THE CHRISTCHURCH OCEAN OUTFALL - FROM DESIGN TO COMMISSIONING

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ABSTRACT

New Zealand's longest ocean outfall pipeline was officially opened on March 24th 2010. This pipeline transfers treated wastewater from the Christchurch wastewater treatment plant to diffusers located 3 km offshore. As Christchurch City's largest project ever undertaken, the NZ$85 M project comprises a new pump station and ~5 km of 1800 mm diameter pipeline with a design capacity of 6.5 m^3/s. The pipeline comprises 2.33 km of concrete pipe installed by microtunnelling and 2.53 km of HDPE pipe installed by dredge and lay.

An Interactive Tender Process (ITP) was successfully used as a mechanism to transfer knowledge from the owner and designers to tenderers, with the objective of reducing unknowns, bid contingency allowance and the risk of contract claims.

The microtunnel was constructed in three sections, using a Herrenknecht AVN 1800T/DB Microtunnel Boring Machine (MTBM); firstly an 874 m section beneath a tidal estuary, secondly a 604 m drive along a highly sensitive residential street, and thirdly a 830 m section beneath the dunes and surf zone out to the connection point with the HDPE marine pipeline. A 'wet recovery' of the MTBM took place at this connection point. During the tunnelling operation, 7,000 year old shells and timber from an ancient estuarine area, as well as layers of sediment with higher than anticipated silt content impacted on the contractors progress.

During the construction of the 2.53 km HDPE marine pipeline issues arose which resulted in the project being delivered approximately 18 months behind schedule. These included; adverse sea conditions, HDPE weld failure, connection of the marine works to the microtunnel works, and serious health and safety issues during construction. The resolution of these issues along with the lessons learned will feature as a part of this paper.

This paper describes the contractual framework, ITP, partnering, microtunnel advance, adverse sea conditions, marine pipeline construction issues, safety issues and commissioning of the system including initial dilution testing.

KEYWORDS

Ocean Outfall, Microtunnelling, Interactive Tender Process, Pipelines.

1 INTRODUCTION

Christchurch is the largest city in the South Island of New Zealand, located on the east coast, with a population of about 360,000. The city is traversed by two small rivers, the Avon and the Heathcote, both of which terminate at the Avon-Heathcote estuary.

Sewage is collected in a dedicated network which terminates at the Christchurch Wastewater Treatment Plant (CWTP) where primary and secondary treatment is undertaken. Wastewater is then discharged to a series of tertiary ponds approximately 230 ha in area, where the main pathogen reduction occurs. Historically the wastewater discharged into the Avon-Heathcote estuary over a period of 2 - 4 hours during each high tide, from where it flows out to the coastal area of Pegasus Bay.

Whilst the discharge water is generally of high quality, the community had significant reservations about continuing with the discharge to the estuary, as recreational water quality in the estuary was not being met at all places at all times. There was also concern about the levels of nutrients in the wastewater, which were contributing to high growth rates of the algae sea lettuce, particularly over the summer period. (Tipler and Keller 1999).

Following a lengthy period of options studies, consultation and two resource consent hearings, consents were finally granted in November 2005 for an ocean outfall to discharge 3 km offshore, with a requirement to be operational by 30 September 2009.

Site investigations, preliminary design and consideration of project delivery options were undertaken in parallel with the consent process. Once consents were granted, the project then advanced through detailed design, tendering and into construction.

At the time the Ocean Outfall was agreed to, it was believed that a dig and lay option was the most likely construction method for the "on-shore" section of the pipeline, being approximately 2 km, comprising the estuary crossing and South New Brighton Spit section. The potential effects of a dig and lay pipeline operation requiring sheet piling and dewatering along the full length across the estuary and through residential areas raised a number of environmental and community concerns. The main concerns were:

- the Avon Heathcote estuary section is environmentally sensitive containing areas of salt marsh and fresh water springs;
- the South New Brighton Spit is made up of two geographical areas, essentially a recreational reserve containing an area of mature pine forest, and residential streets;
- when the effects of the dig and lay methodology were described to the community, there was a very strong negative reaction to being host to the pipe route;
- during the consultation process, it became evident that launching a pipe string into the ocean from one of the streets would not be an acceptable construction method; and
- localised effects such as noise from sheet piling work, access to property, dust, and damage to property from vibration.

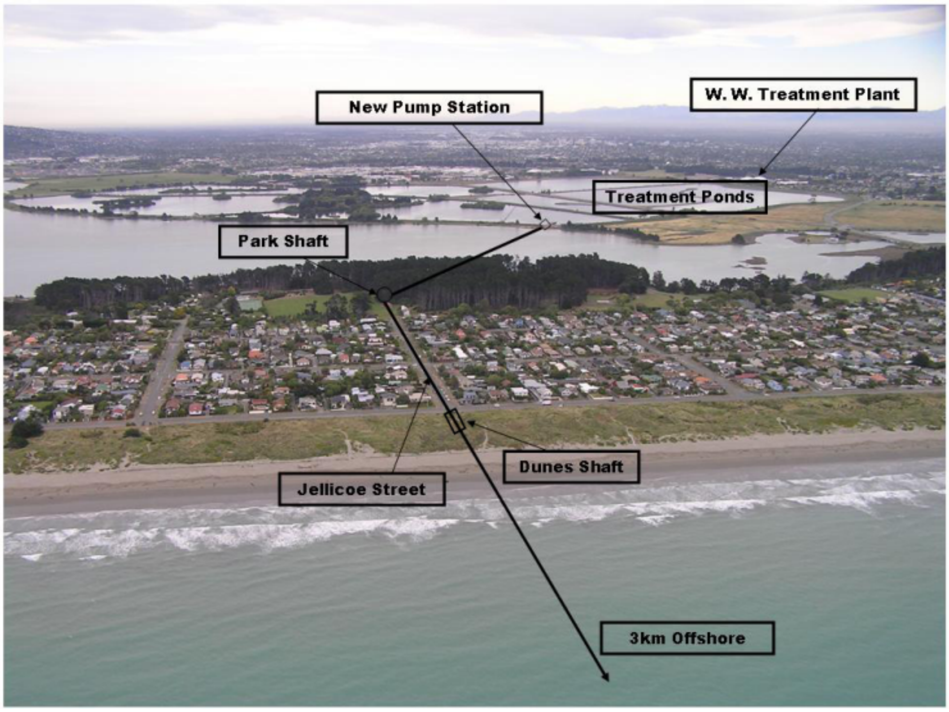
The issues identified encouraged alternative construction methods such as microtunnelling to be explored. Because the technology for the proposed pipe size, and length of drive was not readily available in New Zealand, it was not initially thought that this would be an affordable solution. However, significant advances made in the industry over recent years suggested that, for a project of this magnitude, it may be possible to attract international interest for a microtunnel solution.

With this in mind, designs for both a dig and lay and microtunnel solution were developed. The design for each option has been described in Moore et al (2006).

2 DESIGN CONCEPTS

The pipeline comprises of an on-shore section and an ocean section with the total pipeline length being approximately 5 kilometres. The landward section includes a length of pipeline beneath the Avon-Heathcote Estuary, a section through South New Brighton Park and Jellicoe Street through to an interface point located on the seaward side of the coastal sand dunes. The ocean section includes the section from the interface point, through the beach and surf zone, thence out to the diffuser section located approximately 3 kilometres offshore. This is as shown in Figure 1.

