PLC & Automation	LS		1		90000	$100,000	$130,000	$100,000		$103,333		$103,333
2.14	Building for MCC Dewatering and Blowers	m2	120	150	250	$2,250	$2,500	$3,250	$375,000	162	$2,583		$417,639
3	**Disposal System**												
	Scenario 1: Install Disposal Field (1st stage 0.8ha)	LS		1		$720,000	$800,000	$1,040,000	$800,000		$826,667	$826,666.67	
	Scenario 2: Install Disposal Diffuser outlet pipe	$/m	350	400	450	$225	$242	$260	$96,800	400	$242	$96,867	
	Scenario 2: Install Disposal Diffuser (excavation, reinstatement of land)	$/m3	160	200	220	$10	$15	$20	$3,000	197	$15	$2,950	
	Scenario 2: Install Disposal Diffuser (300mm diameter rock lining)	$/m3	120	130	150	$50	$60	$70	$7,800	132	$60	$7,900	
	Scenario 2: Install Disposal Diffuser (geotextile Lining)	$/m2	220	240	250	10	$15	$20	$3,600	238	$15	$3,575	
	Scenario 2: Install Disposal Diffuser (Boulders)	$/m3	10	12	14	$110	$120	$130	$1,440	12	$120	$1,440	
	Scenario 2: Install Disposal Diffuser (Reinstate Top Soil)	$/m2	220	240	250	$10	$20	$30	$4,800	238	$20	$4,767	
	Scenario 2: Total Costs for Disposal Diffuser	LS										$117,498	
3.1	Disposal System	LS		1		$117,498	$800,000	$1,040,000			$726,250		$726,250
4	**Electrical Supply to Site**												
4.1	Cost of supplying 320 kva power to WWTP site	LS		1		$25,000	$37,000	$45,000	$37,000		$36,333		$36,333
	Cost of supplying 15kva Power for pump station	LS	0	1	1		$20,000		$20,000	1	$24,343		$20,286
6	**Civils**												
6.1	Site access	m2	350	800	1100	$75	$90	$95	$72,000	775	$88		$68,458
6.2	Security fencing	m	5000	7000	8000	$200	$210	$230	$1,470,000	6833	$212		$1,446,389

Item	Description	Unit	Qty (min)	Qty	Qty (max)	Rate (min)	Rate (ave)	Rate (max)	Estimated Cost	Qty (using @Risk)	Rate (using @Risk)	Subtotal	Amount
6.3	Grassing & landscaping	ha	0.5	2	3	$18	$20	$25	$30	2	$20		$32.35
6.4	Stormwater Provisions	LS		1		$25,000	$30,000	$60,000	$30,000		$34,167		$34,166.67
6.5	General Lighting	LS	0	1	1	$25,000	$30,000	$50,000	$30,000	1	$32,500		$27,083
6.6	General transfer Pipework, water retic etc	LS		1		$30,000	$50,000	$100,000	$50,000		$55,000		$55,000
7	**Land Purchase**												
7.1	Subdivision costs	ea		1		$15,000	$20,000	$25,000	$20,000		$20,000		$20,000
7.2	Land Purchase value	ha	1.6	3	5	$8,000	$14,000	$21,000	$35,000	3	$14,167		$39,194
8	**Consenting**												
8.1	Disposal, Land Use and Land Designation Consent Applications	LS		1		$100,000	$150,000	$300,000	$150,000		$166,667		$166,666.67
	Subtotal								$7,441,017				$6,836,639
	Design and Supervision			1		10%	15%	20%	$1,116,152		15%		$1,025,496
	Preliminary and General	LS		1		10%	15%	20%	$1,116,152		15%		$1,025,496
	CAPEX TOTAL								**$9,673,322**				**$8,887,631**
	CAPEX Total for Mechanical Items								**$4,138,857**				**$3,745,166**

Appendix F

NPV Analysis for the new WWTP

BUDGET COSTS ESTIMATE
Description:

Updated 13 June 2014
Franz Josef WWTP Upgrade
Option 1: Construct new WWTP (no upgrade required in existing plant)

Cost of Capital	6.50%
General Inflation	2.50%
Operator Cost / hr	$ 60.00
M&E Maintenance / yr	3%
Apportionment to M&E	50.0%
Labour Input (hr/yr)	1920
Rising main power Input (kW.hr/yr)	17257
Treatment plant power input (kW.hr/Ml)	1300
Acetic Acid Consumption rate (t/ML)	0.0037
Acetic Acid ($/m3)	2152
Caustic Soda Consumption Rate (t/ML)	0.0581
Caustic Sode ($/m3)	1057
Polymer Consumption Rate (kg/ML)	2.0822
Polymer ($/kg)	10
UV Lamp Replacement <2024 (No/yr)	24
UV Lamp Replacement =>2024 (No/yr)	48
UV Lamp cost per unit ($/lamp)	560
Power cost ($/kW.hr)	$ 0.11
Sludge Transport ($/load)	200
Sludge Production Rate (wet t/ML)	0.5229
Sludge Loads per year (loads/yr)	Is based on 2.3 tonne per load
Screening and Grit Transport ($/load)	100
Screening and Grit Production (t/ML)	0.07
Screening and Grit Loads per year (loads/yr)	Is based on 0.67 tonne per load
Sludge, Screenings, Grit Disposal Cost ($/t)	150

