PUMP STATION AND PRESSURE MAIN NO.128

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ABSTRACT

The paper discusses the design and construction aspects of the wastewater Pump Station 128 and the associated Pressure Main 128 being constructed by the Stronger Christchurch Infrastructure Rebuild Team (SCIRT) in the New Brighton area of Christchurch. The existing wastewater pump station was damaged beyond repair by the February 2011 earthquake and the decision was taken to build a new pump station and pressure main at an alternative location to avoid the Red-Zoned area around the existing pump station. The new pump station has a capacity of 625L/s and the new DN 800 PE 100 pressure main is 3km long, routed under the Avon River and is being installed using directional drilling. This makes Pressure Main 128 the biggest directionally-drilled pressure main in Christchurch and the project presents unique challenges. The existing pump station has Archimedes screw pumps whereas the new pump station uses submersible pumps and operates in duty/assist/standby arrangement. The pump station site and pressure main route suffers from poor ground conditions so several innovative design techniques have been used to improve ground conditions and avoid effects of differential settlement.

The paper discusses both the design and construction stages of the projects including pros and cons of directional drilling technology for the pressure main, pipe welding, pipe testing, inlet design of the pump station, wet well hydraulics, proper pump selection to minimise wear and tear, odour treatment and ground improvement strategy to mitigate seismic-related damage and construction issues.

This paper provides valuable lessons to the industry on how to improve the design and construction of these important large wastewater structures.

KEYWORDS

Pump station, pressure main, horizontal directional drilling, earthquake resilience, wastewater, Christchurch

1 INTRODUCTION

Pump Station (PS) 63 at Hulverstone Drive, Bexley, is a lift station which uses three Archimedes screw pumps to lift the wastewater and discharge into DN 900mm RCRRJ gravity sewer. This gravity sewer conveys the wastewater more than 2km to the terminal PS36 at Pages Road, Bromley, for transfer to the Christchurch Wastewater Treatment Plant.

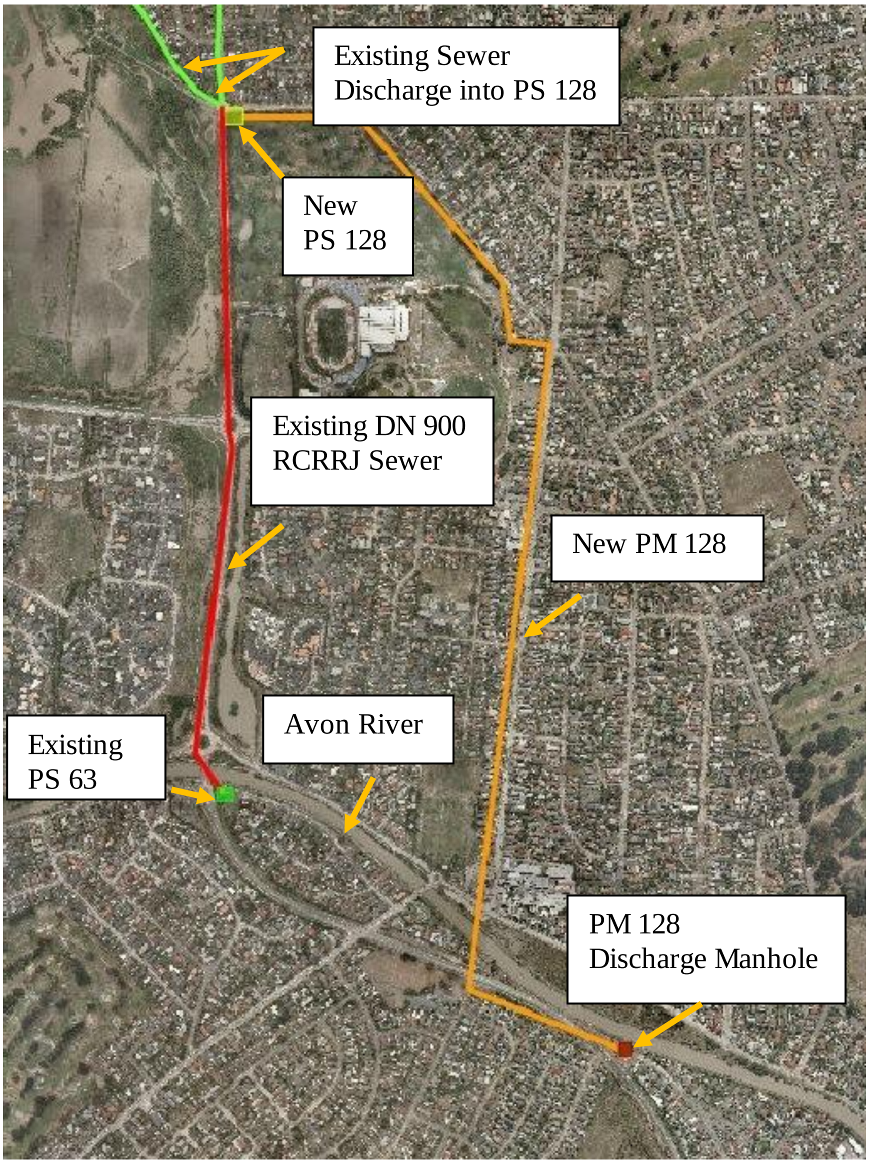
During the 2010 and 2011 seismic events, the PS63 site suffered damage from considerable buoyancy uplift, rotation and lateral spreading, as shown in Photograph 1. Although the pump station continues to operate in a damaged state, there are concerns about its fragility and long-term design life.

Photograph 1: Rotation of Pump Station 63 resulting from buoyancy uplift, lateral spreading and slope instability

2 PUMP STATION 128

The Northern and Coastal Investigation Area Wastewater Strategy completed during the Infrastructure Rebuild Management Office (IRMO) period of the rebuild examined options for repairing and rebuilding the wastewater infrastructure in the north-eastern part of the city, most significantly PS36 and PS63 and the associated trunk infrastructure. The strategy concluded that the most appropriate option was the construction of a new pump station at Beach Road, with a rising main pumping into the existing DN 1075mm RCRRJ trunk main between existing PS63 and PS136. The new PS128 would replace the existing PS63, and allow decommissioning of existing large diameter deep gravity sewers that would require replacement were the pump station to remain in its current location. The new wastewater PS128 will be located at Ascot Links Golf Course on Beach Road with a design capacity of 625L/s. The pump station will discharge wastewater in a new 800 mm OD PE 100 pressure main routed through Queen Elizabeth II Park, along Bower Avenue, under the Avon River, and discharging into the existing DN1075 gravity trunk sewer at Anzac Drive. Aerial plan details of the existing and new systems are shown in Figure 1.