Figure 1 - View of Outfall Route with Christchurch City in Background



The diffusers, located over the final 360 m of pipeline, comprise 13 vertical risers discharging through rosettes with eight duckbill valves each. An over-trawl protection structure has been constructed over each diffuser riser. It was anticipated that a number of construction techniques and pipe materials would have been suitable and therefore the scheme was designed to make allowance for three conforming options:

2.1 PIPELINE OPTIONS

The suitable design options were as follows:

2.1.1 OPTION 1 – DIG AND LAY

Dig and Lay with the proposed pipeline invert generally laid at between 3.0 and 5.0 m below the current ground surface, following the topography, with increased depths at the pump station location and within the South New Brighton Beach fore-dune deposits. This dig and lay construction was expected to be undertaken using a trench up to 5 m depth. The off-shore pipeline would lie between 2.0 and 4.0 m below seabed level.

2.1.2 OPTION 2 – MICROTUNNEL TO BEACH

Microtunnel/Pipe Jacking with the proposed pipeline laid at a constant gradient from the oxidation ponds to the South New Brighton Beach fore-dunes where depth to invert would be about 11.5 m below ground level. From the fore-dunes to the diffuser, the pipeline would be constructed in a dig/dredge and lay operation with the depth to invert level reducing off-shore to about 4.0 m at the diffuser.

2.1.3 OPTION 3 – MICROTUNNEL TO SURF ZONE

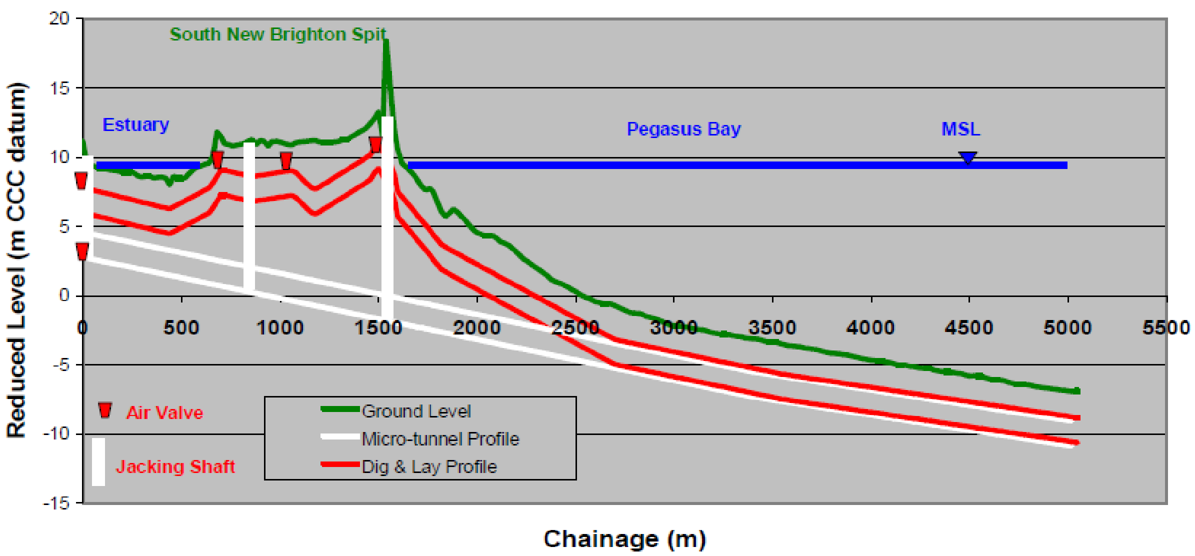
Microtunnel /Pipe Jacking similar to Option 2 with the landward section to beyond the surf zone, then dig/dredge and lay the balance of the off-shore pipe section.

The conforming pipeline materials comprised of 1800 mm NB reinforced concrete rubber ring jointed (RCRRJ) pipe for the Dig and Lay option and 1800 mm NB reinforced concrete jacking (RCJ) pipe for the microtunnel options, and a 1800 mm OD or a twin 1400 mm OD Polyethylene (PE100) pipe for the marine section.

2.2 DESIGN CAPACITY

The average design flow from the Wastewater Treatment Plant is 2.31 m^3/s, with a peak flow requirement of 5.5 m^3/s. The actual capacity of the pipeline as constructed is approximately 6.5 m^3/s.The available static head for gravity flow ranges from 1.9 – 4.8 m, providing gravity flow up to average design flow for nearly all conditions. To facilitate flushing to remove slime growths and air pockets, a maximum diameter of 1800 mm was determined. It was anticipated that flushing at 4.8 m^3/s would be required for one hour each day. Figure 2 depicts the pipeline profiles for the two conforming designs, based upon dig and lay and microtunnelling.

Figure 2: Microtunnel and Dig and Lay Pipeline Profiles



3 PROJECT DELIVERY

3.1 INITIAL CONSIDERATIONS

During the preliminary design phase a number of project delivery workshops and risk workshops were held between the Council, their legal advisors and URS to determine the best approach for this project. Considerations were:

- a single contract covering both the pump station and the pipeline or two separate contracts;

- design-build or design-bid-build form of contract(s);
- General Conditions of Contract most suited to the project;
- the inclusion of a number of construction options in the tender process;
- identification of the project risks and the inclusion of appropriate risk management provisions; and
- the tender process, including a mechanism for early contractor involvement.

The approach taken is discussed in the following sections.

3.2 SINGLE OR SEPARATE CONTRACTS

A single contract covering the pump station and the pipeline construction was initially favoured as it would place the responsibility for co-ordination of the works on the contractor and avoid the Council having to resolve interface issues. This was seen to be efficient in terms of programming and sequencing of the interface works by one contractor, which may have avoided potential delays with one section waiting on the other.

An issue that had to be addressed was which party would be responsible for the operating performance of the pump station and pipeline. The hydraulic performance of the overall system was the responsibility of the hydraulic designers URS, with the contractor(s) being responsible for ensuring that the pump station and pipeline performed to the specified hydraulic parameters. While a single contact was believed to simplify this division of responsibility, it was not considered an insurmountable issue with separate contracts, although the interface would need to be well defined.

In examining the question of single or separate contracts more closely, consideration was given to the required skill sets for construction of the various sections of work. While the pump station construction did face a number of challenges relating to the existing treatment pond embankment, the majority of work was considered straightforward, including sheet piling, earthworks, civil construction, mechanical and electrical procurement and installation. As such it was well within the capabilities of a number of local contractors.

On the other hand, the pipeline comprised three potentially very difficult elements of work that collectively represented the highest risk and highest cost elements on the project. These were the section of pipeline beneath the estuary, the Jellicoe Street section and the marine section. It was considered that the estuary crossing and the marine section in particular required a skill set that was very limited locally and hence was expected to require a specialist marine pipeline contractor with international experience.

