LONGER, HIGH HEAD AND MULTIPLE PUMPING SOLUTIONS FOR WASTEWATER

Grant Pedersen, Bryan Jackson
Harrison Grierson Consultants Limited, Auckland, New Zealand

ABSTRACT

Pumping raw wastewater presents significantly greater challenges than pumping clean water, particularly if there is a long distance, a high head, or multiple pumping stations to pump into a common rising main. This paper looks at the technical challenges and sometimes significant cost savings that can be achieved by adopting smarter approaches to the conveyance of raw sewage.

Some of the challenges and issues considered include:

- High head pumping, outside the normal range of submersible sewage pumps.
- Long rising mains with an undulating route where large sections drain to become empty after each pumping cycle, and then progressively fill during start-up.
- Pumping wastewater from two or more separate pumping stations into a common rising main whilst maintaining self cleansing velocities during normal and peak periods.
- Using storage to reduce peak pump flows.
- Pumping wastewater along extremely hilly terrain.

The technical issues and options analysed consider the issues of maintaining a self-cleansing velocity, reasonable pumping head, the optimum efficiency, initial low flows vs. high future flow, solids transport, septicity, odour release, and many other factors.

KEYWORDS

Wastewater pumping, series, positive-displacement, fouling, solids, air locking,

1 INTRODUCTION

Pumping raw wastewater presents significantly greater challenges than pumping clean water, particularly if there is a long distance, a high head, or multiple pumping stations discharging into a common rising main.

Some of the challenges and issues are noted below. There are a number of possible solutions to these issues. The solutions described in this paper were considered in recent studies and projects undertaken by Harrison Grierson and are provided for the benefit of readers, and are not intended to represent the only solution available:

- High head pumping, outside the normal range of submersible sewage pumps – options include pumps in series or positive displacement pumps.
- Pumping wastewater along extremely hilly terrain – can require several pumping stations, or high pressure pipelines and positive displacement pumps.

- Long rising mains with undulating routes where large sections drain to become empty after each pumping cycle, and then progressively fill during start-up – the solution needs to be considered on a case by case basis, but air and vacuum valves are almost always necessary.

- Pumping wastewater from separate communities or pumping stations into a common rising main whilst maintaining self cleansing velocities during normal and peak periods – options include special control, variable frequency drive or separate pumping stations and rising mains.

- Using storage to reduce peak pump flows – often uneconomic unless rising mains are long.

1.1 TECHNICAL ISSUES

1.1.1 SOLIDS TRANSPORT AND VELOCITY

The main technical issues for pumping wastewater are related to the transport of solids, and in particular, the nature of those solids.

To effectively transport solids, ideally a velocity of 1.1 m/s should be achieved to re-suspend solids which have settled to the bottom of the pipe (Metcalf & Eddy, 1981, p381). Some other sources quote two feet per second (0.6m/s) as the minimum velocity, however as noted by Metcalf and Eddy, this may be insufficient to re-suspend solids that have settled. At a cost, pigging can be used to keep pipelines clean.

Once the solids have been re-suspended, a minimum velocity of 0.6 m/s should be maintained to ensure those solids continue to be transported. Lower velocities can be tolerated, as long as the system operates at higher velocities to re-suspend solids on a regular basis (e.g. periodic flushing).

In many cases, rising mains operate at lower velocities due to pump wear, air locks, or simply because it is impractical or uneconomic for the design to incorporate self cleansing velocities, which would result in excessive pump or pipeline pressures. In such cases, the design may be a compromise from the ideal, which may be acceptable if measures are taken to address the issues that may arise from that design.

1.1.2 SEPTICITY

The solids in wastewater, being predominantly biological in nature, will become progressively more septic when retained in a rising main devoid of free air for long periods of time. This leads to the generation of foul smelling sewage, primarily from hydrogen sulfide and other compounds generated under anaerobic conditions. This issue is also related to that of entrained and dissolved gases.

1.1.3 ENTRAINED AND DISSOLVED GASES

Air and other gases can become entrained in wastewater being transported in a rising main, either as dissolved gases or bubbles, or from being present in a rising main that has partially filled with air after a shutdown. In addition, biological action can generate foul gasses such as hydrogen sulfide.

The presence of air or gases creates two problems:

- The air accumulates at high points and can significantly increase the apparent head (or pressure) required by the pump to convey the design flow. The obstruction of flow by entrained air is generally referred to as an air lock. In a long pipeline with multiple high points this can be a significant issue.

- The venting (release) of air is usually achieved by air release valves. The air release is usually very odorous (containing hydrogen sulphide, and traces of methane, and other gases). Thus the venting can result in significant odour issues at the location of the air release.

1.2 POTENTIAL COST SAVINGS

Engineers are always looking for ways to save both capital and operating costs related to the pumping of wastewater. The very nature of wastewater means it is inherently more difficult and usually more expensive to convey than clean water.

Some ways in which costs can be reduced is by reducing pump head, reducing the pipe velocity, reducing the rising main diameter or pressure rating, or by using a less expensive pipe material. Cost savings may also be effected reducing the number of pumping stations, combining two parallel rising mains into one, or directionally drilling through high points to reduce the overall pumping head.

Storage at pumping stations can be used not only for emergency storage, but also to reduce peak flows by storing short-term peak inflows and allowing a lesser peak flow in the pipeline. This can be particularly successful for long pipelines.

1.3 BALANCING ISSUES AND COST SAVINGS

The challenge is to arrive at a design concept that addresses all the key requirements and issues whilst minimising both capital and operating cost and maintaining a low level of risk. While these factors all may not be able to be simultaneously optimised, a good design will achieve the optimal balance.

2 EXAMPLE PROJECTS

2.1 SPRINGHILL TO TE KAUWHATA PUMPING STATIONS AND RISING MAINS

The construction of the South Auckland Men's Correction Facility (Springhill), located northwest of Te Kauwhata, resulted in the need for a wastewater reticulation system to convey sewage from the Correction Facility to the Te Kauwhata Wastewater Treatment Plant (WWTP). The system comprised a pumping station (PS1) at the Correction Facility site with two high head pumps in series and a 2.4km rising main discharging to a second pumping station (PS2) and an associated 8.8km rising main discharging at the Te Kauwhata WWTP.

Figure 1: Location Map

Map Sourced from LINZ, Crown Copyright reserved

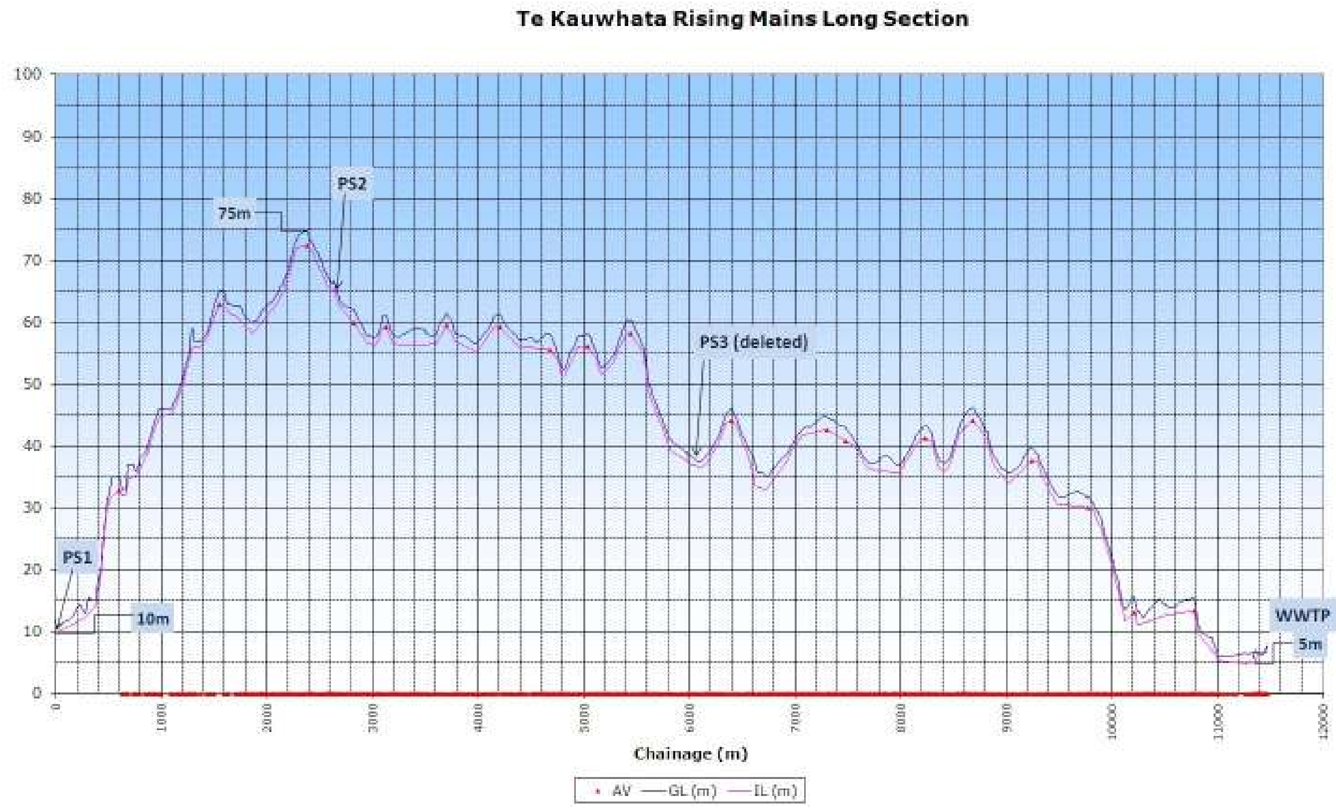
The main issues requiring careful consideration during the detailed design phase were:

- Significant elevation differences along the rising main alignment (PS 1 static head 67.5m);
- Length of rising mains (PS2 rising main 8.8km);

- Potential risk of odour;
- Sedimentation and fouling.