Figure 1: Existing PS63 and new PS128 system overview



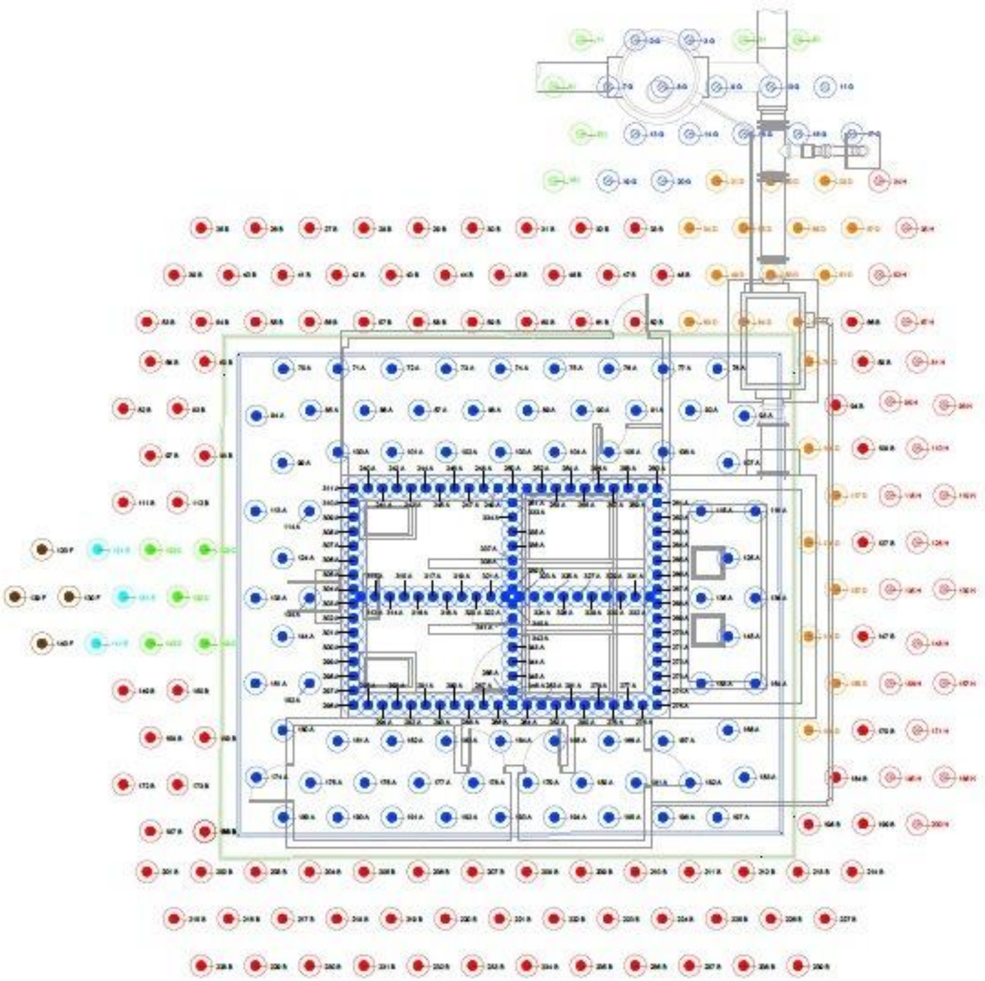
2.1 GEOTECHNICAL DESIGN

The new PS128 site is generally flat and the geological map indicates that the site is underlain by fixed and semi-fixed dunes of sand of Christchurch formation (Brown & Weeber, 1992). An 1856 map ("Black Maps") of the area shows that the sandy ground lies to the east and swamp to the west of the pump station site. The site has high liquefaction potential and recent earthquakes in Christchurch have produced accelerations that are similar to the Ultimate Limit State (ULS) ground acceleration of 0.61g.

Ground improvements were carried out at the site to reduce the risk of damage in case of future earthquake events. These improvements consisted of the installation of 600mm diameter 20MPa concrete, high modulus Continuous Flight Auger (CFA) columns to mitigate liquefaction potential for the ULS earthquake. Low capital cost, lowest construction risk and quick installation were the main reason for selecting CFA columns for the ground improvement. A lattice arrangement of contiguous columns under the wet well structure and individual columns in the surrounding ground was installed (see Figure 2). The length of the columns would taper along the sides of the pump station to provide a gradual transition.

The wet well ground improvement will extend beneath the control room foundations and under the pump station. This ground improvement will extend a minimum of 3m beyond the footprint of the electrical and control room. The ground improvement will limit the magnitude of seismic settlement beneath the control room building; settlement is anticipated to be similar to that of the wet well. Differential settlement between the control room building and the PS128 wet well/valve chamber structure has been minimised by installation of ground improvement down to the same elevation. Following a ULS event, differential settlement between the wet well/valve chamber and control room building is likely to be in the order of <10mm to 25mm, with the control room building displacing down relative to the main structure.

Figure 2: Location and distribution of CFA columns



2.2 WET WELL

The wet well structure is 9.1m long, 6.7m wide and 7.75m deep (see Photographs 2 and 3 of the wet well under construction). The wet well has been divided into three chambers by partition walls to allow for isolation of individual pump bays for maintenance. The salient design features include maximum of 10 starts per hour, sufficient operational volume for pump ramp-up and ramp-down and minimum footprint and depth of excavation. During normal operation, the operational volume will be held in all three bays. The operational volume has been based on minimum required for correct pump operation. When one pump bay is isolated for maintenance purposes, the operational volume will be held in two bays. During peak wet weather the pump starts can be alternated between two pumps to avoid increased pump starts per hour due to the reduced available volume. The operation of the pumps via VSDs (variable speed drives) will allow the number of start/stop cycles to be reduced.

Photograph 2: Pump station wet well under construction

Photograph 3: Pump station wet well under construction

2.3 PUMPS

A number of pumps and pumping arrangements were investigated and the final chosen pumps were three Xylem pumps NP3400/735 (curve 670, 470mm impeller) submersible pumps with 140kW motor. Duty/assist/standby configuration has been adopted, with all pumps being of the same capacity. This pumping arrangement allows operation of the pumps within the preferred operating region (POR) which results in efficient operation and reduces pump wear and tear. For normal pumping operation during average dry weather condition, only one pump will be required to operate and the pumping operation will be alternated between the pumps to reduce the wear and tear. During peak wet weather, two pumps will operate together. The design duty point of the pump station is as follows:

- 708L/s @ 21.8m (duty/assist pumps operating together)
- 570L/s @ 16.3m (single pump operation at 50Hz)
- Minimum flow = 300L/s @ 7.0m (single pump operation at 30Hz)

2.4 OPERATING REGIME

Smart control functionality will be incorporated to allow pumping operation within POR by switching from one to two pump operation at less than 50Hz speed as required, based on real-time continuous “PORmax” algorithm added into the controls. Conversely, pumping operation will switch from two to one pump at greater than minimum speed required using “PORmin” algorithm.

