AN INTRODUCTION TO THE CENTRAL PLAINS IRRIGATION SCHEME

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ABSTRACT

Stage 1 of the Central Plains Water Scheme will irrigate approximately 20,000 hectares of farmland in the Canterbury Plains in an area bordered by the Rakaia and Hororata Rivers. An intake on the Rakaia River will direct water into a 17km long gravity fed headrace. A network of underground pipes will then distribute water from the headrace to over 20,000ha of farmland. The key components of Stage 1 are:

- An intake and headworks at the Rakaia River to bring water into the headrace.
- A headrace alongside and traversing 'up' the northern bank of the Rakaia River to the top of the main Rakaia Terrace.
- A level headrace across the plains to convey water north and into the reticulation system.
- Four piped reticulation networks providing irrigation water to all shareholder farms in the scheme area.

The paper will discuss the design and construction of the four reticulation networks.

The key deliverables of the scheme will be presented, followed by an outline of the numerous and complex design challenges inherent in delivering those requirements.

The paper will also touch on the feasibility and planning stages of the whole Central Plains Scheme, as well as some of the innovative construction methods that have been employed.

KEYWORDS

Central Plains, Irrigation, Planning, Design, Construction

1 INTRODUCTION

Central Plains Water Limited (CPWL) wish to supply their farmer shareholders with irrigation water via a piped reticulation system, fed from the CPWL headrace canal.

CPWL engaged Downer New Zealand Ltd to provide the design, supply, construction, and commissioning for this piped irrigation distribution system.

The Downer consortium is composed of Downer Group, MWH New Zealand Ltd (MWH), Aquaduct New Zealand Ltd (Aquaduct), Millennium Electrical, and Rubicon.

2 SCHEME OVERVIEW

The scheme lies between the Southern Alps to the west, and SH1 and the Waimakariri and Rakaia Rivers, lying entirely within the Selwyn District Council boundary. The irrigation scheme has been designed for 60,000Ha, with potential to irrigate up to 80,000Ha if more water is available. This will be one of the largest construction projects ever undertaken in the South Island.

The scheme will utilise river water from both the Rakaia and Waimakariri Rivers. The rivers will be linked by a 56km headrace canal running around the foothills and will channel water via around 500km of piped reticulation. Irrigable land above the canal can be watered via pumped systems.