The original concept design was based on a conveyance system comprising three pumping stations - one located at the Corrections Facility, and two other pumping stations located along the rising main route to the Te Kauwhata WWTP. Following an extensive options analysis, the system as detailed below was selected to provide the best balance between cost, head loss, velocity, pump selection, reliability and time in transit. Deletion of one pumping station saved approximately $300,000 in capital cost, plus land acquisition, power supply and ongoing operation and maintenance costs.

Figure 2: Longitudinal Section Showing PS1, PS2 and Rising Mains



2.1.1 DESIGN FLOWS

The system is designed to convey up to 400m^3/day from the Springhill Correction Facility at a required pumping rate of 16.3 L/s.

2.1.2 DESCRIPTION OF PUMPING SYSTEM

PS1 and PS2 and their associated rising mains combine to form a single pumping system. Essentially, when PS1 commences pumping, PS2 will commence pumping a short time later. Furthermore there is a telemetry interlock between the two pumping stations to provide overall system control.

PUMPING STATION 1

This pumping station pumps all the wastewater flow from the Springhill Correction Facility to PS2. The pumping station comprises a wetwell (4mx4m plan dimensions), pump building, stand alone emergency storage tank (450m^3), duty and 100% standby pumpsets, control system and flow meter. In addition, a macerator was installed on the wetwell inlet, in accordance with standard practice for wastewater flows from such facilities.

The design utilised two Flygt 3171 SH274 (22kW each) high head duty pumps installed in series (one in the wetwell and one on the floor above in the pump station control building) to achieve the required pumping head of 95m. The PS1 rising main is of moderate length (2.4km) and rises steeply (up to 14% gradient) in isolated locations (refer Figure 2). As such it was desirable that the pump run time was sufficiently long to fully resuspend settled solids and to exchange the total volume of Rising Main 1 at each pump cycle. Consequently the wetwell was sized to achieve this requirement.

A 160mm OD PE rising main (with the first 200m as 180mm OD to reduce headloss) was selected following an extensive options analysis. A higher pressure rating pipe was used for the initial 1000m due to the high operating pressure and transient pressure analysis.

The PLC starts and stops the pumps based on set operating levels. The speed of each pump is controlled by a PLC and Variable Frequency Drives (VFD)'s to maintain a constant preset flow (16.3 L/s).

PUMPING STATION 2

This pumping station pumps the flow received from PS1 together with a small flow from the adjacent Whangamarino Water Treatment Plant. The pumping station comprises a wetwell (3.0m dia), valve chamber, satellite manhole/storage tank, duty and 100% standby pumps, by-pass diversion manhole, control system and flow meter. The design utilised a single Flygt 3171 SH272 (22kW) high head duty pump.

A further design consideration was that the rising main falls 51m over the 8.8km length. This unusual configuration required careful design for two reasons:

a) Firstly, air valves would be required at strategic locations along the pipeline to prevent air becoming trapped at high points and at downhill sections where the gradient increased;

b) Secondly, the pipeline would partially empty when the pumps stopped. On pump start-up, the pump at PS2 would discharge into a pipeline that was empty for the initial 200m, thus overloading the pump. To overcome this, a gravity bypass system was installed (refer to Section 2.1.3 below for details).

A 160mm OD PE pipeline was chosen to ensure an acceptable self cleansing velocity (>1.0m/sec) and hence reliability without an excessive volume of sewage sitting in the pipeline for extended periods. To reduce the headloss, the pipeline was increased to 180mm OD for discrete downhill sections, while still maintaining sediment transport.

Each pump is controlled by a PLC and VFD, similar to PS1. However for PS2, the speed of the pumps is controlled to maintain a preset water level in the wetwell. The design parameters for PS1 and PS2 are summarised in Table 1.

Table 1: Pumping Stations 1 and 2

	PS 1 and Rising Main	PS 2 and Rising Main
Design Flow	16.3 L/s	16.3 L/s
Static Head	67.5m	-51m
Total Operating Head	95m	53m
Rising Main	200m (180mm OD) 2180m (160mm OD)	6450m (160mm OD) 2380m (180mm OD)

2.1.3 DESIGN CHALLENGES

This pumping system is different to most pumping stations and rising mains due to the relatively low flow, the significant distance (11.2km) between PS1 and the Te Kauwhata WWTP, and the intervening topography.

HIGH HEAD

The static head for PS1 is 67.5m, requiring the use of two high head pumps in series. The high pressures in the initial section of Rising Main 1 requires a special slow starting control sequence, whereby the wetwell pump speed is slowly ramped up and the dry mounted pump commences ramping up after a 20 second delay, until the preset system flow (16.3L/s) is achieved.

NEGATIVE HEAD

The very long rising main (8.8km) from PS2 to the Te Kauwhata WWTP falls steeply from the pumping station and undulates over several summits and dips. The total fall is approximately 51m which partially counters the more than 100m of friction loss. However this negative head creates a problem on starting the pump against a free discharge (empty pipe), as the pump can overload if started against a low operating head. This challenge was solved by incorporating a hydraulic bypass diversion system which ensures the first 2.8km of pipe has been filled before the pump is started.

The following schematics illustrate in general terms the filling of the PS2 rising main via the bypass diversion system. The solid sections indicate the pipeline sections which are full and the partly shaded sections indicate wastewater flowing part full by gravity.

Figure 3: Prior to the Operation of PS1. Rising Main 2 will be partly empty.

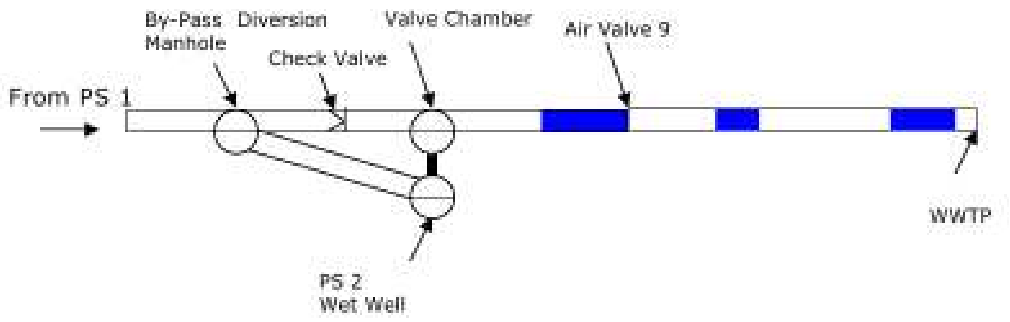


Figure 4: When PS1 starts, flow will pass through the bypass diversion MH and directly by gravity, downstream of the PS2 check valves and gate valves, into the downhill section of PS2 Rising Main.

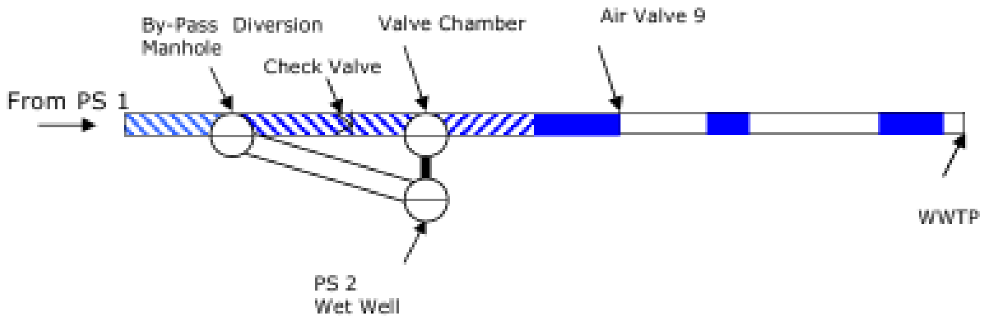


Figure 5: The flow from PS1 discharges into the PS2 rising main by gravity to the first low point. The flow then backs up until such time that the level reaches the bypass diversion manhole and commences to flow into the PS2 wetwell