2.5 FLUSHING CYCLE

During pumping operation other than at PWWF, the velocity in pipeline and related shear stress is below the normal design criteria of 2m/s and 4N/m^2. To avoid deposition of slime in the pressure main, a daily flushing cycle will be run with both pumps operating at maximum speed. To achieve this, the wet well will be filled with wastewater to the highest operating level and then both pumps will be turned on.

2.6 GENERATOR

A diesel-powered emergency generator and fuel storage is installed on site to provide full back-up power in the event of loss of mains power to the area. The generator is sized to allow Christchurch City Council (CCC) to export power to the network if desired.

The backup generator will be installed on a concrete pad beyond the ground improvement zone. To allow an uninterrupted operation during vertical and lateral movement induced by an earthquake event, around 200mm of slack in the cable and pipe connections would be provided between the generator and the pump station.

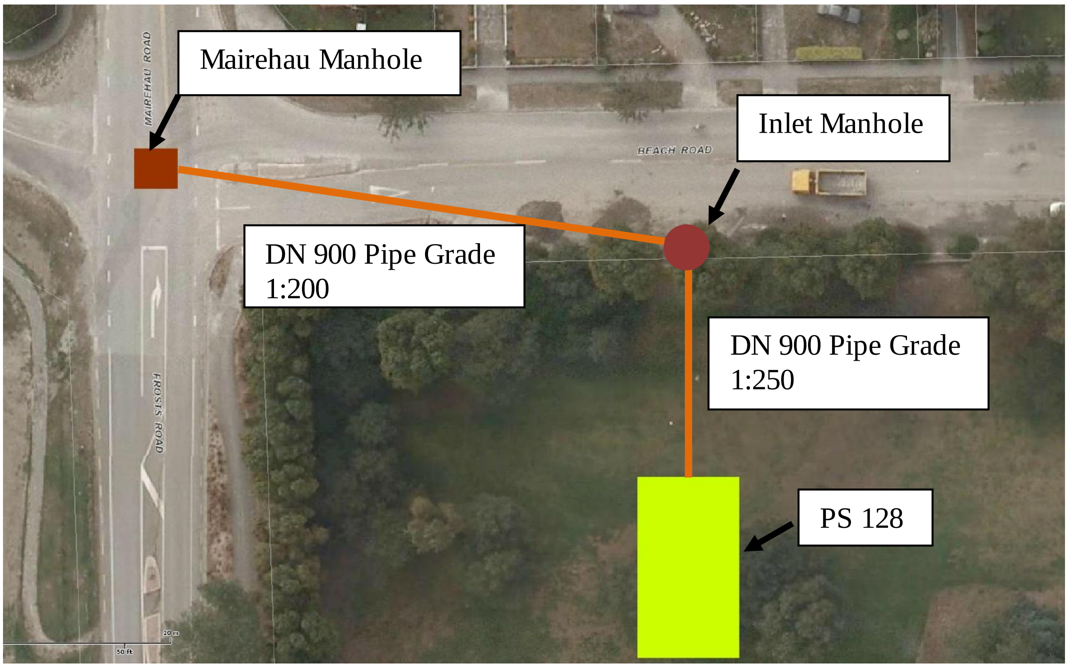
2.7 RESILIENCE FEATURES

The following resilience features, in addition to ground improvement, have been incorporated in the pump station design.

2.7.1 STEEP INLET PIPEWORK

To provide resilience against differential settlement between the pump station and the upstream surrounding network, the inlet pipe from the Mairehau Road manhole to the inlet manhole is laid at a grade of 1:200 and the pipe from inlet manhole to the wet well is laid at a grade of 1:250. This steep grade provides 300mm vertical drop from the new manhole at Mairehau Road to the pump station (see Figure 3). This has been calculated as sufficient to maintain positive grade even after two significant seismic events. This grade has been selected as a suitable balance between providing resilience in the design, and maintaining optimal hydraulics of the inlet pipe. A risk remains that settlement beyond the estimated maximum occurs in future seismic activity, thereby resulting in flat or negative grade inlet pipes. Should this happen, flows would continue to enter the station due to its location at the base of the catchment, but the inlet pipe would require relaying at some point following such an occurrence.

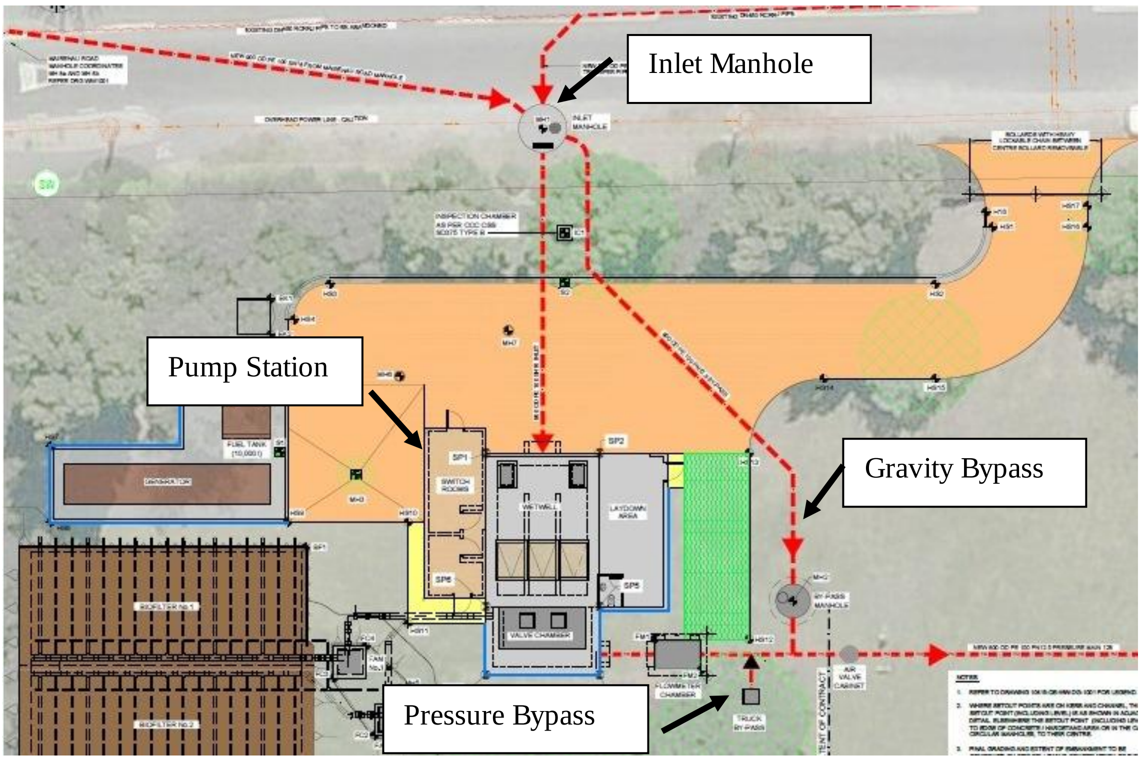
Figure 3: DN 900 inlet pipe at steeper grade



