DESIGNING PRESSURE SEWER SYSTEMS FOR TOPOGRAPHICALLY CHALLENGING CATCHMENTS: AN ACCOUNT OF THE OPUA PRESSURE SEWER SCHEME

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ABSTRACT

This paper will present the challenges faced in designing a reticulated sewer system for Opua, and how those challenges were overcome in the design of a pressure sewer system to reticulate the township.

Opua, in the Bay of Islands, New Zealand, presents significant challenges to the design of a reticulated wastewater system. The area consists primarily of bush covered steep hills and gully's, predominantly with narrow roads running along ridge lines and conservation reserves in the gullies. The area has poor geotechnical conditions with either hard rock or thin soils on steep slopes which are prone to slipping when subjected to high rainfall, or disturbance such as that from pipe laying.

This paper will outline the use of best practice design methodologies in both the hydraulic design of the pressure sewer, and the over-arching design process of optioneering and final design configuration, resulting in a successful 'design-build' tender. The benefits of a pressure sewer in steep and rocky terrain will be highlighted and critically compared to the attributes of a gravity sewer network with multiple pump stations. Pump station design innovations will also be show-cased.

This paper will present the beneficial attributes of a pressure sewer that may be applied to other geographically challenged sites in New Zealand for which an affordable reticulated sanitary sewerage system is required.

KEYWORDS

Pressure Sewer, Difficult Terrain, Energy Efficiency, Progressive Cavity Pumps, Innovative Design Solutions.

1 INTRODUCTION

Opua has been developed on steep terrain, bounded by the Waikare Inlet waters on the eastern side and rising to ridges over 80 m above sea level on its western edge. Much of the area is bush covered and zoned conservation. Because of the steepness of the terrain, the potential for land slip, and the need to retain vegetation cover, the overall density of the development is sparse and tends to be concentrated along roads following ridgelines.

The majority of the community has been serviced by wastewater onsite disposal systems, with the exception of the Marina area and Beechy Street, running along the foreshore. Combined with the poor geography of the area, the disposal of wastewater via onsite disposal fields has been attributed in part to contributing to significant landslides, particularly along Sir George Back Street during storm events on the 29th March 2007. Sir George Back Street itself was serviced by a pressure sewer following the slip events while investigations were carried out regarding the reticulation of the remainder of Opua.

Initial designs focused on traditional gravity wastewater reticulation. Funding was applied for via the Sanitary Works Subsidy Scheme (SWSS) programme. The initial gravity design approved for SWSS funding comprised of:

- 5,650 m conventional gravity wastewater pipes
- 2,931 m rising mains
- 4,170 m private wastewater lateral
- 14 conventional pumping stations

Following the successful implementation of a pressure sewer in Sir George Back Street, the decision was made to utilise the positive attributes of a pressure sewer across the wider Opua catchment. The ability of pressure sewer to be laid to minimum ground cover rather than to grade was significant in avoiding the risk of encountering rock close to the surface. The ability to transfer flows uphill by pump instead of downhill by gravity also removed the issue of laying gravity pipes across unstable slopes.

The procurement of a pressure sewer scheme for Opua was eventually gained by a design build contract. United Civil Construction and GHD partnered for the successful design build tender, with the network consisting of:

- 6,465 m pressure sewer pipe, between 50 mm OD and 90 mm OD
- 626 m rising main
- 1 pump station incorporating benefits achievable from the attributes of pressure sewer
- 182 separate property connections, each with a pressure sewer tank and grinder pump.

2 TOPOGRAPHICAL & HYDRAULIC CONSTRAINTS OF OPUA

The topography of Opua is steep with significant elevation changes, refer to Figure 1 below. A number of properties front the beach and in effect have elevations little more than sea level. The required discharge point for the reticulation is at an elevation of 78 m, at an existing manhole in Paihia Road near the junction with Oromahoe Road. Surveyed grinder pump elevations range from 2.2 m on English Bay Road, to 88.4 m on Oromahoe Road.