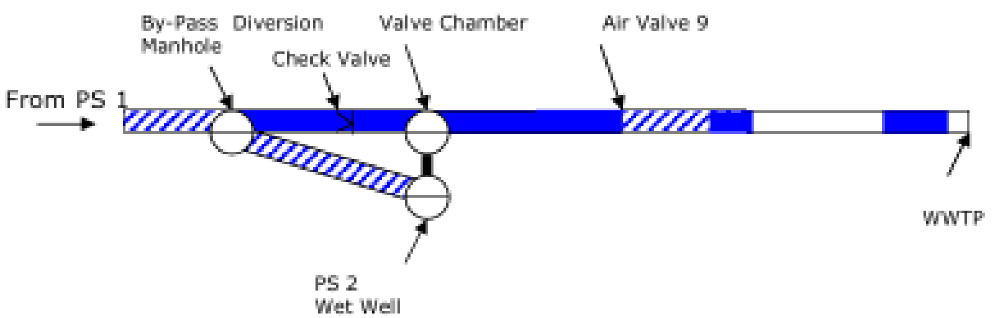
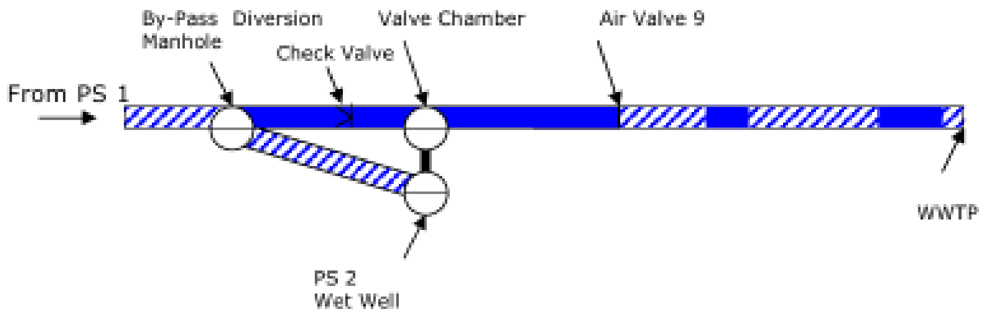
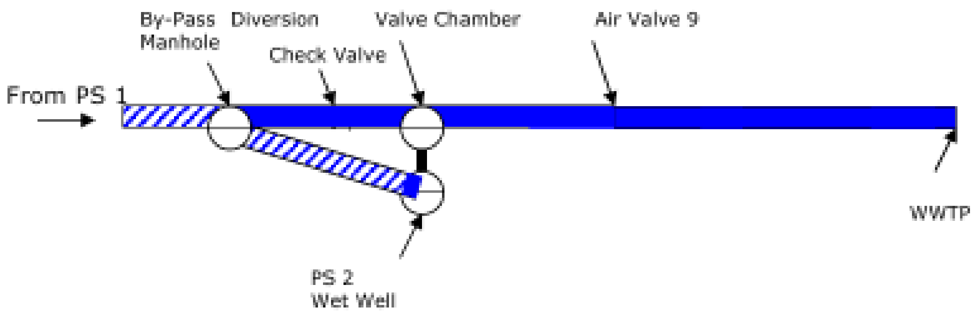


Figure 6: When the wetwell of PS2 reaches the pump start level, the pump starts. The rising main is then sufficiently full and the pump operates in the "safe" part of its performance curve.



A counterweighted check valve is installed on the bypass between the PS2 valve chamber and the bypass diversion MH. When PS2 starts, the bypass check valve closes to prevent the pumped flow from PS2 flowing back up to the bypass diversion MH.

Figure 7: The duty pump in PS2 will continue to operate to maintain a set level in the wetwell. After a pump run time of approx 30 minutes, Rising Main 2 is substantially full. Once the system stops pumping, Rising Main 2 will start to drain, the bypass check valve will open and the pipeline will revert to the situation depicted in Figure 3.



Consideration was given to alternative solutions to overcome this challenge (e.g. electrically actuated valves at PS2 or at the WWTP, and jockey pumps), however the hydraulic bypass diversion arrangement was chosen due to its simplicity.

POTENTIAL RISK OF ODOUR

At average flows, sewage was likely to sit in the rising mains all night. The potential for wastewater in the rising mains to become anaerobic and generate hydrogen sulphide and other gases was recognised. Two methods were used to mitigate the potential effects of odour being emitted by air valves. These were:

a) Treatment of odorous air using an activated carbon filter for those air valves in close proximity to residential properties;

b) Dispersal of odorous air using an elevated vent pipe for those air valves remote from residential properties or public areas.

In addition, the materials used for the rising main (PE) and ancillary structures (sulphate resistant cement and epoxy coating) were specified to mitigate the potential effects of sulphuric acid corrosion. While the option of dosing chemicals to control odour was considered, this was not pursued due to cost. It was decided to wait and see if any significant problems occurred.

SEDIMENTATION / FOULING

Failure to maintain a reasonable velocity in the rising mains could result in siltation and excessive fouling of the pipe walls, resulting in increased friction losses and a reduced flowrate. Given the very long length of Rising

Main 2, any significant increase in friction would have a major effect on the total pumping head. The velocity in the rising and inverted sections is in excess of 1.0m/s and in the falling section the velocity is in excess of 0.8m/s.

In addition, Rising Main 2 has three locations where the pipework has been configured to allow a cleaning pig to be introduced and retrieved from the pipeline. Both PS1 and PS2 have flowmeters installed, therefore Council are able to monitor the flow and performance of the pumping system. Using the flow information, Council are able to identify any trends that suggest flow reduction and to programme flushing or pigging maintenance as necessary to maintain the performance requirements of the pumping system.

2.1.4 OPERATION

The system was commissioned in March 2007 and the operating flow/head results were consistent with the design. Shortly following the introduction of wastewater there was a single complaint of odour. An inspection of the area was made shortly after and no odour was detected. Since then the system has been operating successfully with no reported issues.

2.2 LYTTELTON HARBOUR WASTEWATER STUDY

For this study, it was necessary to investigate options for the treatment and disposal of wastewater from three separate communities, Lyttelton, Diamond Harbour and Governors Bay. Options included improved treatment and harbour disposal, land irrigation and conveyance to Christchurch. This paper considers the option involving conveyance to Christchurch.

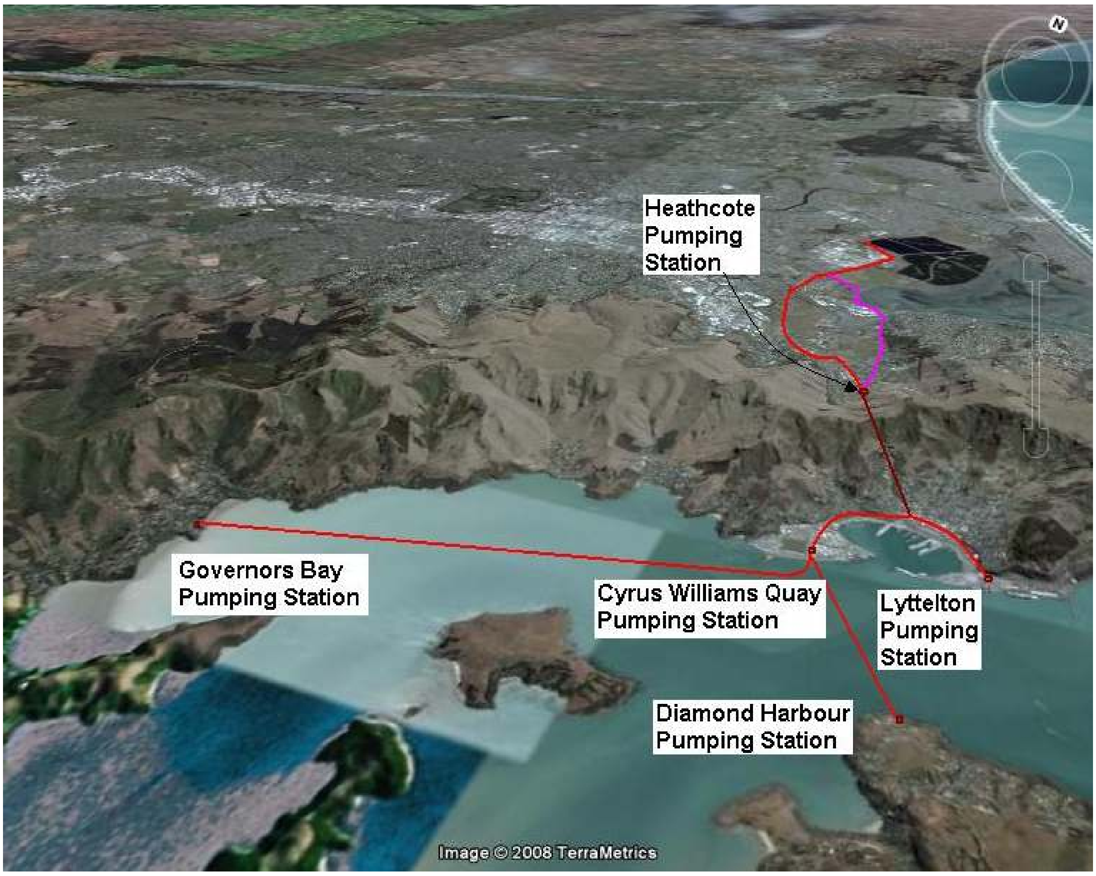
With all options considered, the aim was to reduce the overall cost, yet maintain self cleansing pipeline velocities during normal and peak flow conditions.

The main issues associated with the project were as follows

- A very high peak flow ratio, due to very high inflow and infiltration (I&I) (Lyttelton has a peak flow over 10 times Average Dry Weather Flow (ADWF).
- Long length of rising mains, several > 5km.
- Use of under-sea pipelines to convey wastewater under the harbour.
- Conveyance through the Port Hills by either the road or rail tunnel.

The figure below shows the general layout of the communities and the intended conveyance under the harbour, through the Port Hills and to Christchurch WWTP.

Figure 8: *Lyttelton Harbour Wastewater Scheme*



Wastewater would be pumped from Governors Bay near the existing WWTP and harbour outfall, through a new under-sea pipeline, to Cyrus Williams Quay, Lyttelton. The overland route would be significantly longer and have excessive elevation differences. Similarly, wastewater would also be pumped from Diamond Harbour near the existing WWTP, through a new under-sea pipeline, also to Cyrus Williams Quay.

At Cyrus Williams Quay, a new pumping station and pipeline would pump wastewater to the rail or road tunnel and to Christchurch reticulation.

Wastewater from Lyttelton would be conveyed through the existing Lyttelton system to the Lyttelton WWTP, east of the town. At this location, a new pumping station and pipeline would pump wastewater to the road or rail tunnel and to Christchurch.