2.7.2 BYPASS ARRANGEMENT

Both gravity and pressure bypass arrangements have been provided at the pump station to allow up to 250L/s of flow to be diverted to the pressure main in case of mechanical or electrical failure. The gravity bypass consists of DN 800mm PE 100 SN 16 profile pipe. The bypass will be operator-controlled and the bypass valve at PS128 can be controlled through SCADA. To effect the bypass arrangement, the overtopping weir penstock within the discharge manhole of PM128 (by Anzac Drive) will need to be raised manually. The pressure bypass consists of DN 200 ductile iron pipe. The flow would be pumped out from the inlet manhole and diverted to the pressure main via this bypass by using portable pumping equipment. See Figure 4 for bypass details.

Figure 4: Gravity and Pressure Bypass Arrangement



2.7.3 FLEXIBLE EXPANSION FITTINGS

To reduce the risk of pipe shear between pump station valve chamber and the flow meter chamber during differential settlement, the entire manifold pipework has been installed in the valve chamber, hence allowing a single pipe to exit which essentially means that there is a risk of only one point of shear failure (see Figure 5). This risk has been further minimised by the use of a force-balanced Flex-Tend flexible ductile iron fitting between the valve chamber and the flow meter chamber (see Figure 6). Whilst both structures are founded within the improved ground zone their close proximity heightens the possibility of damaging differential settlement of up to 300mm occurring in a significant event. This fitting will be embedded in a 500mm of uncompacted pea metal to allow movement in an earthquake event. This fitting is designed to allow a differential settlement of approximately 500mm whilst still maintaining the integrity of the connection. The fitting can also withstand the thrust force arising from differential settlement. The fitting will be bedded in 500mm of uncompacted pea metal to allow movement in a seismic event.

Use of PE (polyethylene) pipe as a flexible material between the valve chamber and flow meter chamber was considered but the bending strain induced as a result of a 300mm differential settlement meant that the pipe would undergo creep and lose its design life. In such an instance, CCC would have to replace that section of the pipe which would cause delay in pumping operation and for this main reason this option was not adopted.

Figure 5: Flexible Fitting between Valve Chamber and Flow meter Chamber

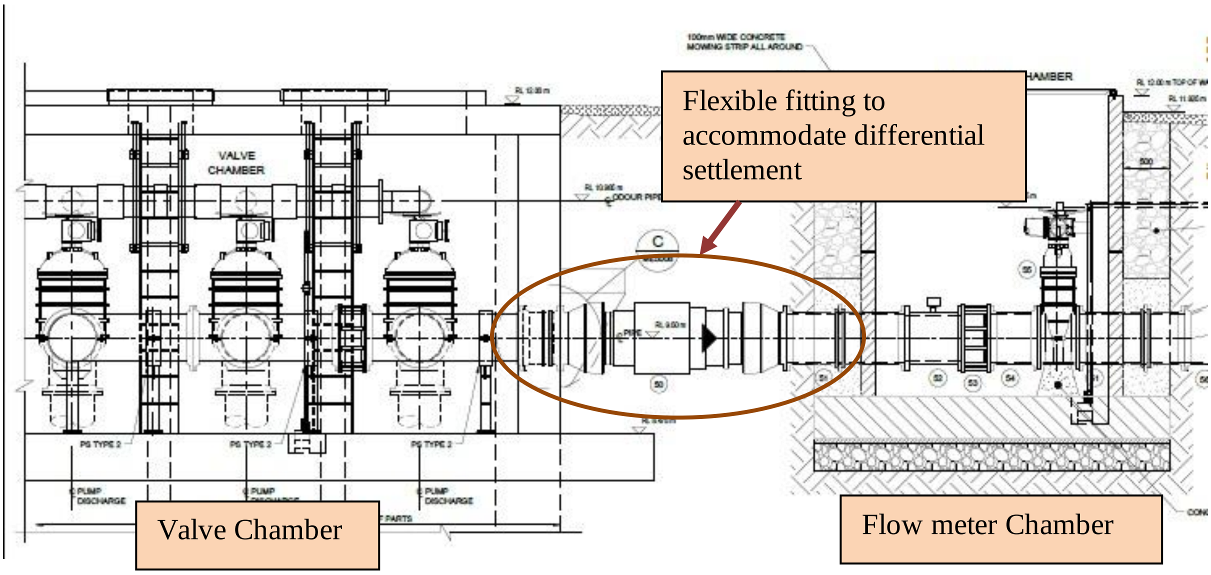
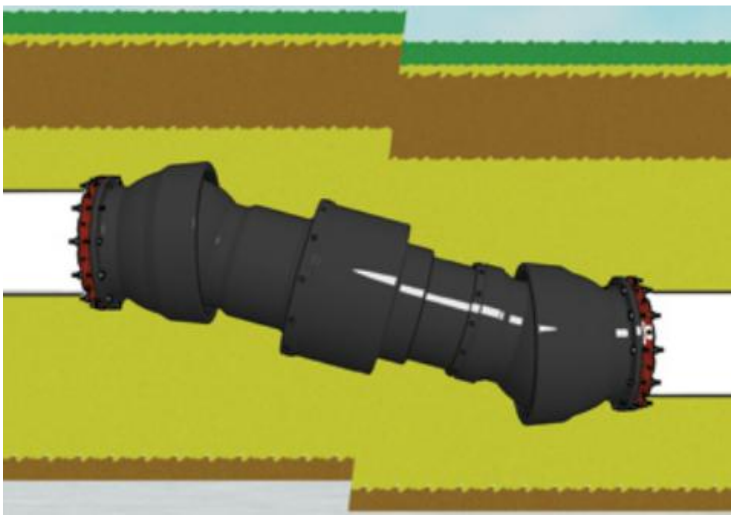


Figure 6: Force-balanced "Flex-Tend" (image courtesy of EBAA Iron Inc.)



2.8 OPERATION AND MAINTENANCE OF THE PUMP STATION

Several design features have been incorporated for the operation and maintenance of the pump station and these are discussed below.

Isolation of the wet well for maintenance will be achieved by an inlet penstock mounted over the inlet pipe within the wet well. The penstock will be fitted with an electric actuator to allow operation from local or remote SCADA.

Isolation of individual bays in the wet well will be achieved by installing stop boards in slots located in front of each pump. Two stop boards of 7.5m high and 5.0m high will be held in the pump station for this purpose. The shorter units allow isolation of the bay for draining and cleaning down from the wet well access platform. The taller units allow for complete isolation of a bay with added protection against overtopping under extreme liquid levels.