Opua is interspersed with ridgelines dropping away steeply to gullies and the marine foreshore. Roads generally run along the ridgelines with properties positioned near the roads at the top of ridges. A number of gullies are classified as conservation areas. As such the tender documents required pipes were not routed through these areas. This restricted possible pipe routes to public roadways, limiting options for the designer and requiring significant elevation changes along viable pipe routes.

Commercially available domestic grinder pumps have maximum continuous operating head limits in the order of 60 m. The grinder pump selected for use in Opua was the E-One Extreme Series with a maximum continuous operating head limit of 58 m. Clearly it is not hydraulically possible to configure the Opua reticulation to operate without the use of one or more booster stations within the network. The final design resulted in one centrally located booster station being required for the network.

Figure 1: Opua Street Plan and 5 m Contour Levels

3 OPTIONEERING PROCESS – FINAL CONFIGURATION

3.1 HYDRAULIC DESIGN

The E-One Extreme series grinder pump was selected for the Opua reticulation, giving a maximum continuous operating head of 58 m. The designer must then balance the Total Dynamic Head (TDH) to be not more than 58 m on any pump in the system. TDH consists of static head and accumulated dynamic head. A simple trade-off must be made between static and dynamic head, being the greater the static head present, the less dynamic head can be allowed. The greater the component of dynamic head available the greater the distance that can be pumped by private grinder pumps. In addition, the designer must balance the selected pipe size to ensure self-cleansing velocities are reliably achieved, whilst ensuring accumulated dynamic head does not cause the TDH to exceed 58 m.

Effective utilisation of available dynamic head allows the designer to produce a design that reduces residence time to the absolute minimum, and gives a high degree of reliability in achieving predicted flow velocities. This ensures the network will consistently achieve self-cleansing velocities. Therefore, by effectively utilising available dynamic head, the designer can produce a network that significantly reduces the occurrence of solids retention and septicity, preventing odour and corrosion issues at air discharge points and within downstream infrastructure.

For this reason careful attention was paid to the selection of viable potential locations for booster stations, in order to reduce static head in sections of the network, and to minimise pipeline lengths, both to reduce cost and to allow greater flexibility in pipe sizing within the constraints of the hydraulic parameters of 58 m TDH, and the need to achieve self-cleansing velocities.

3.2 OPTIONEERING

The tendering design team carried out an optioneering exercise that was approached in a collaborative but also competitive manner, i.e. individual designers were working to produce the ‘best idea’, to then share with the team to collaborate on the strongest viable options. Three viable options for reticulation of Opua were generated with, in some cases, significantly different approaches to achieve the design solution.

These three options were presented to the Contractor, United Civil Construction Ltd to encourage valuable constructor’s experience input into the final design solution. The options were assessed for personal preference (i.e. construction methodology well suited to the Contractor’s experience and expertise), risk, capital cost and associated operational costs.

Due to the additional capital cost, as well as operational and maintenance cost incurred by each additional pump station, the design team focused on a configuration that required only one booster station without incurring excessive additional costs, nor undue risk during construction or operation.