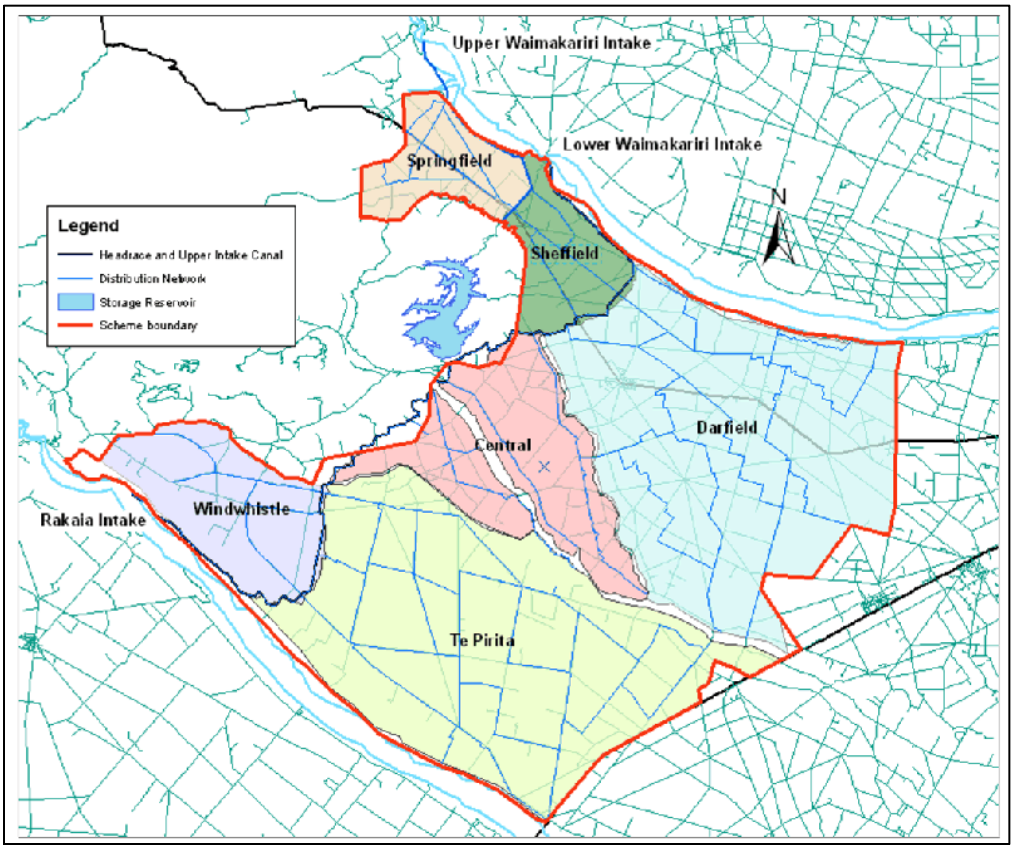

Figure 1: CPW 'command area'

Stage 1 of the scheme irrigates an area of 20,000 ha between the Rakaia and Selwyn Rivers. Stage 1 is undergoing commissioning and is expected to be operational by the 2015/2016 irrigation season.

2.1 RELIABILITY & STORAGE

Reliability of water supply is the key goal of irrigation. The advantages of high reliability are:

- The most efficient use of water – just enough as and when needed, not as much as possible whenever available

- flexibility in land use
- high value crops can be grown with certainty
- encourages investment – investors require certainty

The initial CPWL scheme proposed a 280million m^3 water storage reservoir in the Waianiwaniwa Valley. This would have provided 99% reliability for the CPWL scheme.

Commissioners advised in April 2009 that they would not be approving the reservoir on the ground that its effects on the community and environment would be more than minor, and CPWL was given the opportunity to propose a revised run-of-river scheme. This revised scheme was presented in October 2009 and commissioners granted consents on 1st June 2010 for both Rakaia and Waimakariri takes.

CPWL is now pursuing a number of alternative options to secure storage for the scheme, including:

- Reaching agreements with existing consent holders of Band two and three consents on the Rakaia River; water sharing arrangements with Barrhill Chertsey Irrigation Limited for temporary use of band four Rakaia River water, continued use of groundwater.
- Co-operation through water user groups on the Rakaia River and Waimakariri River; and
- Arrangements with TrustPower Limited to store water in Lake Coleridge.

A number of other options for water storage are available to shareholders individually or in small clusters. These include on farm storage of water, the use of ground water to augment water from the scheme, and ground water storage.

3 STAGE 1 PIPE DISTRIBUTION NETWORK

3.1.1 WHY PIPES?

The benefits of a piped distribution network are;

- Pressurised water – no 'on-farm' pumping
- Minimal disruption to land (access)
- Reduced water losses – mainly seepage
- Enhanced control

3.2 KEY DELIVERABLE - FLOW & PRESSURE

The distribution system is required to provide the following two duties of pressure against flow immediately downstream of each connection:

- 40m pressure (relative to ground level at the property) at a flow of 0.5 l/s/ha
- 35m pressure (relative to ground level at the property) at a flow of 0.6 l/s/ha

We approached these requirements as two discrete design scenarios: the first delivering a flow of 0.5 l/s/ha at 40m pressure only to all turnouts in the system, and the second delivering a flow of 0.6 l/s/ha at 35m pressure only.

We did not consider any 'split duty' operating condition where an indeterminate number of properties receive 0.5 l/s/ha at 40 m, whilst the remainder receive 0.6 l/s/.ha at 35 m <u>at the same time</u>. Given there are 127 farmer connections, and two possible duties at each, so the number of possible operating conditions is:

$$2^{127} = 1.7\text{x}10^{38}$$

To (briefly) put that number into context, it is approximately 1.7 trillion times larger than the estimated number of stars in the observable universe. Given that modelling the hydraulic performance of the entire system at the two key design scenarios takes about 5 seconds on a standard issue laptop, the concept of considering every possible permutation quickly becomes unfeasible.

There are probably ways to determine something close to the worst case without looking at every possibility, but this approach was not adopted.