Lockable hinged covers are provided over each pump to provide access for removal. Each cover is split into two, with each leaf weighing less than 20kg for a single-man lift. An overhead travelling crane is provided as part of the pump station building to allow removal of the pumps for maintenance. It is anticipated that most maintenance activities will be performed inside the pump station building, although vehicle access has been provided to allow pumps to be removed if required.

2.9 BIOFILTER

A Biofilter is provided to treat odorous air extracted from the wet well and gravity pipe network. One biofilter bed will be provided, designed as two integrated separate biofilters, each with a duty fan. This system will provide a measure of redundancy in the odour system and allow one side of the bed to be taken offline for maintenance while still providing a reduced level of treatment through the other side.

Extraction from the wet well is via three manifold connections through the wet well / valve chamber wall. Air will be drawn from the inlet pipes through the wet well, thereby providing ventilation to the entire system. Since the inlet pipe is only surcharged under peak flow conditions (when the upstream catchment is surcharged), this arrangement will provide suitable ventilation at all times.

The fans will be operated on VSDs to allow compensation for the increase in back-pressure as the biofilter medium ages. Pressure sensors in the discharge ducts will be reported in the SCADA system to allow monitoring.

3 PRESSURE MAIN 128

Pressure main (PM) 128 is a new 800 OD PE 100, PN 12.5/PN16 wastewater pressure main running from the new PS128 to discharge manhole located on ANZAC Drive in Aranui. As indicated in Section 2, both the pump station and pressure main are designed to replace the existing gravity pipeline and pump station along ANZAC Drive and Hulverstone Drive.

The installation of new pressure main will improve security of the wastewater services to most of North East Christchurch and will serve as a major link to New Brighton, Parklands, Queens Park as well as catchments further afield such as Belfast and Brooklands.

The critical project aspects included selection of pipeline route, structural design of the pipe, surge analysis, installation method, pipe joint testing and design of discharge manhole.

3.1 INSTALLATION METHOD – HORIZONTAL DIRECTIONAL DRILLING

Both open trenching and trenchless options were considered at the design stage of the project. Meetings were held with the SCIRT Delivery Team and it was decided that horizontal directional drilling would be the most appropriate construction methodology for installing the pressure main (see Photographs 4 and 5 for examples of directional drilling). Directional drilling is done in stages. A pilot bore is carried out first between an entry and an exit pit and then the diameter of bore hole is increased in stages using a reamer until it is big enough for the pipe to be pulled through (Ryan and Finney, 2012). Drilling fluid is injected into the bore hole during pilot-boring and during back-reaming to stabilise the hole and to provide lubrication during pipe-pull. The main reasons for choosing directional drilling for this job were:

- Reduced installation cost when compared with trenched installation or other trenchless technology
- Quicker installation of the pipeline
- Required for pipe installation under the Avon River
- Reduced impact on sensitive areas – such as Queen Elizabeth II (QEII) Park and Travis Wetland (potential effects of dewatering large open trenches)
- Low likelihood of existing service interference (due to increased installation depth and minimal open trenching)
- Reduced traffic management requirements
- Lower impact on residents and businesses along the route.

3.1.1 TRACKING OF DIRECTIONALLY DRILLED PIPE

The tracking of directionally drilled pipe is important to make sure that it has been installed as per the design grade. If the installed pressure main has unnecessary elevated points, air can accumulate at these points; increasing frictional resistance, reducing the cross-sectional area of the pipe and adversely affecting the performance of the pumps. A walkover system has been adopted to check the vertical alignment of the pilot bore. Once the alignment is confirmed, the diameter of bore hole is increased in stages until it is big enough for the pipe to be pulled through. Inclinometer survey is being carried out to assess the final installed grade of the pipe.

Photograph 4:Directional Drilling

Photograph 5: Directional Drilling

3.2 SITE SURVEY

LiDAR was used to confirm the pressure main route and to minimise the cost of topographical survey. Due to the critical nature of the Avon River crossing, a topographic and river bed survey was obtained for the crossing route. The topography of the New Brighton area is generally very flat. The route selected through QEII Park is flat, with slight changes in elevation due to the landscaped nature of the golf course. The Bower Ave alignment has a slight slope towards the Avon River along its full length. Either side of the Avon River crossing the ground has a high elevation due to the stopbanks recently installed. Along Anzac Drive, the route selected has a slight rising topography to the location of the discharge structure.

3.3 ROUTE SELECTION

Two pressure main routes were considered – along Frosts Road and ANZAC Drive and other along Bower Avenue to ANZAC Drive. Both routes have liquefiable soils with up to 400mm predicted settlement in a ULS event. The route along Bower Avenue has a slightly lower geotechnical risk due to improved ground conditions which would provide increased resilience to the pressure main against future earthquake events. On the basis of lower geotechnical risk, Bower Avenue route was selected as the final route. The Bower Avenue route however, has a narrow utility corridor with a number of existing services. During Early Contractor Involvement (ECI), it was decided to directionally drill the pressure main to avoid interference with the existing services.

To reduce long-term subsidence of the ground above the pipe, and to reduce seismic softening of the material between the pipe and the bore hole, it was recommended that this annulus be filled with a 100 – 200kPa grout mixture on completion of installation of the pipe string. This was discussed with the Delivery Team during design stages of the project but, later on, the Delivery Team cited equipment issues and decided not to grout the annulus of the pipe.

The pipeline route also took into account the location of drilling pits and sufficient space availability for directional drilling equipment. The main installation equipment consists of drilling machine, slurry plant, slurry pumping equipment and butt welding plant. The pressure main is being installed in seven drill shots. The final pipeline route has slight variations from the initial pipeline route.

3.4 STRUCTURAL DESIGN

Structural design of the pressure main included design for deflection, buckling strength, hoop stress, internal and external pressure and pipe fatigue. The pipe class has been determined based on these structural checks. Structural calculations have been based on an empty pipe scenario, no side support for the pipe (as it would be directionally drilled), poor geotechnical conditions, high traffic load and high expected ground water table – representing a 'worst case' liquefied ground scenario.

In an open trench pipe installation, the resistance to ring deflection is provided by the side support and the pipe stiffness. In the case of horizontal directionally drilled pipe, there is essentially no side support even with the presence of drilling fluid around the installed pipe. The resistance to ring deflection is provided only by the pipe stiffness in this case (ASTM-F1962-11, 2012). To determine the pipe stiffness that would limit the ring deflection to an acceptable value, the equation provided in ASTM F1962-11 has been used which assumes no side support for the directionally drilled pipe.