The following is a list of 5 options that were considered by Harrison Grierson for the conveyance of wastewater:

Option 1 Pump the instantaneous Peak Wet Weather Flows (PWWF's) from Diamond Harbour and Governor's Bay across the Harbour floor to Cyrus Williams Quay at Lyttelton. From Cyrus Williams Quay (CWQ), the wastewater would be pumped to the existing Lyttelton WWTP. From here, the flow would be combined with Lyttelton's wastewater and the combined flow would be pumped through the Lyttelton Rail tunnel to Heathcote Valley. The flow would then be pumped from Heathcote Valley the remaining distance to the Christchurch WWTP.

Option 2 Same as Option 1 above, but instead join the pipes conveying flow from Cyrus Williams Quay and Lyttelton WWTP at a location just before the Lyttelton Rail Tunnel. From there on, Option 2 is the same as Option 1

Option 3 Using the same pipe network at Option 2, the peak wet weather flow would be attenuated in storage tanks to reduce pumping flows. A sensitivity analysis has shown that the optimum balance between storage and pumped flow is approximately 40% of the instantaneous peak flow for all locations except Governors Bay, due to the relatively low flow from this community.

Option 4 Same as Option 3 above, except the Road Tunnel would be used instead of the Rail Tunnel. This option has a higher energy requirement as the end of the Road tunnel has a much higher elevation (approximately 80 metres) than the rail tunnel (25m).

Option 5 Same as Option 3 above, but delete the pumping station at Heathcote Valley and instead pump directly from Cyrus Williams Quay and Lyttelton to the Christchurch WWTP.

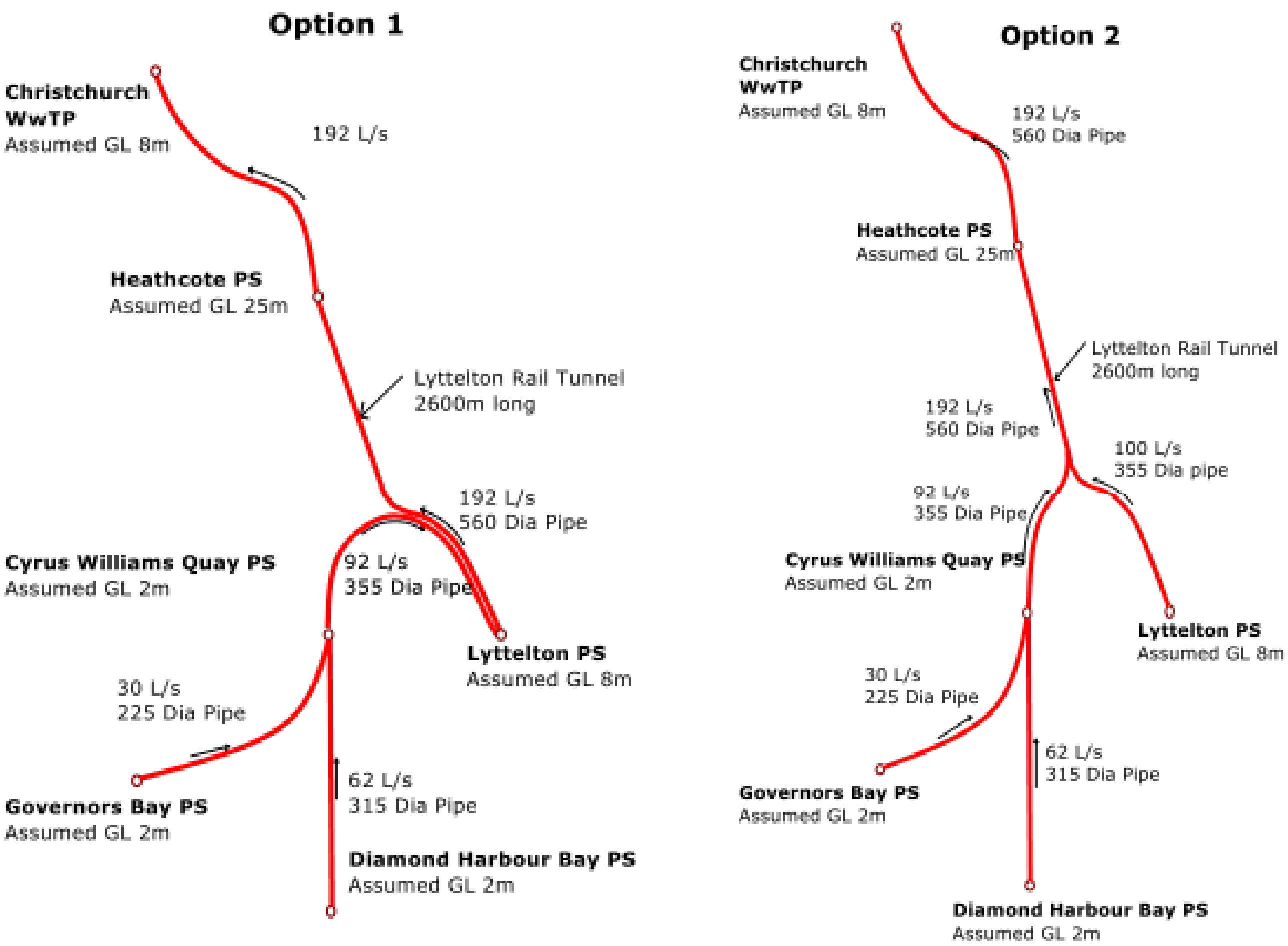
The following table summarises the key features of each option.

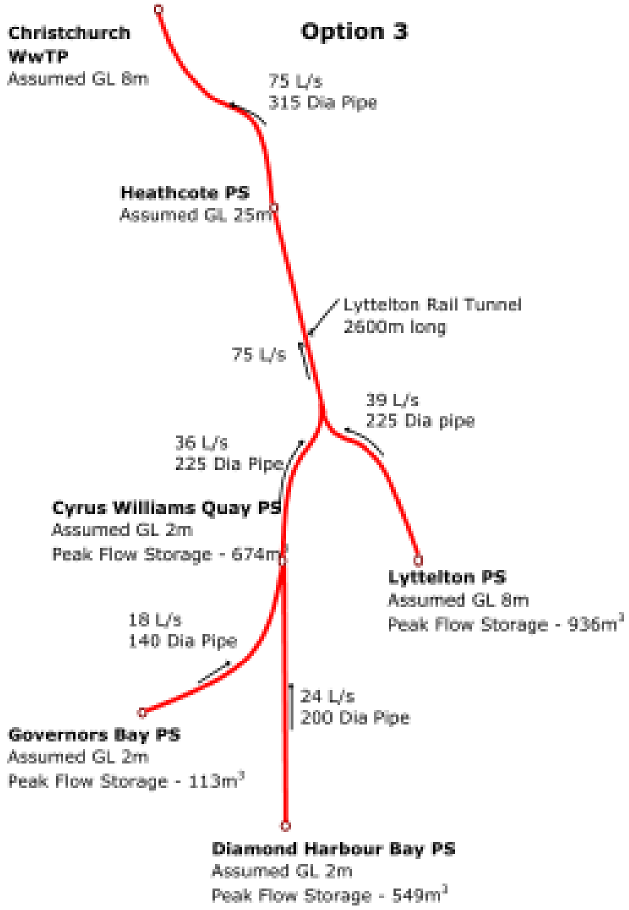
Table 2: Summary of Options considered for Conveyance of Wastewater

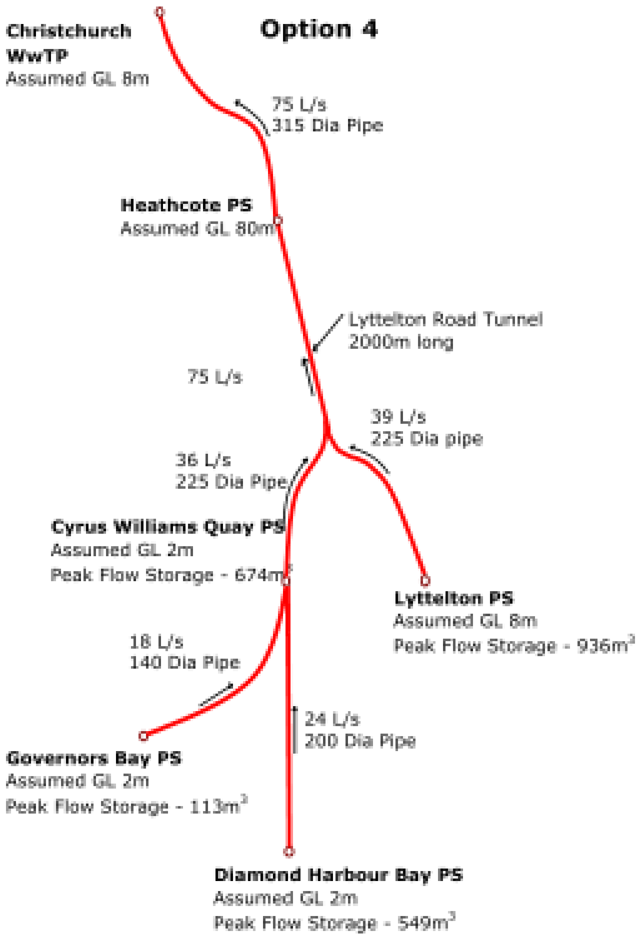
Option	Pump Peak Wet Weather Flow or Balanced Flow	Pump from CWQ to Lyttelton then to Heathcote Valley	Tunnel Utilised	Pump Station at Heathcote Valley
1	Peak	Y	Rail	Y
2	Peak	N	Rail	Y
3	Balanced	N	Rail	Y
4	Balanced	N	Road	Y
5	Balanced	N	Rail	N