3.3 PREFERRED NETWORK CONFIGURATION

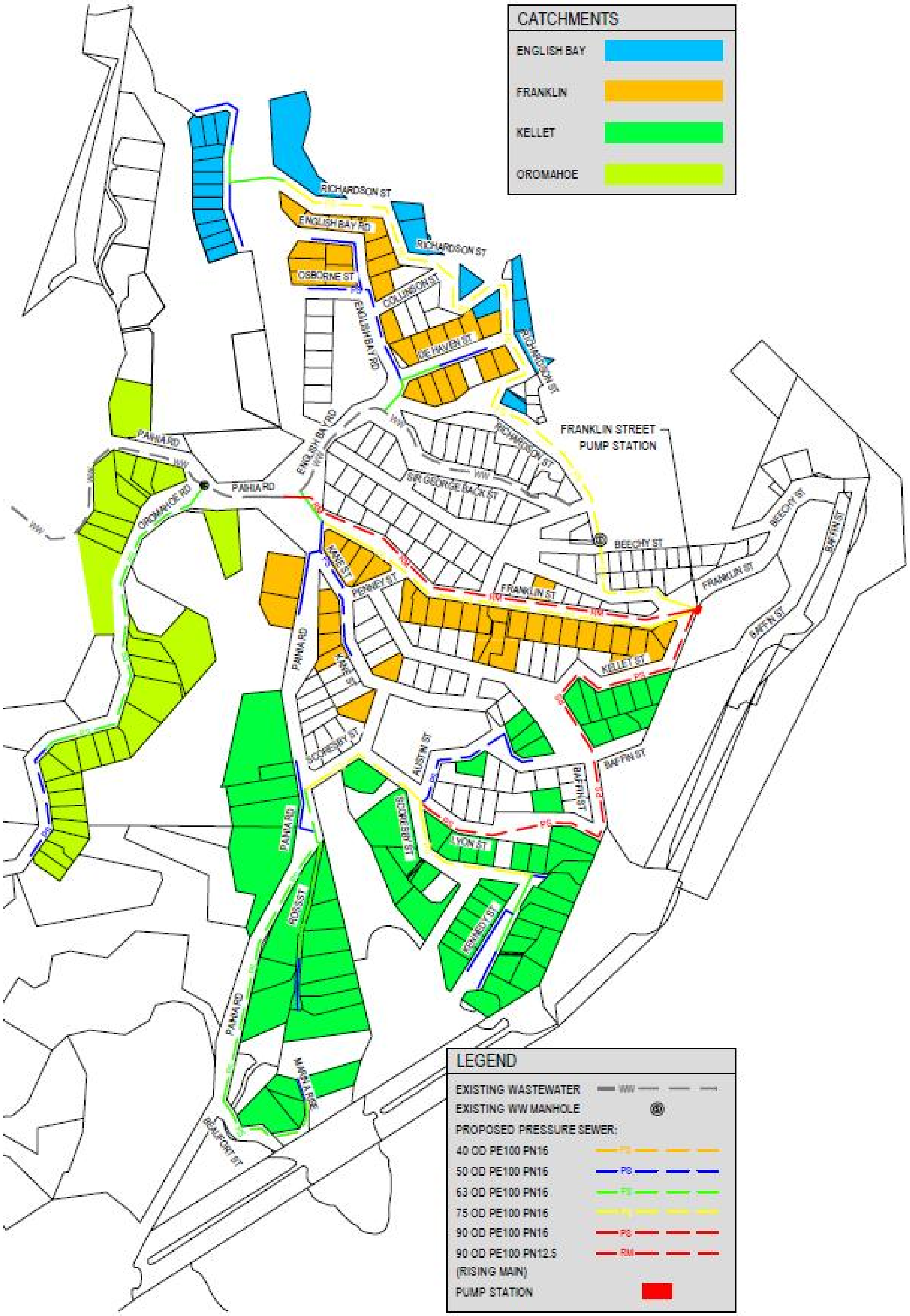
Three viable locations were identified for pump station locations. Each location had existing power supply close by and sufficient space for construction and maintenance within the public road reserve without impinging on the carriageway. The network configuration finally selected is centered on a pump station sited in a large gravel area at the junction of Franklin Street and Kellet Street. The elevation of this site is 38 m RL, which provides pumps at sea level (2.2 m RL at English Bay) to be able to effectively discharge to the pump station, with a sufficient component of dynamic head to achieve self-cleansing velocities.

The Opua catchment was then divided into four sub catchments (refer Figure 2 below) named after the primary road in the sub catchment.

- Oromahoe Sub Catchment, 32 Equivalent Tenements
- English Bay Sub Catchment, 25 Equivalent Tenements
- Franklin Sub Catchment, 70 Equivalent Tenements

- Kellet Sub Catchment, 84 Equivalent Tenements

Figure 2: Opua Catchment Plan



Note: Equivalent Tenements (ET's) do not represent the numbers of properties connected, but the wastewater flow expressed based on the daily loading rate for the design, which in this case is 800 litres per household per day. Properties with higher than normal occupancy, such as B&B's, hotels and schools are expressed as a certain number of equivalent tenements in the design to represent the higher loading rates. In most cases B&B's etc will receive a duplex grinder pump unit and the single connection will count as 2 ET's.