In considering a single contract, it was expected that the lead contractor would be the specialist marine contractor, with the pump station being sub-contracted out. There was considerable doubt whether an overseas specialist marine contractor would be the most appropriate team to manage the pump station contract. Given that Council would have a project team managing the project, the lead contractor’s margin and potential lack of knowledge on pump station works, including the required interfacing with Council operations personnel, was a downside and hence this option did not appear to represent any added value to the project.

The decision was therefore taken to separate the pump station and pipeline works into two contracts, with Council’s project management team managing the interface issues. While there were some interface issues, mainly around the respective timing of work each side of the interface, these were relatively minor in terms of the overall project and were successfully resolved at the time.

Notwithstanding this approach, tenderers were invited to bid on both contracts and provision was made in the two contracts to facilitate this, ensuring that the forms of contract and General Conditions were compatible.

3.3 FORM OF CONTRACT

Initial considerations favoured a design-build contract based on the preliminary design work carried out by Council’s in-house design team on the pump station and URS on the pipeline. This was re-visited during the project delivery and risk workshops where a traditional design-bid-build form of contract was considered as an alternative.

The determining factor was the degree of control the Council would have over the final design. With the design-build approach, the contractor largely determines the design to suit its own methodology and skill set, within the specified owner's requirements. As such, the owner's control over the design and the flexibility to make changes and consider alternatives is more limited than a traditional design-bid-build form of contract. With a design-build contract, options outside the contractor's skill set are unlikely to receive due consideration.

With the Ocean Outfall being a 100 year design life facility and the Council taking over and operating the asset on completion, it was very important that the Council maintained input into and control over the design to ensure that quality and operability were not compromised. This included the flexibility to consider alternative materials and configurations for the structures.

The chosen form of contract for both the pump station and the pipeline was therefore a design-bid-build contract.

3.4 GENERAL CONDITIONS OF CONTRACT

The two criteria used to determine the most appropriate General Conditions of Contract were:

- familiarity with the documents for an international contractor; and
- compatibility between the pump station and pipeline contracts in the event that tenderers wished to bid on both.

NZS3910 was considered insufficiently robust for the high risk nature of the work without extensive special conditions. Many of the international marine contractors expected to bid for the project would not have used this form of contract before.

NEC was also considered and while the collaborative emphasis of NEC was a positive aspect, it has not been used widely locally and hence it was not favoured.

The widely used international conditions of contract FIDIC (Fédération Internationale des Ingénieurs-Conseils), was chosen as the most appropriate for the pipeline contract, given that there was likely to be interest from European bidders. The "Red Book" (Conditions of Contract for Construction) was used on the pipeline and the pump station contracts.

3.5 TENDER OPTIONS – PIPELINE CONTRACT

During the design phase a number of options for the pipeline were evaluated to determine whether there was an optimum solution in terms of technical feasibility, capital cost and operating cost. For this value engineering exercise the pipeline was split into the "land-based" section, comprising the estuary crossing and the sand spit section; and the marine section, comprising the section from the sand dunes out to the diffusers offshore.

3.5.1 ON-SHORE SECTION

Options considered were:

DIG AND LAY METHODOLOGY

Dig and lay methodology using sheet piling and trench dewatering is the traditional pipe laying methodology and there are a number of very capable local contractors with extensive experience in these conditions. However, this construction would not be a normal, straightforward dig and lay operation and would result in a number of risks or probable additional costs as outlined below.

- The section across the estuary would require the construction of a temporary embankment or working platform, sheet piling of the trench and dewatering to permit pipe laying and backfilling to take place. This would be difficult construction which had a high potential to produce adverse construction effects on the estuarine environment. It would be a deep, high risk, high cost trenching operation through marine sediments.
- There had been considerable concern from local residents over the impact the construction works might have along Jellicoe Street, a residential street on the sand spit.
- The dig and lay pipeline would generally need to follow the ground contours due to digging depth constraints and therefore would not provide a gravity flow solution. There would therefore be an operational cost penalty for this option for the continuous pumping.

MICROTUNNELLING METHODOLOGY

Microtunnelling methodology was considered during the initial design phase and almost discarded as likely to be too expensive. However, during the value engineering process when the difficulties associated with dig and lay were being examined and fully understood, it was decided to re-visit the microtunnelling option.

The advantages this option offered over the dig and lay methodology were:

- no adverse environmental effect on the estuary;
- minimal adverse effects on the Jellicoe Street residents;

- no depth limitations, resulting in a gravity pipeline, reducing pumping requirements to peak and flushing flows;
- no drop structure required at the sand dunes;
- faster construction of the pipeline;
- no dewatering required and hence reduced potential to have any adverse effect on adjacent structures such as houses; and
- potential to continue the microtunnel out beyond the surf zone, overcoming a potentially high risk section of the marine pipeline.

The main disadvantage appeared to be the extra cost of microtunnelling over dig and lay in a normal pipe laying operation.

Comparative cost estimates were developed during the design for both the dig and lay and microtunnel options which demonstrated that the two options were likely to be very similar in terms of scheduled cost, although the microtunnel option offered a number of intangible benefits. It was therefore decided to include both methodologies in the tender documents for pricing under the following options:

- Option 1 – dig and lay from the pump station through to the sand dunes, including a drop structure;
- Option 2 – microtunnel from the pump station through to the sand dune, to a connection to the marine pipeline; and
- Option 3 – microtunnel from the pump station through to a point beyond the surf zone, to a connection with the marine pipeline.

While tender prices were very close, Option 3 came through as the lowest price tender by a small margin, but as described above, it provided substantial advantages in terms of environmental and social effects, and whole of life costs.

3.5.2 MARINE SECTION

A risk identified during the design phase was the limited suppliers of 1800 mm OD longitudinally extruded thick wall plastic (HDPE) pipe to the required specification for the marine section. Investigations indicated that supply would be likely ex Europe, with a very high cost component for transport.

As an alternative, twin 1400 mm diameter pipes were included in the tender documents for pricing, as these could be manufactured in Australia or Asia. Subsequently, tenders for this option proved more expensive and hence this option was not taken any further. The successful tenderer, McConnell Dowell Constructors Ltd, (MacDow) offered and had accepted a locally manufactured alternative, tangentially extruded HDPE pipe, overcoming the high transport costs.

The tender documents also made provision for the contractor to submit alternative designs for elements of the marine section, including the microtunnel / marine pipe connection, the location of that connection point, the design of ballast blocks along the pipeline and the design of the diffuser structures including the over-trawl structures. MacDow re-designed these elements to optimise constructability. All alternative designs were subject to review and acceptance by URS and Council.

3.6 RISK MANAGEMENT PROVISIONS

The pipeline project was recognised from the outset as having a number of high risk elements that needed special attention to mitigate the risks to Council to an acceptable level, whilst providing a fair sharing of risks between the Council and contractor.

The following risk management provisions were adopted for the pipeline project:

3.6.1 GEOTECHNICAL BASELINE REPORT (GBR)

In addition to the factual Geotechnical Data Report (GDR), an interpretative GBR was provided to tenderers. This was specifically relevant to the land-based dig and lay or microtunnel works and is commonly used on underground works to set trigger points or baseline levels which define the point where the risk transfers from the contractor to the owner. A condition more adverse than the specified baseline provides grounds for a changed condition claim, subject to the contractor demonstrating an adverse effect on its operation in terms of time or cost.