3.3 DEMAND DIVERSITY

Municipal potable water reticulation systems are designed in the knowledge that not every person using it will want water at exactly the same time. Demand diversity factors are generally employed to account for this, and to ensure the system is not unnecessarily oversized, and thus uneconomic.

For this system, that philosophy did not apply, i.e. the demand diversity adopted for design was 100%. This means the system will deliver the guaranteed flow & pressure to every single connection on it, at the same time.

More than that, the system will deliver guaranteed flow and pressure to a number of additional ‘dry’ connections. These are essentially farms that may connect to the system at some point in the future.

4 DESIGN OPTIMISATION PHASE

4.1 SPECIMEN DESIGN

A Specimen Design was developed by Opus prior to the tender phase. This was of sufficient detail to take straight to tender stage with little more detail design input. However, CPWL wished to explore opportunities to optimise the design, and so tenderers were invited to do so as part of their tender.

4.2 TENDER DESIGNS

The Downer team offered two designs as part of its tender.

The first design was fundamentally the same as the Specimen Design. In terms of major savings on pipe sizes, the analysis of the Specimen Design bore little fruit.

The alternative design eliminated the need for Pressure Reducing Valves and/or high pressure classes, minimised pumping requirements, and offered operational cost savings.

Three of four main trunk lines served only the upper part of the scheme by a primarily gravity driven network. These lines did not extend as low down the plains as the Specimen Design mains, eliminating the need for higher pressure class pipes or Pressure Reducing Valves.

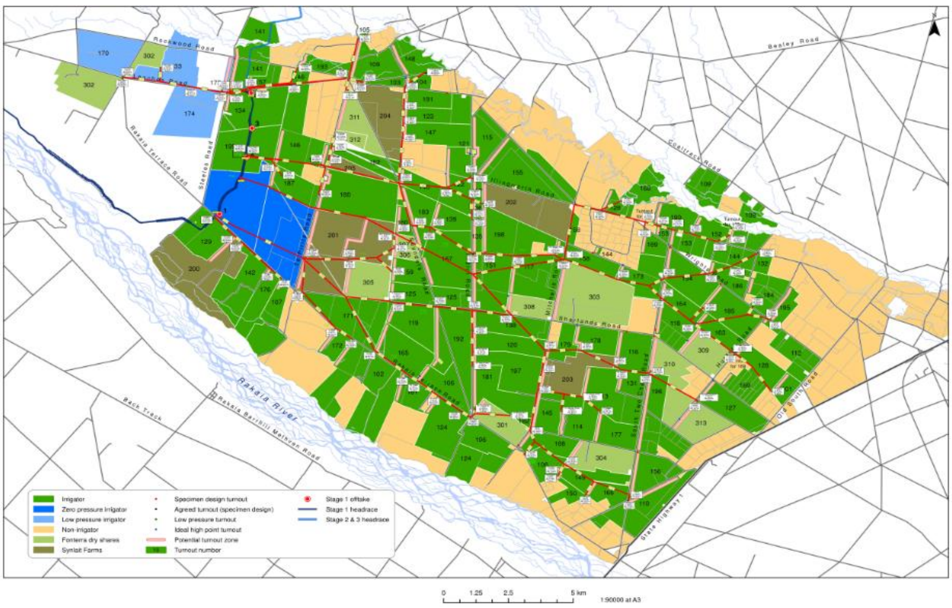

Figure 2: *Alternative Design*

The fourth offtake served all properties in the lowermost half of the scheme (furthest from the headrace) by a single gravity main via a pressure break pond.

4.3 FINAL DESIGN

The combined efforts and innovations of the Downer team were sufficient to secure the project. However, CPWL decided to revert to the Specimen Design. The reasons being:

- The stakeholder negotiations for easements for the pipe routes were already at a relatively advanced stage and the client did not wish to incur the risk of delay in starting afresh.
- The Specimen Design was a better fit to the client's whole of life funding model, being lower CAPEX, (but inevitably higher OPEX).

The final design comprises four independent main trunk lines distributing water from offtakes on the headrace canal. Each trunk line is generally arranged parallel to the fall of the plains, with various branch lines taking water off to where it is needed.