3.3.1 OROMAHOE SUB CATCHMENT

The Oromahoe sub catchment is the only catchment that does not discharge to the pump station. Oromahoe Road runs along a high ridgeline, meeting Paihia Road at an elevation of 78 m. Surveyed grinder pump elevations along Oromahoe Road range from 60.7 m to 88.4 m.

The discharge point for the entire Opua catchment is at an existing manhole in Paihia Road at the junction with Oromahoe Road. This manhole is at the head of an existing PVC trunk main that runs along Paihia Road towards Paihia and discharges into the existing Bay of Islands wastewater infrastructure.

Therefore Oromahoe Road was designed as a separate sub catchment discharging directly to the head of the existing rising main.

3.3.2 ENGLISH BAY SUB CATCHMENT

The English Bay sub catchment was perhaps the most challenging to the designers in achieving acceptable reliability in self-cleansing velocities while maintaining TDH on every pump under the 58 m limit. The sub catchment has a total of 25 ET's, 14 of which are in English Bay itself with elevations ranging from 2.2 m to 9.6 m, and a total length of pipeline to the discharge point of 1,490 m.

The pipeline leads out of English Bay itself, along Richardson Road East, Richardson Road West and directly up to the pump station on Franklin Street. Paper road routes have been utilised between Richardson Roads East & West, and from Beechy Street up to the Franklin Street Pump Station. The paper road section of Richardson Road is formed, but overgrown with regenerating scrub. The identification of the formed paper road enabled a viable pipe route from English Bay to Franklin Street and removed the requirement for a second booster station on English Bay Road to lift flows from near sea level through to elevations of a minimum of 68 m RL, or up to the discharge of 78 m RL.

The identification of the formed paper road is evidence of the value of a thorough site investigation during the optioneering process.

3.3.3 FRANKLIN SUB CATCHMENT

The Franklin sub catchment services properties on English Bay Road above the 38 m contour, de Haven Street, Sir George Back Street, Kane Street and Franklin Street itself, discharging to the pump station at the bottom of Franklin Street (at the Kellet Street intersection).

Sir George Back Street (SGBS) is an existing pressure sewer system that has been operating successfully for several years. The line starts as 40 mm OD at the very bottom of SGBS, changing to 63 mm OD and then 90 mm OD on SGBS. In its current configuration the 90 mm OD line runs along English Bay Road through an elevation of 68 m and continues up Paihia Road to the discharge manhole at the intersection of Oromahoe Road, at an elevation of 78 m RL, where it discharges.

The Franklin sub catchment incorporated the existing SGBS system, specifically adopting the existing 160 m length of 90 mm OD on English Bay Road. Predicted peak flowrates in this section of line increase due to the inclusion of flows from properties on English Bay Road and de Haven Street, which increases the dynamic head losses in the line and observed by the existing pumps on SGBS. To counteract this, the discharge for the zone is at the Franklin PS, elevation 38 m RL, rather than the existing discharge manhole at elevation 78 m RL. The highest point on the line from SGBS is 68m RL on English Bay Road, rather than the 78 m at the discharge manhole. This reduces the static head observed by SGBS pumps by 10 m, comfortably compensating for the increased dynamic head losses in this section of line.

3.3.4 KELLET SUB CATCHMENT

The Kellet sub catchment is the largest in the Opua network with 84 equivalent tenements, and also demonstrates some challenging topography with properties at relatively low elevations having to pump up significant elevation changes and along relatively long distances. The highest point on the discharge line is 41 m RL on Scoresby Street, however properties on both Ross Street and Kennedy Street have grinder pumps above this elevation.