The GBR for the pipeline contract was primarily aimed at the microtunnel operation where the presence of obstructions such as buried trees or roots, ship wrecks, services, lack of cover under the estuary and the anticipated ground conditions are key risk components. For a slurry microtunnel operation the percentage of silt anticipated determines the size of the separation plant and hence is a critical factor.

Other baseline conditions such as weather conditions or sea state conditions were defined in the contract documents. For the marine works, sea state conditions over a range of wave heights for different activities (0.5 - 1.5 m) were defined as grounds for an extension of time, but not recovery of costs.

3.6.2 DISPUTES REVIEW BOARD (DRB)

The DRB is another initiative that has come out of the international tunnel industry, to avoid the need for premature legal involvement, arbitration or litigation on claims at great expense to both parties.

The three person DRB set up for the pipeline project comprised experienced New Zealand underground and construction engineers. The DRB was initiated soon after commencement of the project and met every three months or so to receive updates on progress and any contractual issues that might be developing. The intent of the DRB is that should a dispute arise which goes beyond the Engineer’s discretion, the DRB would be in a position to quickly understand the issue, hear the respective viewpoints and make a decision, without the “waters being muddied” with legal arguments. While each party has recourse to arbitration following that decision, the history of DRBs indicates that this has rarely happened.

The DRB process may be viewed as an insurance mechanism to avoid litigation. However, in testament to the good relationships between the Contractor, Council and Engineer, all claims were resolved before being escalated to an Engineer’s decision and hence the DRB was not called on to hear a dispute.

3.6.3 ESCROW BID DOCUMENTS (EBD)

The EBD comprise a copy of the successful tenderers detailed bid calculations and assumptions, which following award, are sealed by the tenderer and stored in a secure storage (escrow). In the event of a claim, these documents may be opened by either party or the engineer under strictly controlled conditions, to facilitate resolving a dispute over the basis of the tender. The need to access the EBDs did not eventuate on the pipeline contract and the documents were returned to the contractor unopened.

3.6.4 PARTNERING

Council signalled during the tender process that a collaborative approach to the project was preferred and asked tenderers to offer a partnering or similarly collaborative mechanism. This would be outside the Contract Conditions.

MacDow submitted a partnering proposal which was then developed jointly and successfully introduced into the project. This involved regular meetings at a senior level between Council, MacDow and URS to review key performance indicators and identify areas for improvement. It also provided a very strong mechanism to develop relationships during the project and informally air concerns.

3.7 INTERACTIVE TENDER PROCESS

The project included a number of elements with a very high degree of difficulty and a number of construction risks as mentioned above, such that it was important to identify and where practicable provide for these in the tender documents. It was considered crucial that the Procurement Plan included early engagement of tenderers during the later stages of design to obtain constructability input and contractor buy-in as well as ensuring that the contractor had the right mix of resources, relevant experience and financial backing to undertake and complete the project.

An Interactive Tender Process (ITP) as described below, was utilised to achieve these goals.

3.7.1 PRE-SELECTION OF TENDERERS

A registration of interest document was advertised internationally which drew a good response from local and international contractors. This document comprised a brief project outline, a description of the various elements of the project and a number of returnable schedules covering: Details of Contracting Party; a General Statement of Intent (proposed methodology); Relevant Experience; Track Record; Key Personnel; Management Details; Financial Capacity and proposed Major Plant and Equipment. No pricing was required at this stage.

A weighted attribute scoring system was used by the Tender Evaluation Team (TET) to assess the respondants and from this process, five potential tenderers were selected.

3.7.2 INTERACTIVE TENDER PROCESS

The pre-selection of tenderer was staged so that the ITP could commence when the detailed design was at or about the 90% complete stage and draft tender documents were ready for issue. The ITP comprised the following process.

- 90% Draft Tender Documents issued to the five pre-selected tenderers for review and comment.
- Interactive meetings were held on site over two days which included a combined group site visit, a group meeting, access to the data room containing reference reports and two hour individual meetings with each tenderer. At the group meeting the project scope and constraints were presented by URS and Council, and Council outlined its expectations for the project. Questions and comments from the groups were welcomed. The individual meetings were to permit tenderers to raise queries of a commercially confidential nature including potential innovations for the project.
- Receipt and consideration of written comments from the tenderers on the tender documents. Council reserved the right to accept or reject any suggestions.
- Completion of the tender documents, incorporating comments where accepted.
- Issue of the final tender documents to each tenderer for pricing, including a “tracked changes” copy to simplify the tenderers’ task. Tenderers were also required to re-submit returnable schedules, focusing more specifically on the final scope of work and methodology.
- Receipt of tenders and tender evaluation using a weighted attribute system.
- Determination of a preferred tenderer and negotiations to finalise the contract.

The ITP has proven to be a successful risk management mechanism on a number of projects in New Zealand. The process achieves a number of important goals including: a greater transfer of knowledge from the designer and owner to tenderers; good constructability input into the design before it is “set in concrete”; and an opportunity for individual tenderers to ask commercially confidential questions on innovative ideas. Experience from a number of projects suggests that the ITP achieves the objective of reducing contractor risk contingency money in the bid because of a better understanding of the risks and reduces the opportunity to claim latent conditions for the same reason.

3.8 TENDERS RECEIVED

Four tenderers provided six conforming and ten alternate tenders for the project. One tenderer submitted seven alternative tenders. A fifth selected tenderer declined to bid. Tenderers had been encouraged to provide innovative solutions and the alternative tenders and the number of tenders received reflected this innovation.

The tenders received were based on different combinations of the scheduled options for dig and lay or microtunnelling, with a number of non-conforming options including a different contract form, non-conforming pipe materials and reduced pipe diameters. Methodologies and the plant offered varied widely. While the on-shore sections were simply dig and lay or microtunnelling, a number of innovative methodologies for the estuary section and the marine pipeline were received using different types of dredges and pipe laying equipment.

The successful tenderer MacDow, provided innovative approaches to the microtunnel shaft design, the microtunnel-marine pipeline joint, the marine pipeline ballast blocks and the diffuser / over-trawl structure design. These were all based around improving the constructability of the structures. MacDow's chosen option was Option 3, comprising microtunnelling out beyond the surf zone and a single 1800 mm OD HDPE marine pipe beyond to the diffusers. MacDow achieved cost savings by having the concrete microtunnel pipes manufactured in Thailand and the HDPE pipes manufactured locally using a tangentially extruded HDPE system with the equipment supplied from Germany.

As predicted during the design phase cost estimating, the difference in cost between dig and lay and microtunnelling was small, with the preferred microtunnelling tender being marginally lower priced. Council indicated to tenderers during the ITP phase that a microtunnel option was likely to be preferred over dig and lay, subject to the cost being competitive. It was also stated in the tender documents that a number of "intangible" factors that favoured microtunnelling may be taken into account in the tender evaluation if the respective tender prices were very close. These factors included environmental and community impacts, and the difference in whole of life pumping costs. With the microtunnelling option being the lowest conforming tender this provision was not required.