	2014	2015	2016	2017	2018	2019	2020	2021	2022	2023
Year	**0**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
New Plant										
Average Daily Flow (m3/day)			552	570	587	605	622	640	660	680
Average Daily Flow (ML/yr)			201	208	214	221	227	233	241	248
Annual Power consumption (kW.hr/yr)			261924	270237	278550	286864	295177	303490	313170	322850
Acetic (t/yr)			0.75	0.77	0.79	0.82	0.84	0.86	0.89	0.92
Caustic Soda (t/yr)			12	12	12	13	13	14	14	14
Polymer(kg/yr)			420	433	446	459	473	486	502	517
Sludge (t/yr)			105	109	112	115	119	122	126	130
Sludge removal (loads per year)			46	47	49	50	52	53	55	57
Screenings and Grid (t/yr)			14.10	14.55	15.00	15.45	15.89	16.34	16.86	17.38
Sludge removal (loads per year)			21	22	22	23	24	24	25	26
UV Lamp Inflation ($/lamp)			588	603	618	634	649	666	682	699
Labour Costs $/hr			63	65	66	68	70	71	73	75
Power cost ($/kW.hr)			0.12	0.12	0.12	0.12	0.13	0.13	0.13	0.14
CAPEX & Renewals			$8,887,631							
Rising Main Power Consumption ($/yr)				$ 2,044	$ 2,095	$ 2,148	$ 2,201	$ 2,256	$ 2,313	$ 2,371
Treatment plant Power Consumption ($/yr)				$ 32,012	$ 33,821	$ 35,702	$ 37,655	$ 39,683	$ 41,972	$ 44,351
Acetic Acid Annual cost ($/yr)				$ 1,782	$ 1,883	$ 1,988	$ 2,097	$ 2,210	$ 2,337	$ 2,470
Alkalinity Control Annual cost ($/yr)				$ 13,748	$ 14,525	$ 15,332	$ 16,171	$ 17,042	$ 18,025	$ 19,047
Polymer Annual cost ($/yr)				$ 4,661	$ 4,925	$ 5,198	$ 5,483	$ 5,778	$ 6,112	$ 6,458
Labour Input ($/yr)				$ 124,058	$ 127,159	$ 130,338	$ 133,597	$ 136,937	$ 140,360	$ 143,869
Maintenance ($/yr)				$ 66,856.44	$ 147,154	$ 150,833	$ 154,604	$ 158,469	$ 162,431	$ 166,491
Monitoring Cost ($/yr)				$ 64,613.44	$ 66,229	$ 67,884	$ 69,582	$ 71,321	$ 73,104	$ 74,932
Sludge Transport ($/yr)				$ 10,222	$ 10,800	$ 11,400	$ 12,024	$ 12,671	$ 13,402	$ 14,162
Sludge Disposal ($/yr)				$ 17,558	$ 18,551	$ 19,582	$ 20,653	$ 21,766	$ 23,022	$ 24,327
Screenings and Grit Transport ($/yr)				$ 2,339	$ 2,471	$ 2,608	$ 2,751	$ 2,899	$ 3,067	$ 3,240
Screenings and Grit Disposal ($/yr)				$ 2,351	$ 2,483	$ 2,621	$ 2,765	$ 2,914	$ 3,082	$ 3,257
Fixed Energy Tariff ($120/day)				$ 43,800.00	$ 44,895	$ 46,017	$ 47,168	$ 48,347	$ 49,556	$ 50,795
UV Lamp Replacement($/yr)				$ 14,473	$ 14,835	$ 15,206	$ 15,586	$ 15,976	$ 16,375	$ 16,785
Annual OPEX Contingency (30%)				$120,155	$147,548	$152,058	$156,701	$161,481	$166,547	$171,766
Total										
Yr Total										
CAPEX Total			$13,029,786							
OPEX Total				$ 520,673	$ 639,375	$ 658,917	$ 679,036	$ 699,750	$ 721,705	$ 744,320
Annual Cost - PV				$ 431,039	$ 497,001	$ 480,931	$ 465,367	$ 450,294	$ 436,077	$ 422,292
Whole of Life - PV				**$23,251,529**						