The most hydraulically disadvantaged pumps are on Marina Rise, in the very south of the catchment, and at the bottom of Kennedy Street. Road access to Marina Rise is via SH 11 – Paihia Road. Several options for servicing Marina Rise were investigated including crossing private land to the top of Ross Street. However elevations in the order of 45 m at the top of Ross Street introduced unacceptable Hydraulic Grade Lines (HGL's). Construction methodologies identified to address the unacceptable HGL's were felt to incur excessive project risk. The project team eventually concluded the least cost and least risk option was to lay pipeline along SH 11 from Marina Rise to Scoresby Street, a distance of approximately 500 m.

Kennedy Street has properties with surveyed grinder pump elevations as low as 11.4 m RL. The high point of Kennedy Street passes through 40.5 m at the edge of the carriageway, giving grinder pumps nearly 30 m static head. While this amount of static head is less than some properties observe on the English Bay sub catchment, the significantly greater flows generated by the larger population on the Kellet sub catchment must be considered. The greater flows require careful analysis be carried out to ensure dynamic head losses do not cause TDH to exceed 58 m at time of peak flow. The design team adopted a methodology of analysis of hydraulic grade lines to ensure the hydraulically disadvantaged pumps on both Marina Rise and Kennedy Street operate at below 58 m TDH.

4 PUMP STATION INNOVATIONS

The central pumping station in the network, the 'Franklin PS' contains two design features that are directly facilitated by the attributes of pressure sewer systems over conventional gravity systems and conventional pump stations with centrifugal pumps. These features reduce capital cost and time for construction, significantly reduce future operational costs through lower electricity consumption and completely remove the risk of overflows at the pump station.

The pump station consists of a 1.8 m diameter wet well, approximately 3 m deep, manufactured from standard precast concrete sections. Emergency storage of 10 m^3 is provided, manufactured from precast concrete stormwater culverts. Twin, dry mounted, progressive cavity pumps are provided in a shallow concrete chamber. The high head pumps driving a small diameter rising main offer very good hydraulic characteristics. A manifold and valve chamber is provided at the pressure sewer pipelines discharge to the pump station. A simple Montrose style electrical cabinet houses the control system and SCADA transmission unit.

Two innovations have been incorporated into the design that ensure that the design offers a low capital cost, low running cost pump station with zero risk of overflow

4.1 INNOVATION ONE – FAIL-SAFE VALVES

The Opua pressure sewer system contains 100 m^3 of emergency storage in the on-property grinder pump tanks, equivalent to approximately 18 hours storage at the design loading rate of 800 l / household / day. Traditionally pump stations require a volume of emergency storage to meet the relevant local authority containment standards. This varies by local authority but is often in the order of 12 hrs storage. In the example of the Franklin Street Pump Station, this volume would have been in the order of 68 m^3, which would have incurred significant capital cost, in addition to risk of odour, and still not entirely removed the risk of overflow from the pump station occurring.

To effectively utilise the 100 m^3 of emergency storage in the on property grinder pump tanks, a fail-safe valve was installed on the inlet to the Franklin Street Pump Station.

The Franklin Street Pump Station itself contains 10 m^3 of emergency storage above the pump on level. This consists of 3 m^3 in the wet well and a separate 7 m^3 tank. The storage at the pump station has been provided to ensure that at times such as power outage recovery situations, when station inflow may be higher than pump delivery capacity and flow attenuation is required, ample storage volume above normal pump operating levels will be available to reduce the cycling frequency of the fail-safe valve and pump starts to a safe level. Calculations confirm the expected minimum cycle time for the fail-safe valve is very acceptable, and pump starts are minimised in both normal operation and in ‘recovery’ events, such as a re-start after a network shut down or an extended power outage.

4.1.1 OPERATION OF THE FAIL-SAFE VALVE

The fail-safe valve will be triggered to close by a high wet well level alarm. Battery back-up is provided to enable valve closure during periods of loss of mains power. The fail-safe valve will reopen automatically once wet well levels have reduced to normal operating levels. In reality this is set at ‘standby pump on’, meaning the wet well level will be reduced by both duty and standby pumps running. The fail-safe valve will reopen at the point the wet well is at its lowest, but before the standby pump would switch off. This means that during network recovery events, both duty and standby pumps operate continuously until the network has fully recovered. Storage volumes at the pump station provide attenuation for flows so that the fail-safe valve has a relatively long period between operations.