4 CONSTRUCTION SEQUENCE

The outfall pipeline from the pump station at the CWTP ponds to the discharge point some 3 km off-shore in Pegasus Bay comprises:

- 2,309 m of 1800 ID concrete lined microtunnel from the CWTP Pump Station to 600 m off-shore c/w Pigging 'Wye' and Air Relief Valves at the pump station end;
- 2,537 m of 1800 OD SDR26 HDPE pipeline with 15 T Ballast Blocks at 6.75 m centres buried under seabed with minimum cover of 2 m;
- microtunnel/marine pipeline connection; and
- 13 Diffuser risers at 30 m centres c/w duckbill valves, concrete protection structures and scour blanket.

4.1 MICRO-TUNNEL CONSTRUCTION

The microtunnel pipeline was constructed in 3 sections (drives).

- The 1st drive was 874 m - a record length for Australasia - from the main shaft in South New Brighton Park back to a smaller and shallower shaft at the CWTP Pump Station.
- The 2nd drive was 604 m from the South New Brighton Park shaft to a smaller and deeper shaft in the dunes at the end of Jellicoe Street.
- The 3rd drive was 830 m from Dune shaft 600 m out to sea to where it is connected to the marine pipeline.

The drives were all straight, with a down gradient of 0.3% from the pump station at Construction Management Area 1, (CMA1). At CMA2, in South New Brighton Park, there is a 30 degree change in direction.

4.1.1 SHAFT CONSTRUCTION

The shafts were constructed as sheet piled pits, with CMA 1 and 3 shafts being rectangular 6 m x 12 m, while CMA 2 was a circular shaft, 14 m diameter. Sheet pile types AZ34 or AZ36 with a length up to 24 m were used for the shafts.

The CMA 1 shaft was constructed adjacent to a large sheet piled cofferdam excavated for the pump station construction (constructed by Downer EDI). The shaft excavation therefore had to be dewatered and constructed in the dry to avoid any adverse effects on the adjacent works. Two stage vacuum well point dewatering was used to depress the water table sufficiently to place the reinforcement and concrete for the base slab in the conventional way.

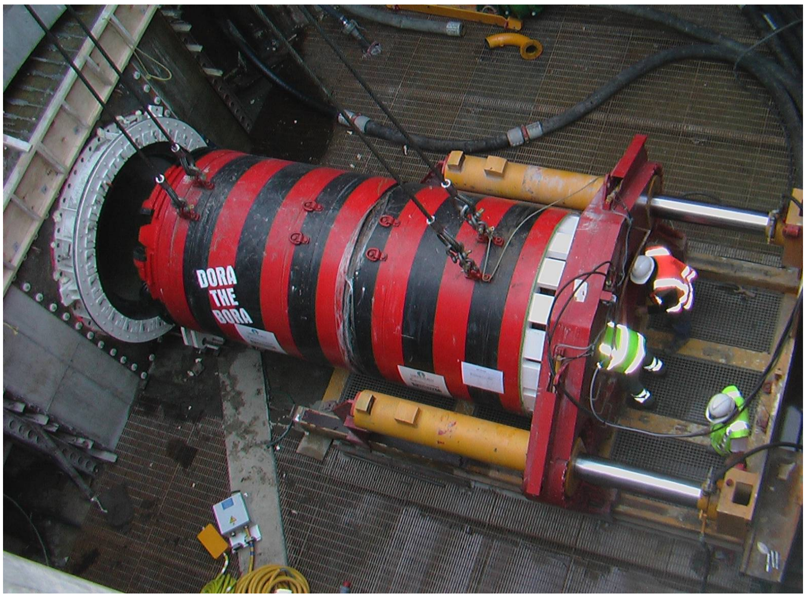
The CMA 2 and 3 shafts were constructed partially "in the wet". They were initially excavated in the dry using a sump pump to lower the water table to about 8 m depth. Walers were fitted before the shaft was allowed to flood, with the remaining excavation being carried out with a clamshell grab in the wet. Final excavation trimming was carried out by divers using airlift equipment. The divers then coordinated the placement of prefabricated reinforcement cages and tremie poured concrete for the base slab. Twelve sand screw anchors were installed in the base of the CMA2 shaft to assist the piles in resisting uplift from the ground water on the base slab. The shaft was then able to be pumped out with the minor leakage remaining being within the capacity of a small sump pump.

4.1.2 MICRO-TUNNEL BORING MACHINE (MTBM)

As the site geology comprised sand that would exhibit a running behaviour under the hydrostatic pressure, a closed face MTBM was specified. A Herrenknecht AVND-1800-AB MTBM was purchased by MacDow specifically for this project and delivered to site in March 2007. The AVND is a slurry shield MTBM that incorporates earth pressure balance technology. The machine can be used in conventional slurry mode or if necessary it can be used in D-mode – a fully automated pressurised mode where a pressurised air "bubble" is used to ensure constant slurry pressure at the face. The latter may be necessary for example in the case of gravelly geological conditions being encountered, an unstable tunnel face, mixed geological conditions or where shallow overburden increases the risk of blow out.

The MTBM chosen for this project had a mixed face cutter head with 10 soft ground picks and 10 discs to be able to cope with any unexpected rock or cobble formations. The cutter head was 2185 mm diameter, cutting a 37.5 mm oversize annulus around the jacking pipe. The cutter head was able to be reversed and operated at about 4.5 RPM. The jacking pipes were vertically dry cast reinforced concrete. 1800 mm ID and 2120 mm OD manufactured in Thailand by CPP Ltd, Bangkok.

Figure 3 - TBM being lowered into the South New Brighton Jacking Pit at CMA 2



4.1.3 SEPARATION PLANT

The slurry MTBM utilised a closed circuit slurry circulation and muck separation system located on the surface. It comprised a number of slurry tanks which were dosed with bentonite and pumped to the MTBM cutter head, providing face support and a means of transporting excavated material to the surface for removal by screens and centrifuge. The slurry was transferred between the MTBM and separation plant through 150 mm diameter pipes and slurry pumps.

4.1.4 DRIVE 1 CONSTRUCTION

Drive 1, the 874 m push across the Estuary, was commenced on 23 May 2007 with the MTBM pushing through the tunnel eye in the CMA2 shaft. Over the first 200 to 300 m of drive some variable ground conditions were encountered that resulted in difficult driving and variable progress. The variable ground conditions encountered included shell formations, cobbles and timber. Most of the timber encountered came through the slurry system as finger sized rounded fragments which, like the shells, probably originated from a ~7,000 year old beach formation. There were some instances where larger fragments of timber had been chewed by the cutter head, jamming it on a number of occasions on what was believed to be larger driftwood timber pieces. As a result of the timber encountered, the cutter head had to be reversed at times in order to advance the machine and several times the slurry flow also had to be reversed to clear the cutter head ports.

Once under the estuary, a higher silt content in the face was periodically encountered that slowed down the extraction of fines in the separation plant, which in turn resulted in a slowing of MTBM progress.

Tunnel invert level varied from 8.4 m below existing ground level at CMA 1 to 11.5 m at CMA2. The lowest point of cover was under the main channel of the estuary where cover was about 4.2 m. The water table was within one metre of ground surface. During the first quarter of the drive, a number of sinkholes appeared above the tunnel line. These appeared to be attributable to the stop-start nature of this first section of drive where these variable conditions were encountered. While of little consequence through this section, the potential for sinkholes in Drive 2 down Jellicoe Street was of concern due to the presence of the existing sewer, other services, residential properties and risks to public safety.