Lines 1-3 are mainly pressurized by gravity, with the upper parts of each being assisted by pressure boosting

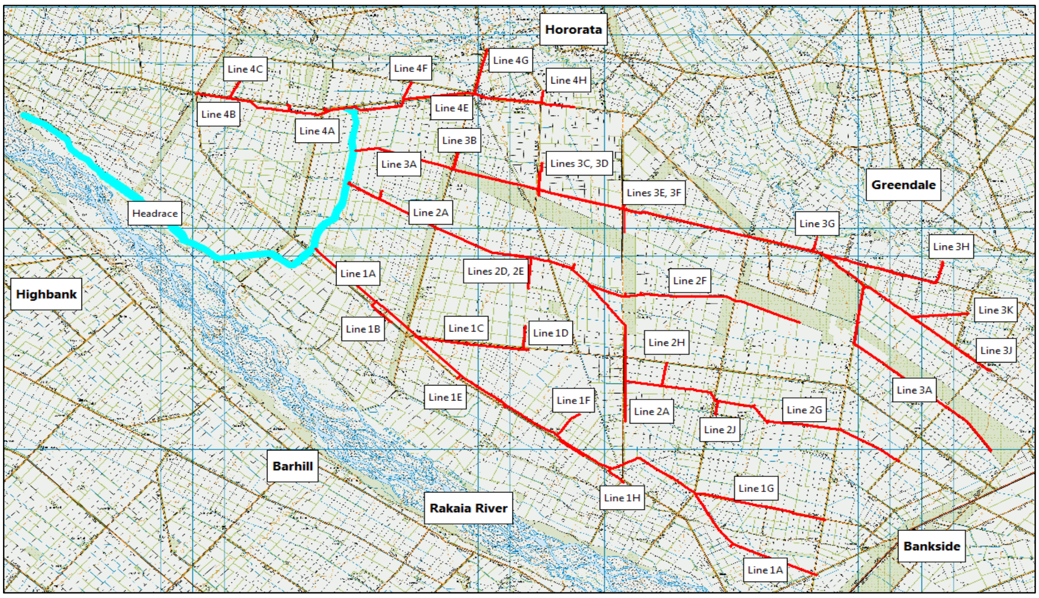

Figure 3: Final Design

pumps where gravity cannot provide the required pressure. Line 4 is entirely pumped.

There are approximately 135km of pipes assisted by 9 pump stations with a total power output of 1800 kW.

5 MATERIALS & CONSTRUCTION METHODOLOGY

Both designs were based on using PE pipe up to DN1600 extruded in long lengths up to 100m. Long lengths were made possible by the use of a mobile extrusion plant, which eliminates the need to transport pipes long distances by road. The long lengths were simply pulled a short distance across country to where they were needed.

Figure 4: mobile extrusion plant

A major advantage of making pipes on site is the resulting speed of installation, since the number of welding operations (which can take many hours at larger diameters) is reduced by a factor of around 7.

Another speed advantage was offered by the innovative installation method, whereby compaction of the embedment is achieved by vibrating the pipe itself using a ‘whacker’ plate mounted on an excavator. Backfill material is then placed in the trench and the vibrating pipe ensures the material flows all the way around the pipe,

Figure 5: Excavator mounted vibration compactor & backfill tool

including the essential and hard to reach ‘haunch zone’ in the bottom 1/3 of the pipe. The method also ensures any large particles do not touch the pipe - they essentially bounce off. This method has been demonstrated to achieve superior results to the NZ Standard method.

A third speed advantage was offered by the use of large scale chain trenchers over traditional hydraulic excavators.

Figure 6: Chain trencher

An additional advantage of on-site extrusion is ‘custom’ pressure classes, whereby pipe can be made to intermediate pressure ratings to reduce material costs. This is discussed in further detail in Section 6.

6 SYSTEM ELEMENTS & DESIGN CONSIDERATIONS

6.1 GRAVITY SYSTEM

The design of the gravity portion of the system is relatively simple. Each line builds pressure as quickly as is reasonably possible until the required threshold pressure is achieved: around 37 – 42m. This pressure is then maintained to the end of each line.