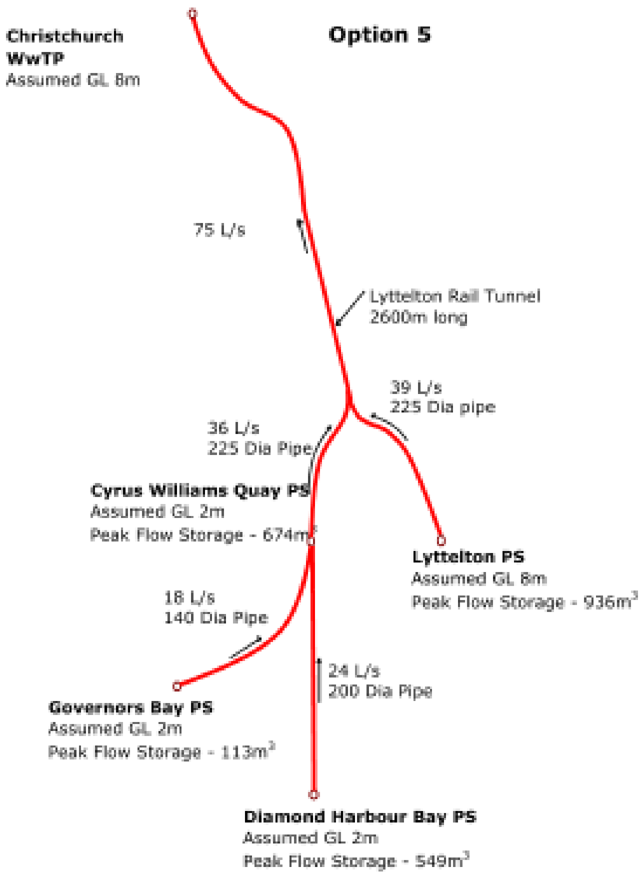
The following schematic diagrams depict the five options.

Figure 9: Five Options




Option 3
Christchurch WwTP
Assumed GL 8m
75 L/s
315 Dia Pipe
Heathcote PS
Assumed GL 25m
Lyttelton Rail Tunnel
2600m long
75 L/s
39 L/s
225 Dia pipe
36 L/s
225 Dia Pipe
Cyrus Williams Quay PS
Assumed GL 2m
Peak Flow Storage - 674m3
Lyttelton PS
Assumed GL 8m
Peak Flow Storage - 936m3
18 L/s
140 Dia Pipe
24 L/s
200 Dia Pipe
Governors Bay PS
Assumed GL 2m
Peak Flow Storage - 113m3
Diamond Harbour Bay PS
Assumed GL 2m
Peak Flow Storage - 549m3


Option 4
Christchurch WwTP
Assumed GL 8m
75 L/s
315 Dia Pipe
Heathcote PS
Assumed GL 80m
Lyttelton Road Tunnel
2000m long
75 L/s
39 L/s
225 Dia pipe
36 L/s
225 Dia Pipe
Cyrus Williams Quay PS
Assumed GL 2m
Peak Flow Storage - 674m3
Lyttelton PS
Assumed GL 8m
Peak Flow Storage - 936m3
18 L/s
140 Dia Pipe
24 L/s
200 Dia Pipe
Governors Bay PS
Assumed GL 2m
Peak Flow Storage - 113m3
Diamond Harbour Bay PS
Assumed GL 2m
Peak Flow Storage - 549m3


Option 5
Christchurch WwTP
Assumed GL 8m
75 L/s
Lyttelton Rail Tunnel
2600m long
39 L/s
225 Dia pipe
36 L/s
225 Dia Pipe
Cyrus Williams Quay PS
Assumed GL 2m
Peak Flow Storage - 674m3
Lyttelton PS
Assumed GL 8m
Peak Flow Storage - 936m3
18 L/s
140 Dia Pipe
24 L/s
200 Dia Pipe
Governors Bay PS
Assumed GL 2m
Peak Flow Storage - 113m3
Diamond Harbour Bay PS
Assumed GL 2m
Peak Flow Storage - 549m3

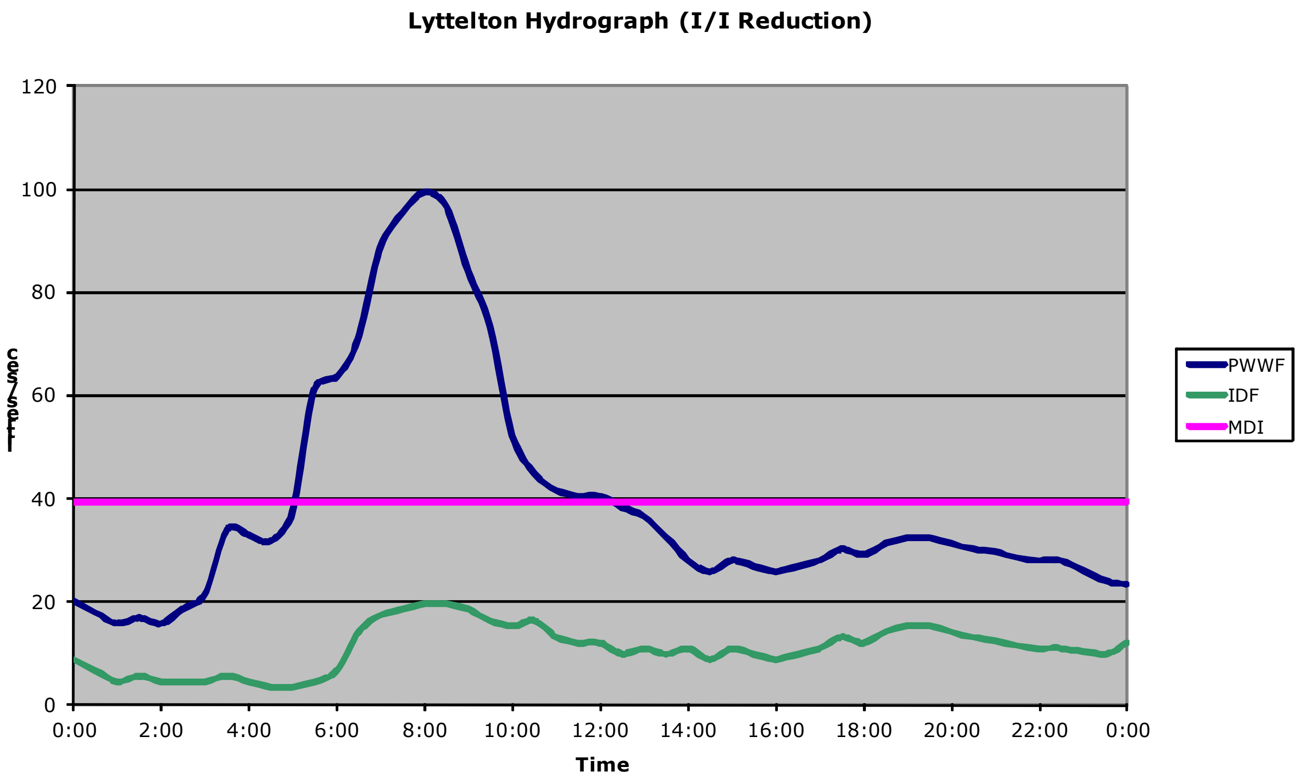
2.2.1 PEAK WET WEATHER STORAGE

For those options that utilise peak flow balancing to reduce pump flows (Options 3, 4 and 5), the storage is calculated based on a synthetic hydrograph of typical wastewater flows and a superimposed synthetic peak storm flow, coinciding at the worst time to give the maximum storage volume requirement.

The hydrograph utilises an Idealised Diurnal Flow (IDF) derived from real data for a similar-sized community, scaled to fit the design flow rates. To this, a conceptual I&I profile was added to give the maximum peak flow, or worst case scenario. The synthetic hydrograph was fine-tuned to provide an overall peak flow hydrograph that matched the recorded peak flow and Maximum Daily Inflow (MDI) data records available.

The synthetic hydrograph developed for Lyttelton is shown below. Similar hydrographs were developed for the other communities.

Figure 10: Synthetic Hydrograph for Lyttelton



Storage requirements were increased by a contingency of 30% to account for the use of synthetic hydrographs. The use of peak wet weather storage for Options 3, 4 & 5 significantly reduced the peak pumped flow to around 40% of the instantaneous peak. The provision of storage and emergency generators at all pumping stations was proposed to enable wet weather pumping station overflows to be reduced to the required 1 in 2 year Annual Recurrence Interval (ARI) with a high degree of confidence.

The provisions above reduced overall capital costs by around 27% when compared to pumping the peak flows, due to smaller diameter rising mains and smaller pumps. The operating costs increased slightly (7%), due to the higher friction loss in the smaller diameter pipelines. An NPV analysis showed that the use of peak flow storage tanks reduced NPV costs by approximately 25%.

2.2.2 COMMON RISING MAIN

Combining pumped flows from two rising mains into a single rising main in the Lyttelton Rail Tunnel also resulted in capital cost savings, and was essential to the viability of the rail tunnel option, as there would not be sufficient room for two pipes.

Although this is less conventional and would create some operational issues when one or other, or both pumping stations are operating, the overall cost savings of not having an additional pumping station are considerable.