The maximum jacking force required on Drive 1 was 600 T, which was after a 72 hour stoppage to reinforce a damaged launch seal. Typical jacking loads for most of the drive were 300 to 400 T, with the jacking force increasing gradually with drive length, as would be expected. This good result was attributed to the automated annulus injection of bentonite around the pipe. Intermediate Jacking Stations were installed but not activated.

Notwithstanding the variable conditions, the 874 m long drive successfully holed through into the CMA 1 shaft on 3 September 2007, approximately 15 weeks after starting, at an average rate of about 7.5 m/day. Delays to pipe deliveries affected the initial progress, however once these had been overcome and driving conditions improved, a steady rate of about 5 pipes were installed each day (15 m/day) on two 12 hour shifts. On the best day, 12 x 3 m pipes were installed in a 20 hour day.

4.1.5 DRIVE 2 CONSTRUCTION

As a result of the sinkholes occurring in Drive 1, there was considerable concern by the team over the implications of sinkholes along the residential street. Additional site investigations were carried out to determine whether variable conditions as encountered in Drive 1 would be present and an emergency contingency plan was prepared.

MacDow activated the D-Mode feature on the MTBM for Drive 2 to mitigate any risks associated with settlement and sinkholes along Jellicoe Street.

The 604 m long Drive 2 was commenced on 11 October 2007 and successfully completed on 28 November 2007, a duration of about 7 weeks and an average rate of 12.5 m/day. Jacking loads increased from around 80 T to 200 T at completion, well within the 850 T capacity of the main jacking station. Intermediate Jacking Stations were not activated.

Importantly, the drive was completed without any sinkholes or any other incidents and settlement monitoring was well within the design parameters, with centreline settlement less than 4mm along Jellicoe Street.

4.1.6 DRIVE 3 CONSTRUCTION

Drive 3 construction commenced from the dunes shaft at CMA3 on 3 January 2008, after the separation plant and other facilities were relocated from CMA2. The original drive length was to be 830 m, however as the marine trench dredging had commenced, MacDow made the decision to stop the drive some 22 m short to ensure that the MTBM and the leading pipes remained in stable ground.

The drive was completed on 4 February 2008, an average rate of 25 m/day, on two 12 hour shifts, seven days a week. The best day was 45 m. Ground conditions were excellent, with very little silty material being encountered. The jacking loads increased to over 600 T at about 700 m, prompting the activation of up to three of the interjack stations.

4.1.7 WET RECOVERY OF MTBM

Drive 3 was from the CMA3 shaft in the sand dunes to a point some 650 m beyond the Mean Sea Level shore line. At that point the water depth is approximately 7.5 m below MSL and there was approximately 3.7 m cover to the MTBM. MacDow had chosen to recover the MTBM using a “wet” recovery method rather than constructing a recovery shaft. The dredger ‘Machiavelli’ completed the MTBM excavation on 12th March 2009. The MTBM was refloated and towed back to Lyttelton on 29th March 2009.

4.2 MARINE PIPELINE

The marine section of the outfall was constructed in the following sequence.

- Assemble 7 x 360 m pipe strings on land in Lyttelton.
- Launch and sink the pipe strings into temporary storage in Lyttelton Harbour.
- Use the Backhoe Dredger ‘Machiavelli’ to excavate the pipe trench.
- Re-float & tow pipe strings to Pegasus Bay and sink them into the trench
- Join & test the pipe strings.
- Dig out and recover the MTBM and connect the tunnel to the marine pipeline.
- Place the diffuser protection structures and partially backfill the diffuser section.
- Place the diffuser scour protection.
- Complete pipe trench backfill and test.

4.2.1 MARINE PIPELINE CONSTRUCTION

Dredging was commenced by MacDow in November 2007 using the spudded dipper dredge Machiavelli, owned by sub-contractor Heron Construction Ltd, a NZ dredging company. The 53 m barge is fitted with a Liebherr P994 excavator using a 5 m^3 bucket with a maximum dredging depth below water of 20.8 m. The Dredge stayed on station by deploying three 30 m long and 60 T spuds and achieved the desired excavation with a DipMate RTK GPS bucket positioning system to dredge a 4 m deep trench, 4 - 6 m wide at the bottom with 1V:3H batters. As each section of trench was completed, a 360 m long pipe string was towed into Pegasus Bay from the storage area in Diamond Harbour to be sunk into the trench and joined together prior to being backfilled.

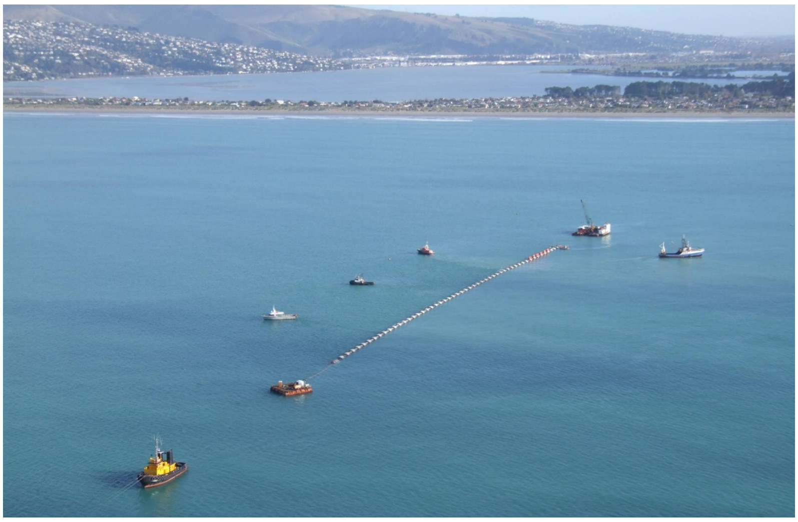
Figure 4 – Machiavelli Dredge

The seven 360 m pipe strings were assembled on a 400 m long slip way construction area at the Lyttelton CMA After capping and testing to 4 Bar, the completed pipe strings were launched and sunk into a temporary storage area in Diamond Harbour. Pipe strings were refloated, towed out and sunk into position whilst maintaining up to 35T axial tension with a 50T winch on board the Tug and winches on board the Flexifloat crane barge, both on anchor.

Figure 5 – Launching diffuser section on slipway at Lyttelton CMA

The last pipe string installed was the diffuser section, which incorporates the 13 risers each with eight duck bill valves that will discharge the wastewater into the ocean environment. Each diffuser riser was then fitted with a pre-cast concrete over-trawl protection structure to prevent damage from trawlers.

Figure 6 – Pipe string held in position in Pegasus Bay prior to sinking



5 CONSTRUCTION CHALLENGES

5.1 HEALTH AND SAFETY

Health and Safety is a high priority for the Council, so during the planning and tendering period, particular attention was given to the selection of the contractor. MacDow scored highly for its management systems, safety record and the paramount importance of Health and Safety in the company culture.