6.1.1 PIPE SIZE

The upper parts of the gravity system are sized to build pressure quickly without dropping below a reasonable minimum velocity – generally around 1m/s. The minimum velocity gives some assurance that the system will self-cleanse, and we found that reducing the minimum velocity much further becomes uneconomic, i.e. that it is more cost effective to simply regain any lost pressure by pumping.

Further downstream, each pipe is sized to maintain the dynamic pressure relative to ground level.

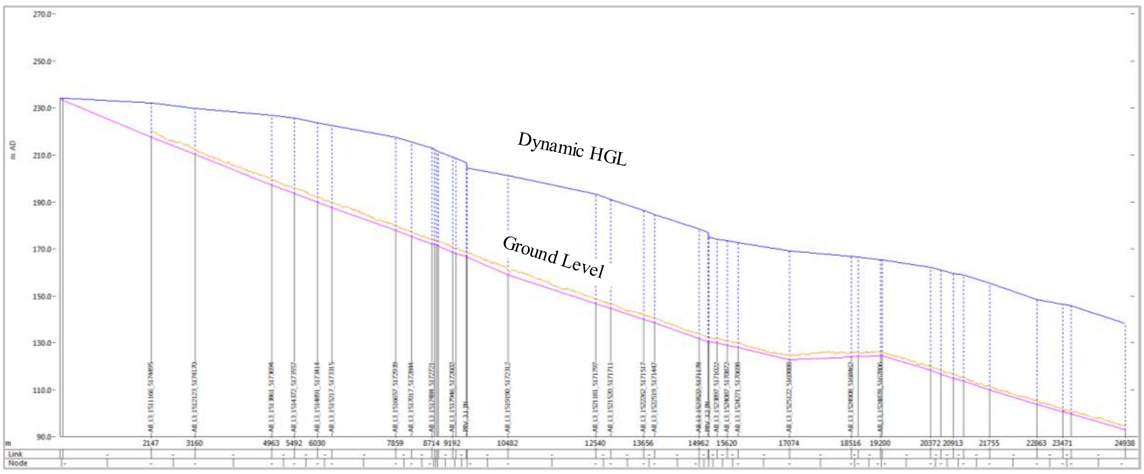

Figure 7: typical dynamic hydraulic grade line

As the flow reduces, so do the pipe sizes. In most cases this increases the design velocity, around 3m/s for the larger diameters, reducing to around 1-2m/s for the smaller diameters.

6.1.2 PRESSURE RATING

Each pipe must be able to safely withstand not only the dynamic pressure, but the maximum possible pressure it will experience.

In the winter, when rain is frequent, the system will not be used so much, if at all. However, it will not be emptied of water at the end of each irrigation season. So the pressure in the system will reach static equilibrium, or static pressure, where the water level in the canal dictates the pressure everywhere in the system. The lowest point on the system is 140m below the canal, so the static pressure here will exceed 140m.

It follows, then, that as each pipe progresses down the plain, its pressure class must be increased before it is exceeded.