The use of a common rising main for Options 2 -5 reduced overall capital costs by around 4% for the equivalent option, due to no duplication of rising mains and pumping synergies. The operating costs increased slightly (2%), and the NPV analysis showed a reduction in overall costs by approximately 4%.

2.2.3 RAIL TUNNEL vs ROAD TUNNEL

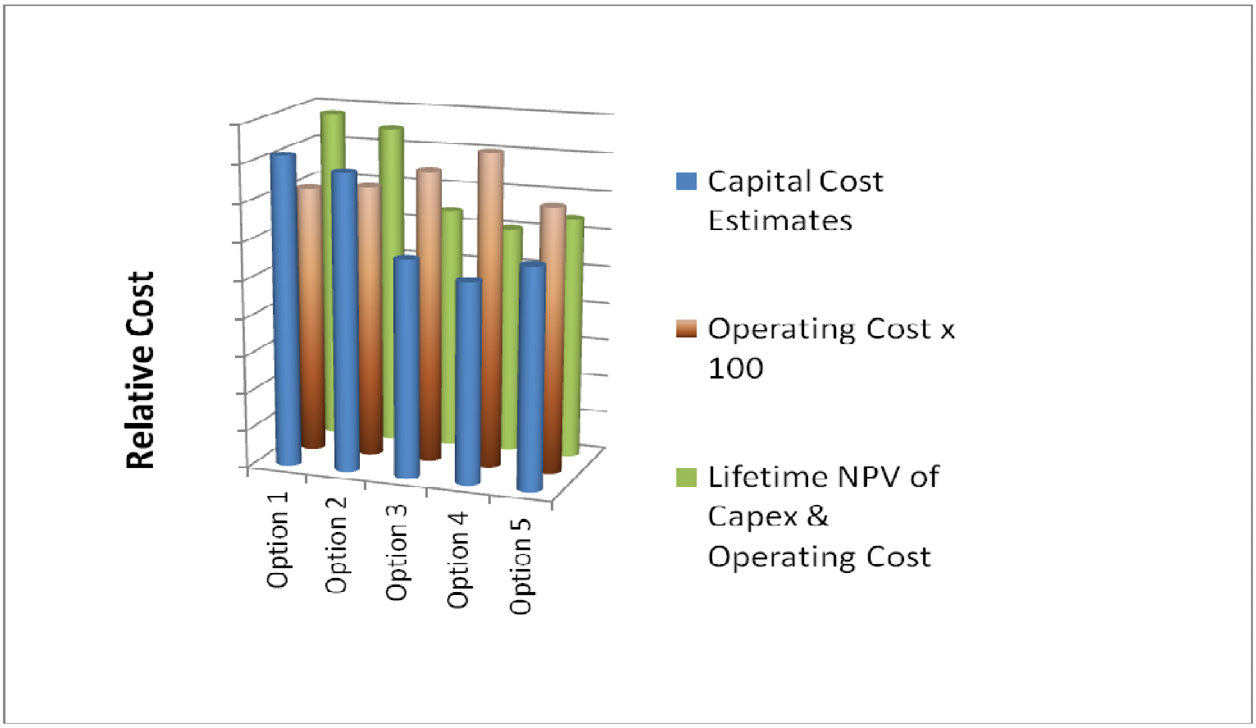
The rail tunnel already has two water mains and there would be insufficient room for more than one wastewater rising main. The rail tunnel is very tight, with little room on either side of a train. The road tunnel also has watermains, but is much larger than the rail tunnel.

Interestingly enough, the road tunnel ended up having a lower overall cost than the rail tunnel, mainly due to the difficulties associated with installing a rising main in the rail tunnel. All work is likely to be required to be carried out during limited hours and at the end of each work session, the tunnel would be required to be free for rail use. These constraints significantly increased the estimated cost and risk to install a wastewater pipeline in the rail tunnel.

2.2.4 RELATIVE COSTS

The following graphic shows the relative costs of the five options, and the relative operating costs and NPV (note the operating cost is shown at a different scale).

Figure 11: Relative Costs of the Five Options



In conclusion, it can be seen that although the operating cost of Options 3 & 4 were higher than 1, 2 and 5, the total NPV was lowest for Options 3 and 4. Considerable cost savings were able to be effected through reducing the peak flows with the use of storage. This would not be the case with all projects, but the relatively long pipeline lengths and very high peak flow ratios had a significant impact in this case.

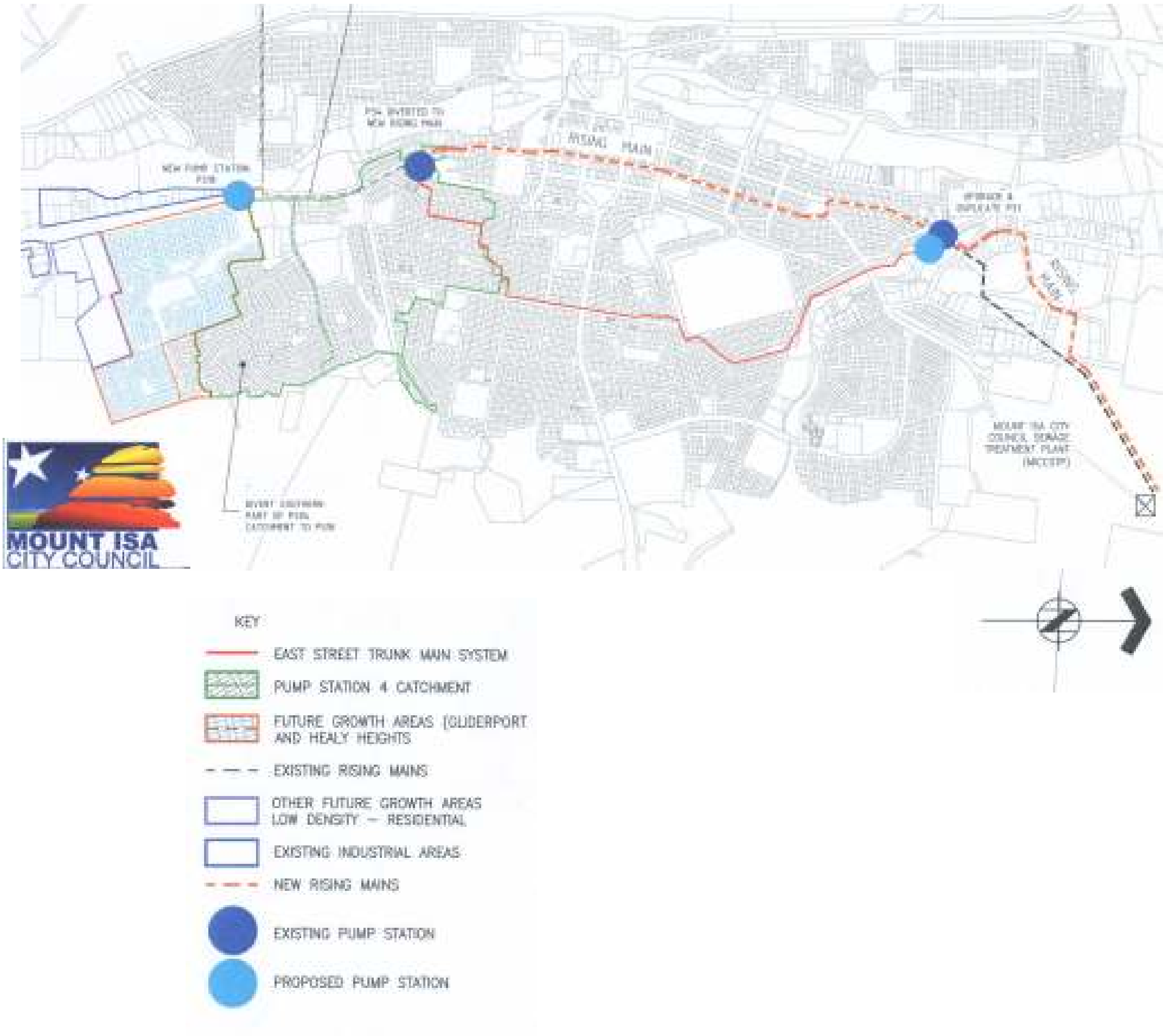
2.3 MOUNT ISA CITY WASTEWATER OPTIONS

This project involved upgrading the city's wastewater network to convey flows from proposed developments at Gliderport and Healy Heights to the south of the city of Mount Isa, in north-western Queensland. The sewage treatment plant is located to the north-east of the city, and all flows from the new developments would need to be conveyed through the existing city.

The existing wastewater network is already mostly at capacity with very high infiltration during heavy rainfall. Although the climate at Mount Isa is usually hot and dry, the area can be subject to intense tropical rain, particularly during the summer months, and this results in very high wastewater flows. The two main trunk sewers, the East Street Trunk and the West Street Trunk (both DN450) are already overloaded during heavy rain. In addition the main terminal pumping station, PS01, is at capacity and cannot cope with peak wet weather flows.

A solution had to be devised to not only convey flows from the new developments, but also to reduce overloading of the city's existing network. A plan of the city, and the proposed works, is shown below.

Figure 12: City Plan and Proposed Works



Several solutions were considered, including replacement of the existing East Street Trunk gravity sewer with a larger diameter pipe. However this option was considerably more expensive than building an additional rising main through the city.

The chosen option was to build a new pumping station, PS18 to serve the new developments at Gliderport and Healy Heights. A number of options were considered:

a) Pumping directly to the plant (7.3km),

b) Pumping directly to the main pumping station, PS01 (4.7km),

c) Pumping to the nearest main pumping station PS04 (1.2km).