The project was originally programmed for a 22 month construction period, but took 42 months largely due to weather and sea state conditions. This protracted construction timeframe, in parallel with environmental conditions, presented extraordinary Health and Safety challenges for the contractor which needed to be carefully managed. The comprehensive systems and processes MacDow put in place included reviews of all construction and operational procedures and auditing undertaken on regular occasions, during which great emphasis was placed on hazard identification and mitigation.

The tunnel drives proved to be very successful with no significant Health and Safety incidents. This was due largely to the systems and processes MacDow implemented, but also the high level of control MacDow had over its work environment.

This contrasted with the marine works required to be carried out in an open ocean environment that cannot be controlled. Weather and sea state impacted on MacDow's ability to safely carry out the works and also caused delays and rework which resulted in a significantly extended contract period and associated risks.

That a number of Health and Safety incidents occurred during the marine works, in spite of MacDow's strong Health and Safety processes and procedures, highlights the extraordinary challenges faced in working in a marine environment and how risks can never be underestimated particularly when working over extended periods in the ocean.

Environmental conditions that need careful attention when considering Health and Safety in a marine environment include sea state - how it can change rapidly and how it impacts on boat motion, and movement relative to other plant. With the marine site being 12 nautical miles from the onshore site in Lyttelton Harbour, an unexpected and sudden change in conditions that required moored barges, workboats and personnel to immediately cease work and return to shore could be particularly challenging.

Diving conditions were also challenging, with divers working at depths up to 21 m in near zero or zero visibility. Difficulties were experienced sourcing sufficient skilled labour as well as maintaining continuity of staff across some key rolls. Working with subcontractors and understanding their particular risks and Health and Safety issues required close consideration. For both tunnelling and marine works, working 24 hours a day, seven days a week over multiple sites, also created significant Health and Safety issues.

5.2 DIFFERING PHYSICAL CONDITIONS CLAIM

Soon after commencement of the first microtunnel drive, the MTBM encountered conditions in the face that periodically jammed the cutter head and resulted in the MTBM stopping or advancing very slowly in an intermittent manner. Apart from the difficulty in progressing the tunnelling, intermittent stopping and starting a slurry type MTBM in soft ground conditions can lead to face collapse and sink holes appearing in the ground above. The initial section of pipe also appeared to be "floating" behind the MTBM, rising up outside tolerance, thought to be caused by the soft and disturbed ground conditions above the tunnel crown.

Investigation drilling carried out prior to letting the contract had encountered silty sand material throughout with very spasmodic indications of shells. No timber was noted. Apart from the unidentified obstructions adversely affecting advance rates, it was noticeable that considerable quantities of shells and timber were removed on the separation plant screens. The timber and the significant volume of shells encountered were not anticipated in the GBR and the adverse effect resulted in a Differing Physical Conditions claim by MacDow, which was accepted by Council.

5.3 PIPE STRING FAILURE

MacDow attempted to install the first pipe string on 19th Dec 2007 following the dredging works in Pegasus Bay. However, during the sinking of the pipe string into the prepared trench, the pipe suffered a rupture at a weld, resulting in an uncontrolled sinking of the pipe. The failed weld was cut out of the pipe string and retrieved for examination and investigation. The two sections of pipe string were subsequently removed from the trench and were temporarily stored on the seabed before recovery and repair.

The HDPE weld process is reliant on good practice and consistent welding procedures. The contract specification called for 1 % of the butt welded joints to be independently tested. This is a destructive test process as the weld has to be cut out of the pipe joint with the result that passing weld tests have to be re-done. The only form of non destructive monitoring was by examining the consistency of the welding process for timing, temperature, pressure and visual inspection of the weld for bead size and width.

The consequence of the failure was a delay to installing the first pipe string due to having to recover the damaged string 1, investigate the failure, review and modify welding procedures, cut out, test and redo other welds made from on or about the time the failed weld was done, and gain confidence in the pipeline integrity and installation procedures. MacDow also increased the frequency of the destructive tests. Delays for rework on the trench excavation due to the natural backfilling of the trench were exacerbated by weather/sea conditions which limited operations in Pegasus Bay.

5.4 JOINT CLOSURE

The marine section of the outfall pipeline was assembled from seven HDPE pipe strings each 360 m long. These pipe strings all needed to be joined underwater. Each pipe string had a series of 15 T precast concrete weights bolted at 6.75 m centres resulting in each string weighing 1300 T in air. The methodology developed to join the pipe string in the 4 to 6 m deep pre dredged trench at underwater depths of between 8 to 20 m involved the manufacture of a pair of mating frames that attached to the ends of the pipe strings allowing the frame on the pipeline in place to receive the next pipe string as it is was sunk into place. During the sinking of each pipe string an up to 35 T axial load was applied to each string by the winch on a moored tug to limit the amount of bend in the pipe string during the procedure. The end of the pipe string being sunk into place was pulled down to be received by the mating frame attached to the previous string. Air was then carefully bled out of the pipeline at the surface while at the same time water was introduced at the submerged end so that the pipe gradually sunk from one end to the other.

During the joining of the first two sections, it was found the control that could be achieved during the sinking of the pipe string was not adequate to allow the bolting of the flanged connections during the installation procedure window, with dry runs on land not proving realistic at sea. Poor or nil visibility, and lack of clearance between the pipe ends and under the joint, made this a very dangerous and time consuming operation. This resulted in the pipe string being sunk approximately 0.5 m short of the first pipe string. A spool piece was manufactured and inserted to close the gap between these first two strings.

In considering the solution of installing a spool piece, the Council was concerned about the additional risk posed by an extra joint, changes to the Cathodic Protection and associated monitoring and maintenance requirements. Although this was accepted for this first joint, a revised construction methodology was requested to overcome the difficulties encountered with the first joint.

MacDow devised a new procedure that would allow the subsequent pipe strings to be laid within a 0.5 m gap. The properties of the polythene pipe would be taken advantage of to stretch the pipe to close the gap and allow the pipe to relax over time to reduce any tension at the joint. This methodology involved the loosening of bolts on the weight blocks for 180 m on one side of the joint, inserting long bolts to span the gap between strings, then using hydraulic rams fitted to the mating frames to stretch the pipe so the two flange faces would meet. The long bolts were then replaced with appropriate length bolts and the weight blocks retightened around the pipe.

Although this met the Council's need for reduced risk regarding the number of joints and on going maintenance and monitoring, there was a significant down side regarding the time it took to complete each joint and the amount of diver time required during this process. A methodology that was successful in bringing the two pipe string faces together at the time of sinking would have been more efficient. MacDow is to be congratulated for recognising the Council's concerns and its tenacity in achieving the required outcome.

5.5 MICROTUNNEL- HDPE PIPE JOINT

5.5.1 FAILURE

In June 2009 to connect the microtunnel and marine pipelines, a 40 m HDPE length was installed after the MTBM was recovered. Then a shorter spool piece was inserted. An attempt was then made to install a "spade plate" to provide a seal between the tunnel and the HDPE pipeline to facilitate the tunnel hydrostatic test. The steel inter-jack pipe on the end of the microtunnel had to be extended towards the HDPE pipeline by a pair of hollow core jacks off the back of the flange at opposite sides of a tunnel pipe, connected to a common hydraulic manifold. While this was being done, a diver at the connection heard a bang. The joint between the first and second concrete pipes had opened indicating a probable failure of the carbon fibre straps at that tunnel joint. The carbon fibre straps were designed to provide fixity over the leading four pipes, to withstand tensile loads during the joining of the tunnel and HDPE pipelines. It was found that one of the pair of jacks was inoperative because of a faulty hydraulic connection causing a large eccentric load to be placed on the pipe joints. The tunnel was pressure tested but failed because the O ring at the displaced joint was pushed out of its recess and could not be corrected.