PN 12.5 (PE100 SDR 13.6) pressure class has been proposed for the majority of the route as it satisfies the structural requirements outlined in both AS 2566.1:1998 and ASTM F1962-11 and the maximum allowable operating pressure limits described in AS 4130:2009. The section of pipe underneath the river requires higher ring stiffness due to the increased depth and corresponding soil loading. To maintain a suitable factor of safety when satisfying all the requirements, PE 100 PN 16 (SDR 11) pressure class has been adopted for that section. There is a high potential for liquefaction and lateral spreading at the Avon River banks. However the depth of pipe over this section and the use of naturally flexible PE pipe would mean that the pipeline is not expected to encounter structural damage or significant vertical alignment changes.

3.5 SURGE ANALYSIS

A detailed surge analysis has been performed and various scenarios have been considered for the surge analysis, encompassing normal and abnormal operating conditions. Elevated positive pressure surges are not encountered due to the relatively flat nature of the pressure main profile. However, negative pressure is encountered in the pipe for a few different situations, the critical scenarios being both power failure and controlled shut-down of pumps.

During power failure, the pressure within the entire pipe reaches negative 10m. This scenario is not likely to happen often and PE 100 PN 12.5 pipe is capable of withstanding this negative pressure during power failure. No other mitigation is considered necessary for this scenario. The surge analysis predicted a maximum positive pressure of 30m during power failure, which is not significant and hence no surge mitigation for positive rise in pressure is required.

The controlled pump(s) shut-down scenario was simulated using CCC's recommended VSD ramp-down rate of 100rpm/s from minimum to zero speed. This produced negative 10m pressure in certain sections of the pipe, which was not recommended because pumps are likely to start and stop a number of times per day. This can be avoided by decreasing the VSD ramp-down rate to 6rpm/sec from minimum to zero speed. At this suggested reduced ramp-down rate, the pipe experiences 4m of negative pressure and PE 100 PN 12.5 pipe is capable of withstanding this negative pressure on a repeated basis. The alternative standard mitigation option of providing vacuum valves was considered, but discounted as these would allow significant amounts of air into the pipe each time the pump(s) are shut down, which would be very difficult to expel during pump start-up and lead to adverse effects on the operation of the pumps and pressure main. These required VSD ramp rates will be specified in the Pumping Station Control Philosophy.

3.6 CONSIDERATIONS DURING DIRECTIONAL DRILLING

During the directional drilling of PM128, the following key elements have been identified that require primary consideration.

3.6.1 SAFE TENSILE STRESS ON THE PIPE

One of the characteristics of PE pipe is that it undergoes creep deformation under continuous load. ASTM F1962-11 provides safe tensile stresses for various durations of pull-back. Thus, one of the main considerations during directional drilling is to monitor the magnitude and time of the pull force on the pipe to avoid deformation of the pipe. The pull force is being monitored during directional drilling of the pressure main.

3.6.2 DRILLING FLUID PRESSURE AND VOLUME

As mentioned earlier, the drilling fluid is pumped into the bore hole during directional drilling to stabilise the hole and to provide lubrication during pipe-pull. Excessive drilling fluid pressure and volume can cause erosion of the bore hole and can result in blowouts, due to the penetration of drilling fluid through the fissures. Sinkholes can develop as a result of bore hole erosion and these sinkholes can cause the installed pipe to dip into the ground, creating undesirable low and high points. This is even more important for large diameter bore holes and poor ground conditions because of the reduced arching effect and running sand. Proper calculation and monitoring of slurry pressure and slurry volume is a primary consideration during directional drilling.

3.6.3 ACCURACY OF PIPE GRADE DURING DRILLING

The accuracy of the directional drilling process to install the pipe as per design grade is dependent on several factors. Pilot bore tracking using a walkover system or a wire line tracking system can aid in achieving an accurate bore alignment, but the results may not be readily applicable to the final grade of the installed pipe. The grade of the pipe during pull-back can be affected by certain factors including bore hole erosion, presence of obstacles and slurry consistency. In order to avoid misalignment of the pipe, such factors have to be considered at the construction stage and a separate grade check for the pipe can be carried out to assess the final alignment of the pipe. The final pipe grade for a couple of drill shots for PM128 has been found to be beyond the tolerance value specified in the design report, even though the corresponding grade of the pilot bore was accurate. The reason for this out-of-alignment of the pressure main is under investigation at the time of writing.

3.7 PE PIPE JOINTING

Pipe jointing has been given primary consideration in the project because of the importance of the asset and depth of the pressure main. To avoid interference with the existing services during directional drilling, 3.5 m cover has been provided for pressure main at certain places. Moreover this is the only pressure main conveying wastewater from PS128 to PS136 and any damage or leakage in the pressure main joint at such a depth would result in the loss of valuable time and will prove very costly. Both butt welding and electro-fusion welding have been used for this pressure main.

3.7.1 BUTT WELDING

In butt welding (or butt fusion), a heater plate is used to melt the two faces of pipe ends and, after a specific time period, the plate is removed and the two melted faces are pressed together to provide a fused joint (Scoby, 2012) (see Photograph 6). Special care has been taken to go through the butt welding parameters for this size of PE 100 pipe. ISO 21307:2011 parameters were adopted for butt welding of the pipe. DVS parameters were also considered and the main difference between the DVS and ISO parameters is fusion jointing pressure and the change over time. The DVS value is 0.02 N/mm^2 less than the ISO equivalent. Numerous meetings were held with the CCC Technical Lead and with the welder to sort out the welding parameters. In the end, ISO parameters were adopted because of higher fusion jointing pressure and small change over time. The fusion jointing pressure is a very important parameter in butt welding and refers to the pressure applied to the pipe ends against the heater plate. The change over time is the time taken to remove the heater plate and join the two pipe ends together to form the joint. Too long a changeover time may result in a cold zone at the interface and the joint will have reduced strength and may fail in a brittle manner.

Photograph 6: Butt welding equipment at the site

Another important parameter during butt welding is the drag pressure; the pressure required to overcome the weight of the pipe and the friction. This drag pressure will be different for different length of pipes that are to be joined together and should be determined for each weld. Using inadequate drag pressure may result in a weak joint.

Because of the size of the pipe and the importance of the asset it was deemed necessary to carry out pre-construction testing to firm-up on the butt welding parameters. Two pre-construction tests were carried out for this purpose to achieve confidence in the welding equipment and the parameters. Both the pre-construction tests were successful.