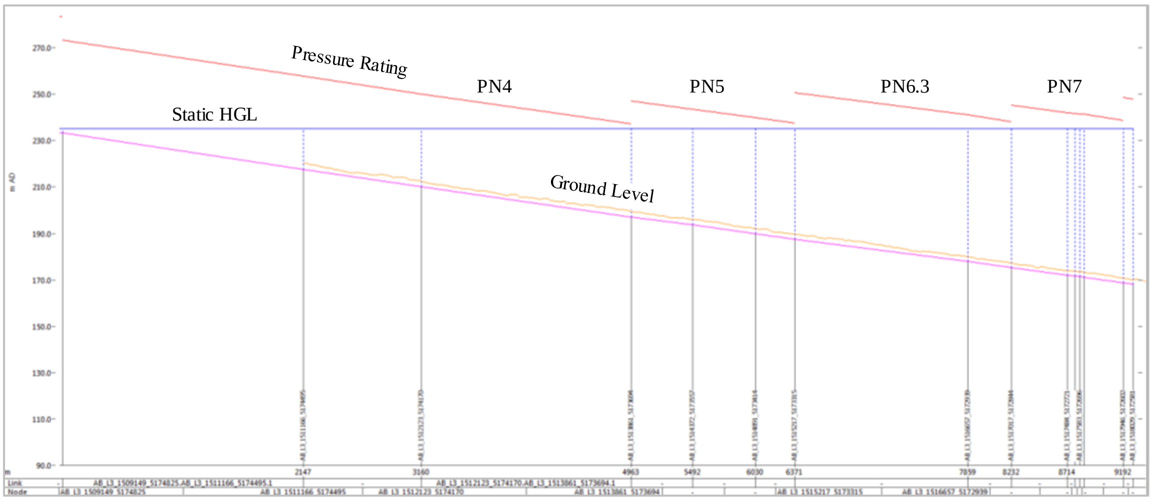

Figure 8: Increasing pressure class to resist static pressure

The pressure class of PE pipe is increased by increasing its wall thickness. This means higher class pipe requires more plastic, and is inevitably more expensive. PN16 pipe is approximately twice the price of PN6.3.

PE pipe is extruded at a constant outside (nominal) diameter, which means that the inside diameter is smaller in higher class pipe. So a second consequence of higher class pipe is reduced hydraulic efficiency. Again comparing PN16 pipe to PN6.3, to get the same hydraulic efficiency as PN6.3 in a PN16 pipe, the next size up must be selected, which increases the cost again, by about 20%.

6.1.3 PRESSURE REDUCING VALVES

Fortunately, there is a way to reduce the need for higher pressure class pipe, realising the twin benefits of lower cost, and increased efficiency.

Pressure Reducing Valves (PRVs) will maintain a set pressure on their downstream side, whatever the upstream condition may be. They do this by opening and closing a circular gate against an orifice. The gate's position is controlled by the pressure differential across the valve. When the flow through the valve is high, the upstream pressure is low, the gate will open and the hydraulic loss across it will be low. When the flow is low, the upstream pressure is high, the gate will close and the loss will be high.

PRVs will maintain a set pressure on the downstream side which is lower than the maximum pressure on the upstream side, thereby reducing the requirement for high pressure classes downstream.

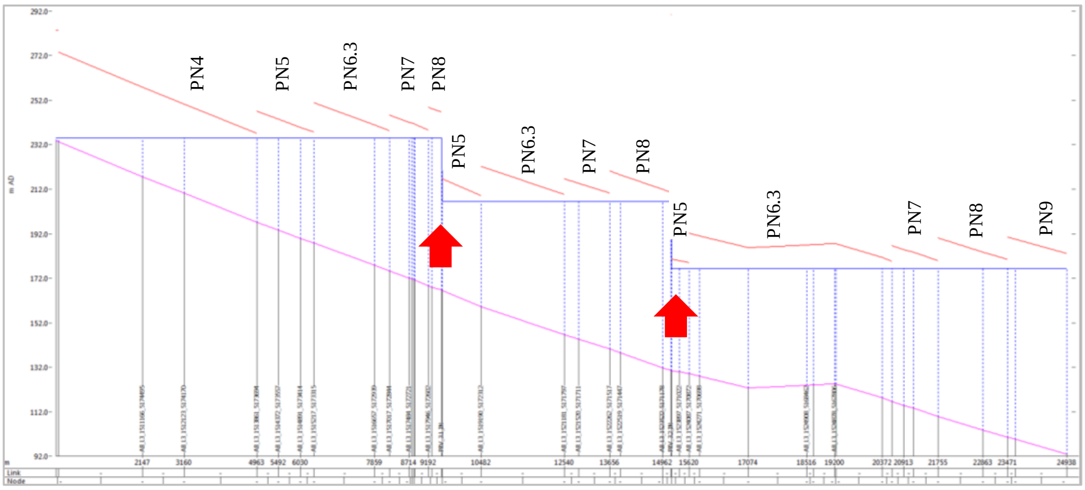

Figure 9: Effect of PRVs on required pressure class

The principal disadvantage of PRVs is that they create a residual hydraulic loss even when fully open, so the pipes immediately downstream may need to be upsized to re-gain the lost pressure.