2024	2025	2026	2027	2028	2029	2030	2031	2032	2033	2034	2035	2036	2037	2038	2039
10	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**
701	721	742	765	789	812	836	859	888	916	944	972	1001	1034	1067	1099
256	263	271	279	288	296	305	314	324	334	345	355	365	377	389	401
332530	342209	351889	363049	374210	385370	396530	407690	421128	434566	448004	461442	474880	490481	506083	521684
0.95	0.97	1.00	1.03	1.07	1.10	1.13	1.16	1.20	1.24	1.28	1.31	1.35	1.40	1.44	1.48
15	15	16	16	17	17	18	18	19	19	20	21	21	22	23	23
533	548	564	581	599	617	635	653	675	696	718	739	761	786	811	836
134	138	142	146	151	155	159	164	169	175	180	186	191	197	204	210
58	60	62	64	66	68	70	72	74	76	79	81	83	86	89	92
17.91	18.43	18.95	19.55	20.15	20.75	21.35	21.95	22.68	23.40	24.12	24.85	25.57	26.41	27.25	28.09
27	28	28	29	30	31	32	33	34	35	36	37	38	39	41	42
717	735	753	772	791	811	831	852	873	895	918	941	964	988	1013	1038
77	79	81	83	85	87	89	91	94	96	98	101	103	106	109	111
0.14	0.14	0.15	0.15	0.16	0.16	0.16	0.17	0.17	0.18	0.18	0.18	0.19	0.19	0.20	0.20
$83,718						$668,028	$704,916						$1,764,611		
$ 2,430	$ 2,491	$ 2,553	$ 2,617	$ 2,682	$ 2,749	$ 2,818	$ 2,888	$ 2,961	$ 3,035	$ 3,111	$ 3,188	$ 3,268	$ 3,350	$ 3,433	$ 3,519
$ 46,823	$ 49,391	$ 52,058	$ 55,051	$ 58,162	$ 61,394	$ 64,752	$ 68,238	$ 72,250	$ 76,419	$ 80,752	$ 85,253	$ 89,929	$ 95,206	$ 100,690	$ 106,389
$ 2,607	$ 2,750	$ 2,899	$ 3,065	$ 3,239	$ 3,419	$ 3,605	$ 3,800	$ 4,023	$ 4,255	$ 4,496	$ 4,747	$ 5,007	$ 5,301	$ 5,607	$ 5,924
$ 20,108	$ 21,211	$ 22,356	$ 23,642	$ 24,978	$ 26,366	$ 27,808	$ 29,305	$ 31,028	$ 32,818	$ 34,679	$ 36,612	$ 38,620	$ 40,886	$ 43,242	$ 45,689
$ 6,818	$ 7,192	$ 7,580	$ 8,016	$ 8,469	$ 8,940	$ 9,428	$ 9,936	$ 10,520	$ 11,127	$ 11,758	$ 12,414	$ 13,094	$ 13,863	$ 14,661	$ 15,491
$ 147,466	$ 151,152	$ 154,931	$ 158,804	$ 162,775	$ 166,844	$ 171,015	$ 175,290	$ 179,673	$ 184,165	$ 188,769	$ 193,488	$ 198,325	$ 203,283	$ 208,365	$ 213,574
$ 170,654	$ 174,920	$ 179,293	$ 183,775	$ 188,370	$ 193,079	$ 197,906	$ 202,854	$ 207,925	$ 213,123	$ 218,451	$ 223,913	$ 229,510	$ 235,248	$ 241,129	$ 247,158
$ 76,805	$ 78,725	$ 80,693	$ 82,711	$ 84,778	$ 86,898	$ 89,070	$ 91,297	$ 93,580	$ 95,919	$ 98,317	$ 100,775	$ 103,294	$ 105,877	$ 108,524	$ 111,237
$ 14,951	$ 15,771	$ 16,623	$ 17,579	$ 18,572	$ 19,604	$ 20,676	$ 21,790	$ 23,070	$ 24,402	$ 25,785	$ 27,223	$ 28,716	$ 30,401	$ 32,152	$ 33,972
$ 25,682	$ 27,091	$ 28,553	$ 30,196	$ 31,902	$ 33,675	$ 35,516	$ 37,429	$ 39,629	$ 41,916	$ 44,292	$ 46,761	$ 49,326	$ 52,220	$ 55,228	$ 58,354
$ 3,421	$ 3,609	$ 3,803	$ 4,022	$ 4,249	$ 4,486	$ 4,731	$ 4,986	$ 5,279	$ 5,583	$ 5,900	$ 6,229	$ 6,570	$ 6,956	$ 7,357	$ 7,773
$ 3,438	$ 3,627	$ 3,822	$ 4,042	$ 4,271	$ 4,508	$ 4,754	$ 5,011	$ 5,305	$ 5,611	$ 5,929	$ 6,260	$ 6,603	$ 6,991	$ 7,393	$ 7,812
$ 52,064	$ 53,366	$ 54,700	$ 56,068	$ 57,469	$ 58,906	$ 60,379	$ 61,888	$ 63,435	$ 65,021	$ 66,647	$ 68,313	$ 70,021	$ 71,771	$ 73,566	$ 75,405
$ 34,409	$ 35,269	$ 36,151	$ 37,054	$ 37,981	$ 38,930	$ 39,904	$ 40,901	$ 41,924	$ 42,972	$ 44,046	$ 45,147	$ 46,276	$ 47,433	$ 48,619	$ 49,834
$182,303	$187,969	$193,805	$199,993	$206,369	$212,939	$219,709	$226,684	$234,180	$241,910	$249,880	$258,097	$266,568	$275,636	$284,990	$294,639
$ 789,980	$ 814,534	$ 839,821	$ 866,636	$ 894,266	$ 922,736	$ 952,071	$ 982,296	$ 1,014,781	$ 1,048,276	$ 1,082,812	$ 1,118,419	$ 1,155,130	$ 1,194,421	$ 1,234,955	$ 1,276,769
$ 420,843	$ 407,440	$ 394,450	$ 382,201	$ 370,316	$ 358,784	$ 347,597	$ 336,743	$ 326,648	$ 316,835	$ 307,299	$ 298,032	$ 289,028	$ 280,619	$ 272,434	$ 264,467