Two types of tests were carried out at the pre-construction stage – ISO 13953: 2001 (Type B) and Modified Type A test. Both these tests are destructive tests. Modified Type A test is similar to ISO 13953 Type A test. The Type A test is used for pipe sizes with wall thicknesses less than 25mm. Both these tests determine the tensile strength and the mode of failure of the joint. In order for the test sample to pass, the strength should match the parent pipe's strength and mode of failure should be ductile. Photograph 7 and Photograph 8 show the butt

Photograph 7: ISO 13953 (Type B) test sample

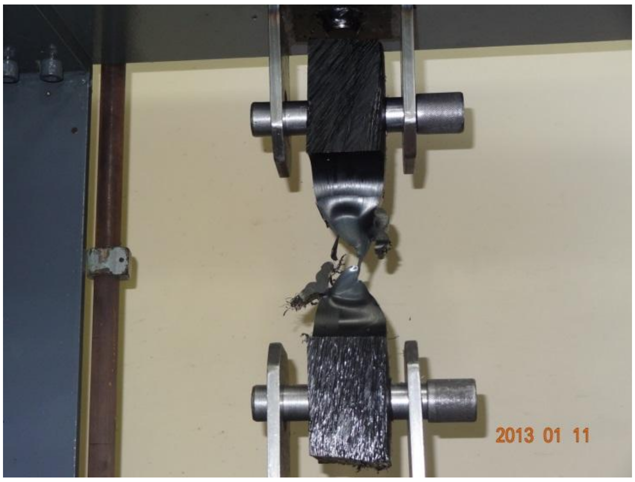
Photograph 8: Modified Type A test sample

welding test samples.

CCC guidelines require butt weld samples to be tested as per Modified Type A. In Modified Type A sample, the actual pipe thickness is reduced to 25mm by shaving off the outer and inner layers and the end sample resembles a Type A sample. The drawback of this test is that it does not test the total thickness of the weld and in case there are cracks or failure zones (cold zone) in the outer and inner part of the joint, these would not be detected. This is why this test alone has not been used to form the basis of confirmation of the welding parameters. During ISO 13953 testing, the total thickness of the weld is tested for strength and failure mode; for this reason, it was decided to carry out both the ISO 13953 test and Modified Type A test to have greater confidence in the welding parameters. At the construction stage, it was decided to use only ISO 13953 test and every one in 20 butt weld is being tested. It was also decided to plot the load deflection graph to better understand the mode of failure of the weld. Photograph 9 shows a successful test sample and Figure 7 shows the corresponding load deflection graph. Figure 8 shows the load deflection graph of a failed test sample that ruptured in a brittle manner.

3.7.2 CONTAMINATION OF THE HEATER PLATE

One of the issues faced during construction stage butt welding was the contamination of the heater plate. Around weld no. 78, the heater plate became contaminated with dust and other foreign matter, such that the welding failed the construction stage testing. The issue was quickly investigated and, once it was determined that heater plate contamination was the cause, it was decided to take extra care in cleaning the heater plate with isopropanol. A dummy weld was carried out after this, which passed the weld test.



Photograph 9:Ductile failure of butt weld during ISO 13953 test

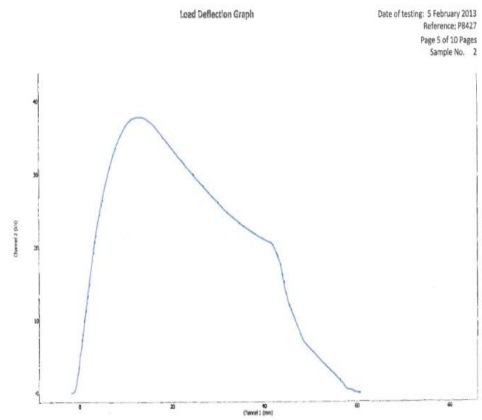
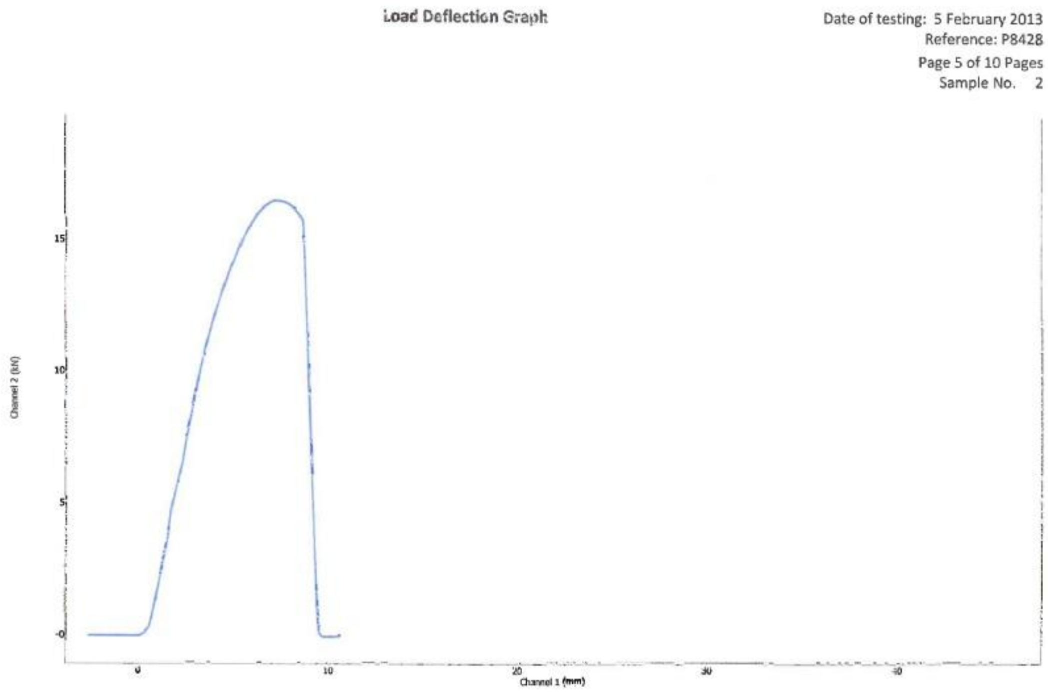