An upgrade to PS01, which is currently overloaded would be required for all options, although Option a) would require a lesser upgrade (PS01 upgrading is not covered in this paper).

There were several other complicating factors with the city's wastewater network. PS04 pumps through a relatively short rising main to the East Street Trunk Sewer, adding to the flow in this already overloaded sewer. The network to the west of PSo4 & PS18 is also overloaded, and so PS11 (located to the west of PS18) could be diverted to PS18 to help relieve this overloading.

1. Pumping PS18 directly to the plant would not help to reduce the current overloading of PS04. An upgrade of PS04 would still be required, in addition to a new rising main for the total flow from PS18. Furthermore, the trunk gravity sewers would continue to be overloaded.

2. If PS18 flows were to be pumped to PS04, a major upgrade of PS04 would be required, in addition to a new rising main for the total flow. This option would help to reduce overloading in the East St Trunk Sewer.

3. If PS18 flows were to be pumped to PS01, PS04 and the East Street Trunk Sewer would still be overloaded. As for point 1 above, an upgrade of PS04 would still be required, in addition to a new rising main for the total flow from PS18.

The option that was finally selected was to pump PS18 directly to the main pumping station, PS01 (3.2km) and for PS04 to pump wastewater directly into the new rising main on the way to PS01. In addition, flows from PS11 can be diverted to PS18, to reduce overloading in the downstream catchments, and in the West St Trunk Sewer.

The advantages of this option are:

- Pumping PS18 will be a medium sized pumping station, but will be in a greenfield site, thus making construction relatively easy.

- The rising main from PS18 to PS01 will reuse an existing DN250 pipeline part of the way from PS18 to PS04.

- The southern part of the catchment of PS04 will also be diverted south to the new PS18, to reduce flows to PS04, and ensure that the existing pumps will be able to cope with ongoing flows.

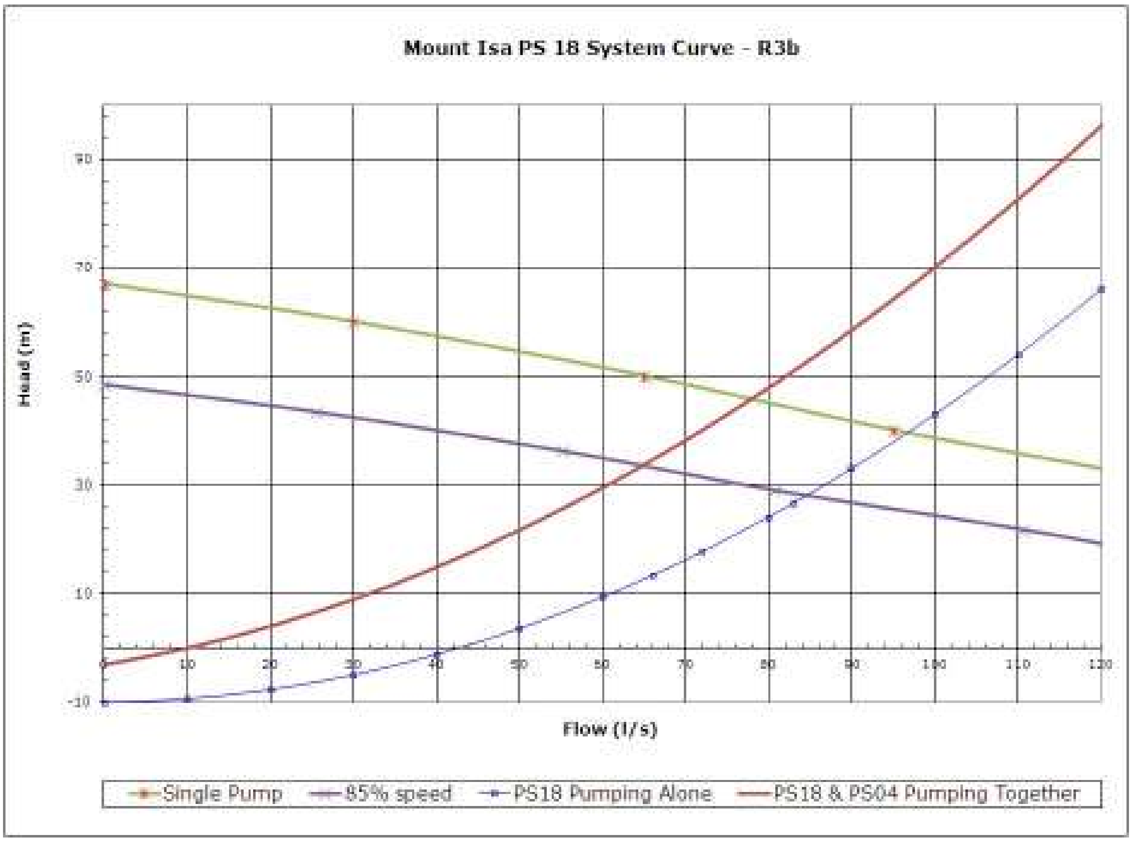
- No significant upgrade to PS04 will be required apart from putting the existing pumps on Variable Frequency Drive (VFD) to control the speed between 35 and 50 Hz. A more extensive upgrade of PS04 would be relatively difficult and expensive, as the existing wet well/dry well is too small to fit larger pumps, is relatively deep, and the site is in a residential/industrial area.

- The diversion of flows from PS04 to the new rising main will reduce flows in the overloaded East St Trunk gravity sewer, which would only require minor upgrading.

- The diversion of flows from PS11 to PS18 will reduce flows in the overloaded gravity sewer to PS06 and the West St Trunk gravity sewer, avoiding the need for major upgrading of this part of the city.

Pump operating curves for PS18 & PS04 are shown below for the case when PS18 is operating, PS04 is operating and when both are operating simultaneously.

Figure 13: Mount Isa PS18 System Curves



The effect on the operation of PS04 is more significant than on PS18, but both pumping stations are able to cope with all pumping scenarios.

While the design is currently being carried out, it is intended that the operation of PS18 and PS04 would by default assume on start-up that each pumping station is operating without the other one pumping. If the pressure is above a set level, and the flow below a set level, the control system would increase speed of the VFD to that required for simultaneous pumping. Thus the feedback loop between the two pumping stations would be hydraulic rather than relying on a complex telemetry link that could be subject to variable reliability at times.

The conclusion is that the selected option achieved overall cost savings and relieved the overloading of other parts of the network through optimizing the pumping arrangements.

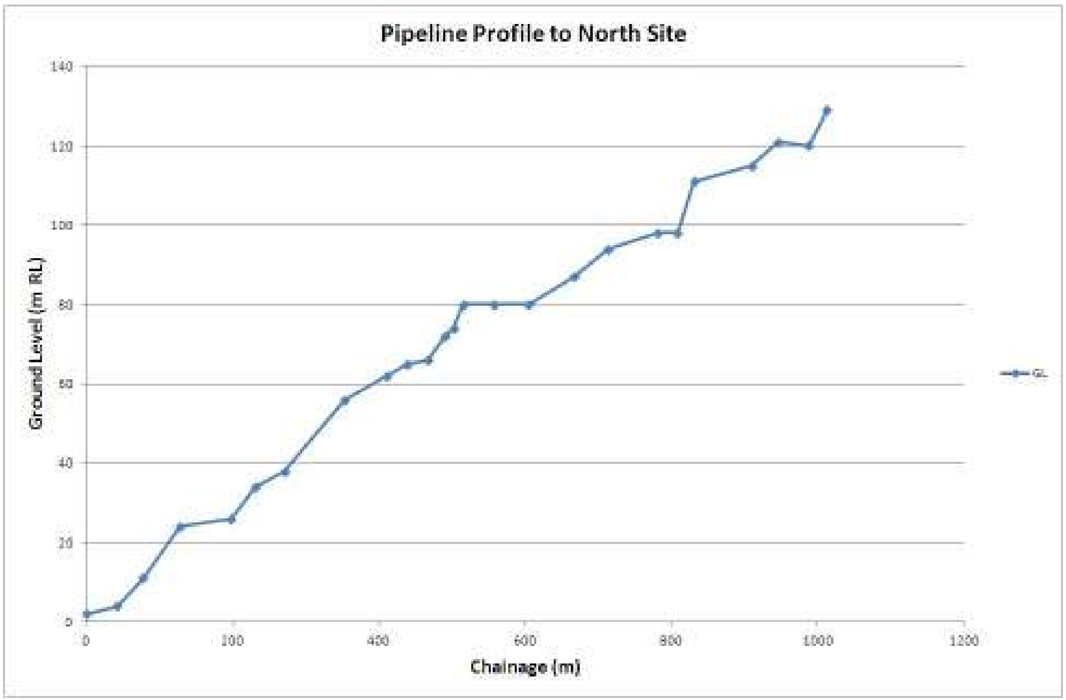
2.4 AKAROA

Harrison Grierson carried out a study to consider options for re-locating the WWTP for Akaroa. Two sites were selected, to the north or south of the town. Either option required a steep rising main with a static head of approximately 130m. It was proposed to screen the wastewater prior to pumping.