2040	2041	2042	2043	2044	2045	2046
26	**27**	**28**	**29**	**30**	**31**	**32**
1132	1165	1204	1243	1282	1406	1544
413	425	439	454	468	513	564
537286	552887	571298	589709	608119	667337	732818
1.53	1.57	1.63	1.68	1.73	1.90	2.09
24	25	26	26	27	30	33
861	886	915	945	974	1069	1174
216	222	230	237	245	268	295
94	97	100	104	107	117	129
28.93	29.77	30.76	31.75	32.74	35.93	39.46
43	44	46	47	49	54	59
1064	1091	1118	1146	1175	1204	1234
114	117	120	123	126	129	132
0.21	0.21	0.22	0.23	0.23	0.24	0.24
			$920,883			
$ 3,607	$ 3,697	$ 3,790	$ 3,885	$ 3,982	$ 4,081	$ 4,183
$ 112,310	$ 118,461	$ 125,465	$ 132,746	$ 140,313	$ 157,826	$ 177,645
$ 6,254	$ 6,596	$ 6,986	$ 7,391	$ 7,813	$ 8,788	$ 9,891
$ 48,232	$ 50,873	$ 53,881	$ 57,008	$ 60,258	$ 67,779	$ 76,290
$ 16,353	$ 17,249	$ 18,269	$ 19,329	$ 20,431	$ 22,981	$ 25,867
$ 218,914	$ 224,387	$ 229,996	$ 235,746	$ 241,640	$ 247,681	$ 253,873
$ 253,337	$ 259,670	$ 266,162	$ 272,816	$ 279,636	$ 286,627	$ 293,793
$ 114,018	$ 116,868	$ 119,790	$ 122,784	$ 125,854	$ 129,000	$ 132,225
$ 35,862	$ 37,826	$ 40,063	$ 42,388	$ 44,804	$ 50,396	$ 56,725
$ 61,602	$ 64,975	$ 68,817	$ 72,811	$ 76,961	$ 86,567	$ 97,438
$ 8,206	$ 8,655	$ 9,167	$ 9,699	$ 10,251	$ 11,531	$ 12,979
$ 8,247	$ 8,698	$ 9,212	$ 9,747	$ 10,303	$ 11,589	$ 13,044
$ 77,290	$ 79,222	$ 81,203	$ 83,233	$ 85,314	$ 87,446	$ 89,633
$ 51,080	$ 52,357	$ 53,666	$ 55,007	$ 56,383	$ 57,792	$ 59,237
$304,593	$314,860	$325,940	$337,377	$349,182	$369,025	$390,846
$ 1,319,902	$ 1,364,394	$ 1,412,407	$ 1,461,968	$ 1,513,124	$ 1,599,109	$ 1,693,668
$ 256,715	$ 249,173	$ 242,198	$ 235,396	$ 228,763	$ 227,007	$ 225,757

Opus International Consultants Ltd
Opus House, Princes Street
Private Bag 3057, Waikato Mail Centre,
Hamilton 3240
New Zealand

t: +64 7 838 9344
f: +64 7 838 9324
w: www.opus.co.nz