GENERAL POWER FAILURE:

In the event of loss of power to the Opua area, power will be lost to all homes and to the pump station. No on-site pumps will be able to operate, hence there will be no inflow to the pump station and emergency storage on each property already built into the standard tanks will be utilised in this scenario.

SPECIFIC POWER FAILURE OR MAINTENANCE SHUTDOWN EVENT AFFECTING THE FRANKLIN STREET PUMP STATION ONLY:

This scenario covers events such as the loss of power to the station; major station failure; and for planned maintenance shutdowns, noting that with twin pumps and check valves, the need to shut down both pumps is remote.

In this scenario, the on-site pumps would continue to operate as normal, but the Franklin Street Pump Station would be unable to operate. In this circumstance the fail-safe valve at the outlet of the collector mains will be automatically closed so the inflow to the pump station is completely shut off. The fail-safe valve will be triggered by a high wet well alarm. Any on-site pump in the network running at the time or subsequently trying to start will experience a shut off head and will shut down until such time as the fail-safe valve is reopened, the pressure in the system falls and the pumps can automatically resume normal operation and pump out as required.

RECOVERY FROM EXTENDED POWER FAILURE*:*

On restoration of power following an extended power outage, many on-site pumps are likely to re-start simultaneously to pump down their tanks. The analysis scenario adopted for design is the loss of power to the entire catchment for 24 hours or longer. Under this scenario it can be assumed that all on-site emergency storage will be used, and all pumps will try and start on restoration of power. The recovery of the pressure sewer network in this situation must be simple (not requiring operator intervention) and timely.

This event can give greater inflows to the pump station than the design daily peak flow, and greater than the pump out rate. GHD estimate possible inflows in the order of 30 l/s, maintained for short periods of time. This is significantly above total pumping capacity and well above the expected capacity of downstream transfer infrastructure from the existing transfer main on Paihia Road and all the way to Paihia.

In this event the water level in the pump station wet well will continue to rise above normal pump start. The pump station emergency storage will then be utilised, which may be sufficient in many events for the pump station to manage flows without the fail-safe valve intervention.

In the event that a ‘high level’ alarm level is reached in the wet well, the fail-safe valve will close, stopping all flow into the wet well and allowing the pumps to rapidly drop the wet well level. Upon the level reaching the normal standby ‘pump on’ point, the fail-safe valve will automatically open, allowing flow back into the wet well and ensuring pumps continue pumping at maximum capacity until such time that the wet well is emptied and ‘pump off’ level is reached.

4.1.2 POWER OUTAGE RECOVERY TIMES

The time taken for the network to 'recover' from a 24 hour power outage, or any other 24 hour shutdown, is determined to be approximately 7 hours.

This is based on an automatic recovery and does not require operator intervention. Considering the downstream capacity constraints, the ability of the network to recover in less than 12 hours is considered highly desirable. A recovery time of approximately 7 hours is considered acceptable.

4.1.3 EFFECT OF FAIL-SAFE VALVE ON THE ON-SITE PUMPS

The E-One pumps to be installed in this design are fitted with well proven over pressure protection. Experience has shown the E-One over pressure protection reliably protects pumps and stators from damage in over pressure situations. E-One also boasts a unique pump re-start randomiser based on the last digit of the pump serial number. This means that on re-start pumps will be staggered, so a small number of pumps start to pump out at one time.

4.2 INNOVATION TWO – PROGRESSIVE CAVITY PUMPS

While progressive cavity pumps themselves are not a new innovation, they are well suited to match the attributes of pressure sewer systems. Progressive cavity pumps are not well suited to high peak flowrates but are very well suited to high-head applications with moderate to steady flowrates. With pressure sewer systems effectively excluding inflow & infiltration, additional pumping capacity to meet peak wet weather flow events is not required.