Figure 14: Akaroa Locality Plan

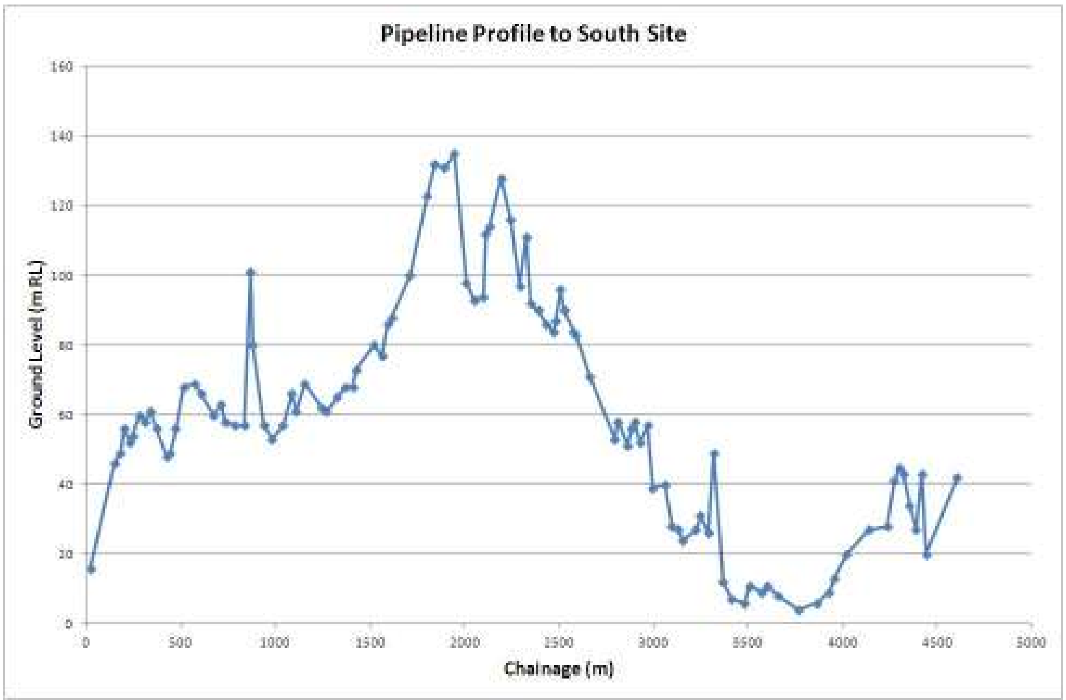
For the northern option a new WWTP to be located near the crest of a hill at 125m RL. The rising main would follow a very steep rise to the treatment plant, a graphical longitudinal section is given below:

Figure 15: Pipeline Profile to North Site



For the southern option, a new WWTP would be located on a relatively gentle sloping area of land at 45m RL 4.6km south of the town. The rising main would follow a very steep rise to a high point at 135m RL, followed by an undulating generally downhill section to a valley, then a rise to the treatment plant. A graphical longitudinal section is given below:

Figure 16: Pipeline Profile to South Site



For both options, the rising main would be a nominal 250mm internal diameter pipeline with a velocity between 1.1 to 1.5m/s. The calculated design pump head for both sites is similar at approximately 150 m, with most of the design head being static head.

The challenge of pumping the peak flow (55 L/s) along steep and undulating terrain is significant, due to the very high static head. The pumping arrangement would either comprise two large high head centrifugal pumps working in series, or two positive displacement pumps working in parallel. A comparison for the relative merits of each option is presented below.

Table 3: Akaroa Terminal Pumping Station Pump Options Comparison

	Positive Displacement pumps	Centrifugal pumps
Make	Mono	Flygt
Model	E1BDC11RPA	NP3315 HT 53-450-00-1150
Overall Duty	55 L/s @ 150m	55 L/s @ 150m
Single Pump duty	27.5 L/s @ 150m	55 L/s @ 75m
Speed	250 RPM	1480 RPM
Motor Rated Power	75kW	105kW
Number of pumps	3 (2 duty, 1 standby)	4 (2 duty, 2 standby)
Wet/Dry mounted	3 dry mounted	2 wet mounted, 2 dry mounted
Total Installed Power	3 x 75kW = 225kW	4x 105kW = 420kW
Power consumed	2 x 55kW	2 x 81kW
Operation with a single pump	27L/s @ 150m	Not possible
Advantages	Lower power consumption Simple parallel pipework arrangement 3 pumps required Lower electrical equipment cost due to lower installed power.	Centrifugal pumps lower maintenance
Disadvantages	Maintenance requirements of positive displacement pumps can be higher.	Complex, in-series pipework arrangement Significantly higher power consumption
Favoured Option	Yes	No

Due to the flow, 2 positive displacement pumps in parallel (a total of 3 pumps, 2 duty and 1 standby) are required to pump the design flow. Even if only one pump was operational, a lesser flow could be pumped for a limited period of time.

While centrifugal pumps in series could have been used, the pumps are quite large (105 kW), and a total of 4 pumps (2 sets of pumps in series for duty and standby) would be required. In addition, if only one pump was working, one centrifugal pump could not develop enough head to overcome the static head, and no flow could be pumped. In this case, the use of positive displacement pumps in parallel is more economic and more efficient than centrifugal pumps in series.

For the above reasons, positive displacement pumps have advantages compared to centrifugal pumps in series in this case and are the preferred option, being simpler to operate and having lower power consumption.

Both the north and south terminal pumping station buildings would be similar in terms of footprint and general configuration.

3 TECHNICAL COMPARISONS

The technical data associated with the examples above are given in brief here.

Table 4: Comparison of Pumping Systems

Project	Flow	Static Head	Total Head	Rising Main Length	Rising Main Pipe	Rising Main Diameter	Rising Main Velocity	Pump Speed	Comments
	L/s	m	m	m		ID mm	m/s	Hz	
Springhill to Te Kauwhata Pumping Stations and Rising Mains Project									
Te Kauwhata PS1	16.3	67.5	95 **Total**	200 400 400 1380 **2180**	180OD PE SDR11 160OD PE SDR11 160OD PE SDR13.6 160OD PE SDR17	146 130 136 141	1.0 1.2 1.1 1.05	47.5	High head pumps in series
Te Kauwhata PS2	16.3	-51	53* **Total**	2380 6450 **8830**	180OD PE SDR17 160OD PE SDR17	158 141	0.83 1.05	30-50 (varies)	Pump head and speed varies with fouling
Lyttelton Harbour Wastewater Study (Option 3 - Common Rising Main in Rail Tunnel)									
Flows when both Cyrus Williams Quay and Lyttelton pumping stations are pumping simultaneously:									
Cyrus Williams Quay PS to tunnel	36	23	67	1200	225OD PE SDR13.6	191	1.3	50	Both pumping stations pumping into common rising main
Lyttelton PS to tunnel	39	17	61	1400	225OD PE SDR13.6	191	1.4	50	
Common Rising Main, Rail Tunnel to Heathcote PS	75		**Total**	3000 **4400**	315OD PE SDR13.6	267	1.3		
Flows when Cyrus Williams Quay or Lyttelton pumping stations pump separately:									
Cyrus Williams Quay PS to Heathcote PS	48	23	64 **Total**	1200 3000 **4200**	225OD PE SDR13.6 315OD PE SDR13.6	191 267	1.67 0.85	50	Both pumping stations operating separately
Lyttelton PS to Heathcote PS	51	17	59 **Total**	1400 3000 **4400**	225OD PE SDR13.6 315OD PE SDR13.6	191 267	1.8 0.9	50	
Mount Isa City Wastewater Options									
PS18 & PS04 pumping simultaneously:									
Mount Isa PS18	66	-10	35	1200 3550	DN250 PVC & PE DN300 DI & PE	249 300	1.35 1.7	44	Both pumping stations pumping into common rising main
Mount Isa PS04	52	-7	31	50 3550	DN200 DI DN300 DI & PE	195 300	1.7 1.7	42.5	
PS04 & PS18 pumping separately:									
Mount Isa PS18	66	-10	35	1200 3550	DN250 PVC & PE DN300 DI & PE	249 300	1.35 1.7	44	Both pumping stations operating separately
Mount Isa PS04	52	-7	31	50 3550	DN200 DI DN300 DI & PE	195 300	1.7 1.7	42.5	
Akaroa Wastewater Study:									
Akaroa Northern WWTP option	55	127	149 **Total**	350 310 350 **1010**	280OD PE SDR9 280OD PE SDR9 280OD PE SDR9	215 226 238	1.5 1.4 1.2	50	Positive Displacement pumps
Akaroa Southern WWTP option	55	138	152 **Total**	1370 340 750 2140 **4600**	355OD PE SDR9 315OD PE SDR11 280OD PE SDR13.6 280OD PE SDR17	273 256 238 246	0.94 1.1 1.2 1.15	50	Positive Displacement pumps

4 CONCLUSIONS

While pumping and conveyance of wastewater can be a relatively straightforward matter, there are often situations that present significant technical challenges to successful operation. In addition, there may be opportunities for cost saving synergies that can be realised with innovative and forward-thinking design, without excessive risk taking.

This paper has shown that alternative systems, though somewhat innovative, can overcome technical challenges and achieve real cost savings and operate as successfully as any conventionally engineered system. The lesson is that investment in options evaluation and pre-design investigations into potential cost-saving solutions have the potential to save money or alleviate other problems, typically saving many times more than the cost of the additional investigation.

The investment in additional investigation is nearly always worthwhile in reducing capital and/or operating expenditure, and risk, while increasing the value and benefits of the system.

ACKNOWLEDGEMENTS

We express our gratitude to the following persons and organisations for the assistance that was given during these projects, and for permission to present information in this paper:

Mervyn Sumanaratne and Richard Bax of Waikato District Council.

Emilio Cianetti of Mount Isa City Council, Queensland, Australia.

Mike Bourke and Zefanja Potgieter of Christchurch City Council.

REFERENCES

Metcalf & Eddy, 1981: Wastewater Engineering: Collection and Pumping of Wastewater