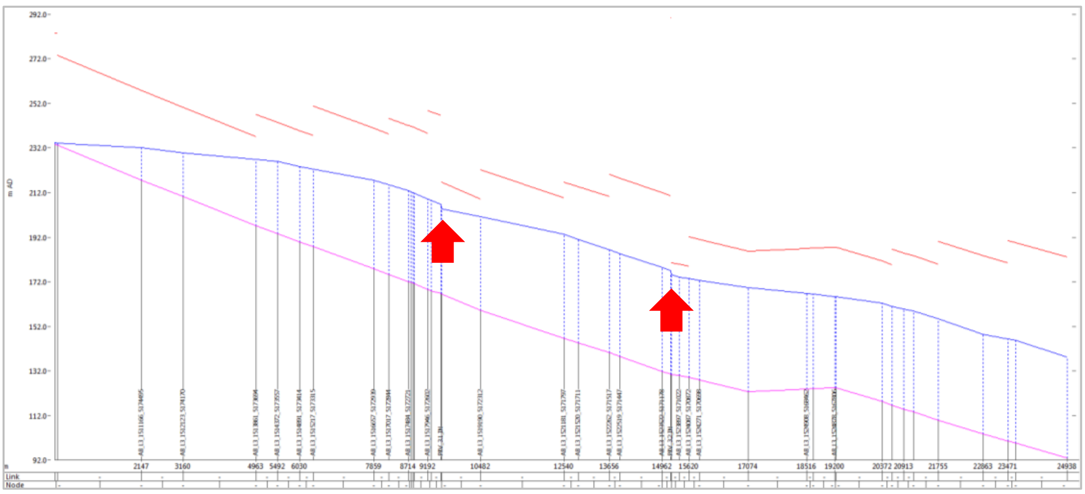

Figure 10: residual hydraulic loss at PRVs

Other considerations of PRVs include:

- They have an associated cost, particularly larger PRVs which generally need to be installed in a chamber.
- They must be maintained.
- They may fail, over-pressurising the pipes downstream. Pressure relief valves are provided as mitigation against this risk.

6.2 PUMPED SYSTEMS

Pump stations have been provided where the system, under design conditions, is unable to deliver the required head at turnouts by gravity supply alone. Pump stations are designed to provide pressure boosting to the turnouts in an effective and reliable manner.

The size and position of each pump station was the result of an optimisation exercise, where the most cost effective solution lies somewhere between the extremes of 'one big pump station' and 'a small pump station at every connection'. Factors affecting the optimum solution, and the interrelationships between them, are complex, involving several disciplines usually at odds with each other:

6.2.1 COMBINED PUMPING POWER

Larger, more powerful pump stations generally cost less per kilowatt produced. Generally it is preferable to combine turnout connections such that a total output power of over 100kW is required.

6.2.2 PIPE PRESSURE CLASS

Most pump stations below the headrace serve turnouts below them, so they are actually pumping downhill. This has the same effect on the selection of pressure classes as the gravity systems – lower down, the pressure class has to go up. There is a point at which it is uneconomic to continue serving turnouts lower down. Instead, a second pump station is constructed, with a new set pressure.

The Specimen Design process settled on 'pumping downhill' pump stations around 20 vertical metres apart.

6.2.3 'WASTED' PRESSURE

A pump station sized to deliver pressure to a remote turnout will deliver too much pressure to a turnout nearby, especially when the remote turnout is higher and the line runs uphill (i.e. a traditional rising main). This problem is compounded when the pipe is too small, and also when intermediate offtakes reduce the flow towards the end of the line. A more cost effective solution may be to provide two or more pump stations along the line providing just enough pressure, or a little more, to every turnout.

6.2.4 DESIGN HGL

Larger pipes waste less energy through friction, but cost more. For pumping downhill, we found that emulating the gravity model of maintaining an HGL roughly parallel to the ground slope gave the best results. For pumping uphill, we found that a design HGL of approximately 7m/km gave the best results. It is somewhat serendipitous that this is very close to the fall of the Canterbury Plains themselves (about 6.5m/km).

6.2.5 NET POSITIVE SUCTION HEAD (NPSH)

The first pump station on any line is usually placed sufficiently below the canal water level to ensure positive suction head at all times.

Ensuring NPSH can be achieved by setting the pumps below ground, or further down the line, once pressure has built above ground. The latter solution was found to be more cost effective.

There are two exceptions to this rule, both of which employ vacuum priming systems.

Lastly, factors such as **access**, **land use** & **ownership**, and the cost of getting **power to the site** can have significant influence on the final location of each pump station.