Figure 7:Load deflection graph depicting ductile failure of the butt weld

Figure 8:Load deflection graph depicting brittle failure of the butt weld sample

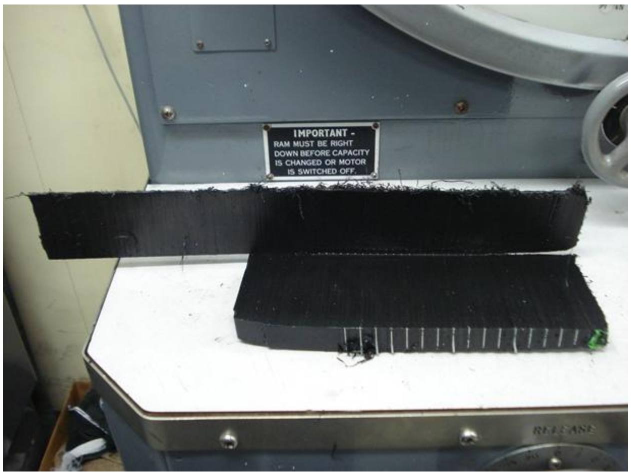


3.7.3 ELECTROFUSION WELDING

The make-up pieces of the pressure main section will be joined using electrofusion couplers. Two ISO tests have been used to determine the strength of electrofusion joint – ISO 13954: 1997 and ISO 21751: 2011; both being destructive tests. ISO 13954 is a peel de-cohesion test and is effective in determining the strength and failure mode of PE electrofusion joints. Two pre-construction weld tests were carried out to check the compatibility of the electrofusion coupler with the PE pipe (see Photographs 10 and 11). During ISO 13954 pre-construction testing, it was found that, due to the thickness of the pipe wall, a significant percentage of failures were in the PE pipe and not at the joint. As per ISO standard, this would be considered a pass, but did not provide any indication as to whether the joint was brittle or ductile. It was decided to test the joint as per ISO 21751: 2011 to get a clear idea about the failure mode of the joint. This is basically a strip-bend test in which thin strips of test samples are bent to check whether the failure is ductile or brittle (see Photographs 12 and 13).

Photograph 10: Peel De-cohesion Test Sample (ISO 13954)

Photograph 11: Ductile Failure after testing



Photograph 12: Strip Bend Test Sample (ISO 21751)

Photograph 13: Ductile Failure after testing

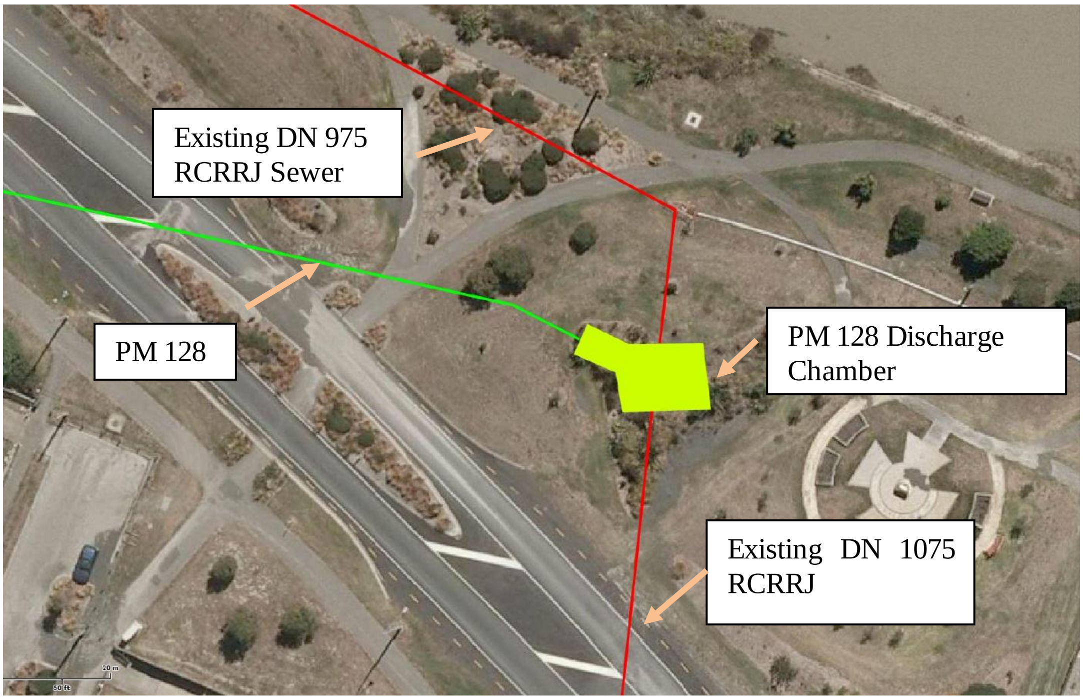
It was decided not to use hydrostatic testing for the EF joint for two reasons: first, it is not a recognised standard test and secondly, it does not provide sufficient information about the joint. In the case of ISO tests (13954 and 21751), the stresses and strains are perpendicular to the weld interface and, if the weld interface is weaker than that of the coupler material or the parent pipe, then the test sample will fail through the interface; thus providing information about the weakness of the joint. During a hydrostatic pressure test, the shear and hoop stresses are induced at the weld interface and, even if the weld interface is weaker than the coupler material and the parent pipe, the sample may not fail at the weld interface, and may fail in the pipe. Hence, pressure test may not provide useful information about the strength and mode of failure of the EF joint (Troughton et al., 2006).

3.8 DISCHARGE MANHOLE

The pressure main will discharge into a manhole to dissipate energy and allow odour release at the designated extraction point. A biofilter will be constructed in the vicinity of the discharge manhole to facilitate odour treatment. This discharge manhole would be located on the existing DN 1075 RCRRJ gravity pipe. The wastewater from this manhole would be conveyed to new PS136 via this RCRRJ pipe. See Figure 9 for an aerial plan showing the configuration.

Due to the falling topography of the selected route, additional static head is required at the discharge end to avoid pressure main drainage and excessive air entrainment during normal pump operation. It was also intended to use the pressure main for gravity bypass during emergency situations by removing this additional static head under controlled conditions. An overtopping weir penstock has been proposed inside the discharge manhole to provide the higher static head. During normal operation, the penstock will be closed to provide a higher head, and will be raised in conjunction with the opening of the automated bypass valve at the inlet of PS128 in order to complete the gravity bypass system.

Figure 9: PM128 Discharge Chamber



CONCLUSIONS

Design of a large pump station and large diameter pressure main presents many challenges and requires careful consideration of several important design factors, especially when earthquake resilience is the governing criteria. Some of these elements for the pump station design have been discussed in this paper; including appropriate ground improvement design, civil design features aiding in earthquake resilience, proper pump configuration and operational and maintenance features. The pressure main project has presented key challenges in terms of route selection, pipe jointing and testing, directional drilling and monitoring of pipe installation. The designer should consider site-specific features when considering directional drilling methodology.

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REFERENCES

ASTM-F1962-11. 2012. Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings. 18 pp. ASTM International.

Brown L.J. and Weeber J.H. 1992. Geology of the Christchurch Urban Area. Scale 1:25,000. Institute of Geological & Nuclear Sciences Geological Map 1.

Ryan, P. and A. Finney. 2012. Pipe Materials and Joint Selection for Trenchless Construction. Pages 928-939 Pipelines 2012: Innovations in Design, Construction, Operations, and Maintenance.

Scoby, G. 2012. Integration of HDPE 4710 Materials for Potable Water Distribution Systems. Pages 177-185 Pipelines 2012: Innovations in Design, Construction, Operations, and Maintenance.

Troughton, M., J. Hessel, and M. Piovano. 2006. Comparison of long-term and short-term tests for electofusion joints in PE pipes. Plastics Pipes XIII conference, Washington DC, USA.