A large number of applications that are well suited to pressure sewer systems are also a significant distance from discharge points or the final wastewater treatment plant location. By adopting high-head pumps such as progressive cavity units, the low peak flow attributes of a pressure sewer can be capitalised on by being able to install smaller diameter rising mains, and/or by being able to pump much further without the need for costly booster stations. Smaller diameter rising mains have the added benefits of lower capital cost and much lower sewage residence times. In addition, self-cleansing velocities can be achieved much more easily in smaller diameter rising mains. This, combined with reduced residence times, means the quality of wastewater at the discharge point can be significantly improved. This can prevent or reduce septicity and associated odour and corrosion issues throughout the downstream infrastructure.

The static head between the Franklin Street Pump Station and the discharge manhole on Paihia Road is 52 m. The design maximum peak flowrate is 7.2 l/s. A pair of Mono Epsilon E16B progressive cavity pumps have been specified for this duty point in a 'duty – assist' configuration.

Capitalising on the high head capabilities of the progressive cavity pumps, a 90 mm OD PE rising main was specified, running from the Franklin Street Pump Station to the discharge point on Paihia Road. At peak design flowrates this size rising main adds approximately 20 m dynamic head loss to the system, giving a TDH of approximately 72 m for the progressive cavity pumps. This duty point, 72 m TDH at 7.2 l/s, is difficult to achieve with centrifugal pump options.

4.2.1 ELECTRICITY CONSUMPTION OF THE PROGRESSIVE CAVITY PUMPS

Progressive cavity pumps offer improved power efficiency over comparable centrifugal pumps.

Each specified pump is equipped with a 7.5 kW motor. The motors have been sized larger than the 5.5 kW's required for direct online starting to facilitate the use of variable speed drives (VSD's) at the pump station.

The absorbed power of one pump at maximum duty point is 3.73 kW.

In duty - assist configuration, 2 pumps will be running, pumping 7.2 l/s and consuming 7.46 kW.

The average dry weather flow for the Franklin St PS is 1.57 l/s.

Utilising the VSD's means that 'follow-the-flow' pumping can be adopted to gain further energy savings, and to reduce the number of pump starts and stops. This has flow on benefits for air and odour management, meaning that air is not forced out of the line rapidly on each pump start, causing a potential odour and nuisance issue.

Table 1: E16 B Progressive Cavity Pump consumed power at various flowrates

Pump Speed	Flowrate	Consumed Power
21 Hz	1.42 l/s	1.27 kW
29 Hz	2.02 l/s	1.82 kW
40 Hz	2.87 l/s	2.73 kW
50 Hz	3.6 l/s	3.73 kW

By running a 'follow-the-flow' pumping system, the progressive cavity pumps can physically pump the average dry weather flow. Under normal applications, centrifugal pumps sized for peak wet weather flow would not be able to pump the actual average dry weather flow and would accumulate volume in the wet well prior to pumping it out at a higher flowrate.

Operating at 21 Hz, 1 kWh can transfer 4.025 m^3 of wastewater.

If a simple soft-starter mechanism was used, requiring the pump to run at 50 Hz whenever running, 1 kWh will transfer 3.475 m^3 of wastewater. The use of VSD's and follow-the-flow pumping represents a potential reduction in electricity consumption of 16%.

5 CONCLUSION

Providing wastewater reticulation to the township of Opua posed unique challenges. The topography, geotechnical conditions, sparse population and proximity of both the marine environment and conservation areas attest to this. A pressure sewer system successfully addressed the challenges of providing Opua with reticulated wastewater. The design for the Opua pressure sewer system incorporated innovations in pump station design. The use of fail-safe valves on the pump station inlet enabled on site storage built into the tanks to be effectively utilised, meaning additional emergency storage at the pump station was not required. The low peak flow attributes of the pressure sewer reticulation meant that progressive cavity pumps could be utilised efficiently at the pump station. Progressive cavity pumps have beneficial attributes of high head capabilities, and very good energy efficiencies. Both of these attributes were utilised to the benefit of the community of Opua. Pressure sewer systems are a reliable and effective way of providing wastewater reticulation to communities with steep and difficult terrain, and with sensitive natural environments.

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