6.3 TRANSIENT EVENTS

Transient – or surge – events can occur as a result of sudden changes to the flow regime, such as might be caused by a pump trip, or a large scale power failure causing multiple irrigators to shut down at the same time. This can result in both positive and negative (vacuum) transient pressures.

6.3.1 POSITIVE PRESSURE

PE pipe, as a visco-elastic material, can withstand high pressures for short periods without significant detriment to its design lifespan. International standards vary in their acknowledgement of this phenomenon. Some (such as the UK) accept up to 200% of the pipe rating for short term pressure events. NZ Standards make no such allowance.

The parties realised that this phenomenon should be allowed for, as it results in significant capital cost savings, and so a threefold rule was adopted:

1. Long term static pressure shall not exceed 95% of the pressure rating. 95% percent is in line with industry practice and primarily allows for construction tolerance issues (e.g. pipes buried deeper than design levels, PN changes in incorrect locations, etc.)
2. Short term transient pressures of 1 minute or less shall not exceed 150% of the pressure rating
3. All other transient events shall not exceed 125% of the pressure rating.

6.3.2 NEGATIVE PRESSURE

The minimum transient pressure in the pipeline was set at -30kPa. This value was chosen as one which would not impose unreasonable restrictions on the structural design of the pipes while still providing sufficient sub atmospheric pressure difference across the air valves, which will open to admit air to the pipe in the event of negative pressure. The air valves are sized to allow air into the pipe at a sufficient rate to avoid vacuum collapse. If the negative pressure threshold is set too low the air valves become excessively large.

Lines 1, 2 and 3 are not expected to generate negative pressures during transient events although negative pressures will occur when the pipe is being drained or after a pipe burst. This is because the flow is generally downhill, including much of the flow from booster pumps.

Pipeline 4 will experience negative pressures in the event of pump trips. This is because this pipeline has uphill flow so that when the pumps stop the flow will reverse until check valves close. This typically generates significant pressure fluctuations, both positive and negative.

6.3.3 SURGE MITIGATION MEASURES

The significant events which generate the surge pressures are the reduction of flow at the turnouts and at the pumps in the event of an area wide power failure.

Mitigation measures include

- increasing the turnout closing time
- Increasing the PRV operation time
- pressure relief valves
- surge pressure tanks
- surge anticipating valves
- anti-shock air valves

Transient, or surge events occur as a result of a sudden restriction in flow at a given point. The most severe instance would be an area wide power failure, which would cause every turnout control valve to close. The closing time at each turnout has been adjusted to be as long as practicable. This is a primary mitigation measure, but the limits of practicality restrict what can be done.

Pressure Reducing Valves have also been limited in their speed of operation for similar reasons.

Pressure relief valves are provided on turnouts in the lower parts of the scheme where the pressure changes occur relatively slowly so that the relief valves have time to operate. Their disadvantage is that they discharge considerable quantities of water. This is dispersed to suitable receiving environments via identified safe secondary flow paths.

Pressure relief valves are also placed immediately downstream of each pressure reducing valve. This acts to protect the lower part of the pipeline in case the pressure reducing valve fails to close completely. It also opens during transient events and helps to limit the maximum transient pressures.

Surge anticipating valves are similar to pressure relief valves but they open before the positive pressure occurs,. They sense the drop in pressure when the pumps stop suddenly and begin to open, so they are already open by the time the returning positive pressure arrives. They are tricky to set up properly and are best suited to high head pumping systems.

Surge pressure tanks have been provided at pump stations where required. We have selected an air bladder type tank as this requires minimal maintenance and negligible ancillary equipment. The alternative air over water tank requires a compressor and associated equipment to maintain the air volume. Surge tanks will empty and fill in response to negative and positive transients, with the enclosed cushion of air behind the bladder acting as a shock absorber.

7 NEXT STEPS

Stage 1 is now largely built and undergoing commissioning. It is expected to be operational in time for the 2015/2016 irrigation season.

CPWL is now in the early feasibility and optioneering phase for Stage 2, an area approximately twice the size of Stage 1. MWH is assisting in evaluating several options, seeking to find the best balance of pipes, canals, and storage.

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