5.5.2 INTERNAL INSPECTION

MacDow fitted an external steel sealing/strengthening band around the displaced joint 1/2. A pressure test showed that a good seal was achieved, however there were concerns about the possible spalling of concrete from the leading edges of the concrete pipe. The tunnel was dewatered and a walk in inspection on 27 October 2009 found debonded carbon fibre strapping across two joints; between pipes 1 and 2 (the seaward most pipes) and between pipes 2 and 3. There was some indication of minor partial debonding at the joint between pipes 3 and 4.

5.5.3 REMEDIAL WORK

With the external sealing/strengthening band in place there was no need for the carbon fibre strapping at the same joint 1/2. Damage to the concrete pipe was made good with epoxy mortar and loose carbon fibre was removed. The gap between pipes 1 and 2 was filled with flexible polyurethane MC-Injekt 2300 NV. The gap between pipes 2 and 3 was filled with Sikadur UA epoxy to prevent any movement in the joint. An external steel strengthening band of the same design as that installed on joint 1/2 was placed on joint 2/3 to securely lock the joint from opening at the crown. This provided a tension and flexural capacity to joint 2/3 greater than required for service loads.

Eight stainless steel straps, 8mm thick and 140mm wide, were fitted across the pipe joints 2/3, 3/4 and 4/5, 2 at the spring lines and 2 close to the crown at joint 2/3 only. The stainless steel straps were to resist construction loads only and have no function after the completion of the connection. The tunnel passed the subsequent pressure tests.

This incident again shows the difficulty of working in the marine environment and the consequences in time and cost of a relatively minor equipment failure. MacDow demonstrated great commitment and resourcefulness in resolving this issue with an effective solution in short order.

5.6 SEA STATE CONDITIONS

During the investigations for the Ocean Outfall a great deal of data was gathered for preparation of consents from various sources including NIWA and through specialist consultants. NIWA was able to provide data extrapolated from a global wave model regarding swell period and wave heights in Pegasus Bay. The Water Research Laboratory (WRL) of the University of New South Wales in Sydney, undertook current measurement through deployment of an Acoustic Doppler Current Profiler (ADCP) for a year. Compilation of the data determined the wave height and frequency distribution in the area of the Ocean Outfall. This information was provided in the tender documents. The contract documents set out the basis for extension of times being granted, but at nil cost where delays were caused due to wave conditions being outside the parameters nominated by the contractor for various operations. The time risk was accepted by the Council. This meant that delays due to excessive sea states exceeding the nominated limits resulted in time extensions being granted, however costs associated with the delay were borne by the contractor. A wave buoy deployed adjacent to the work site by the contractor provided real time and continuous recorded output.

Unfortunately for both MacDow and the Council, significant delays were experienced due to the sea state, as nominated wave height limits were exceeded approximately 50% of the time on average. A storm with a 15 - 20 year return period also occurred in July 2008 which resulted in a one off wave height of 9.5 m, with a significant wave height of 5.5 m. These were considerably greater than the 1 year H_{max} = 6.0 m and H_s = 3.5 m wave heights expected in that part of Pegasus Bay. Over the period of the contract, adverse sea conditions resulted in extensions of time of more than 250 days. In addition, the resulting rework, mobilisation to site, restricted duration of weather windows, inaccurate forecasting and high winds influencing operational safety resulted in an additional ~180 days extension being agreed. The compounding effect of the slower jointing procedures using up good weather conditions added to overall project delay.

One of the effects of the high sea state causing stop / start productivity was the rework required to remove infill of the dredged trench and around dive work sites. The 2000T Machiavelli dredge produced a very accurate trench but this was then subject to natural erosion and infill. Its operation was restricted due to adverse sea states which occurred during the contract.

6 COMMISSIONING AND TESTING

6.1 PRESSURE TESTING

Pressure testing of the overall completed pipeline was carried out in three stages. Each element of the microtunnel and marine pipeline was pressure tested prior to installation as part of the QA contract requirements. However the pipeline itself needed to be tested in stages due to the two different construction methods and the material properties of each section.

The microtunnel was tested to 3.08 bar for exfiltration. After initial pipe saturation/soaking, the tunnel was pressurised by pumping in increments of 0.2 bar up to a maximum test pressure of 3.08 bar (relative to external hydrostatic pressure). If the test pressure dropped more than 5% for any load increment, then additional water was pumped into the pipe string and the volume of water required to re-establish the test pressure was measured.

A successful test corresponded to either net exfiltration at maximum test pressure of not more than 0.1 litres per minute per joint or a sustained test pressure.

The HDPE marine pipeline was tested to 3.0 bar. After initial pipe saturation/soaking pressurisation of the pipeline above the "soak" pressure was carried out. The pressure was raised steadily and smoothly to the test pressure. The system test pressure was maintained by pumping at 5-minute intervals for a period of 30-40 minutes. The amount of make up water required to maintain the pipeline pressure was measured and later compared to a calculated maximum value. The pressure was monitored for at least 60 minutes following the above test period.

The final test involved pressurising the complete pipeline with the main purpose to check the integrity of the joint between the microtunnel and the marine pipeline. This was achieved by divers observing/inspecting the MT/PE joint for leakage during pressurisation of the pipeline to 3.0 bar.

Subsequent commissioning tests for the pipeline involved:

- pipeline flushing to remove sediment from the pipeline invert and pump testing;
- gravity flow testing;
- control system checks;
- pumped flow test; and
- transient pressure check;

6.2 DISPERSION TESTING

WRL was commissioned by Council to undertake field investigations to validate the diffuser performance of the outfall. These field studies, undertaken over three days from the 19th to 21st April 2010, comprised a dive inspection of the operating outfall and two dilution tests at the average design flow of 2.3 m^3/s.

The dilution tests were undertaken to measure the rates of mixing of effluent into seawater being achieved immediately above the diffuser and in the region up to 200 m away. This was achieved by injecting Rhodamine WT fluorescent dye into the effluent discharge and measuring the concentrations in the field using a boat and fluorometer. The number of dilutions achieved was calculated as the ratio between the concentration in the pipe to that measured in the ocean.

Weather conditions were extremely favourable and very similar during both dilution tests. Currents as measured were negligible (below 0.02 m/s) and winds were very mild. The outfall plume could be discerned as slowly moving northwards, although oranges (used as drogues) thrown into the water moved less than 400 m during the three hours on site each day.

The required performance of the diffuser was to achieve an average minimum dilution of 61 times as presented in the resource consent hearings. This dilution represents the expected performance after reasonable mixing within the near-field. Mixing in buoyant plumes can be described as having a bell shaped Gaussian distribution, the centre of which is as predicted by near-field models. When discussing the performance of a diffuser it is important to compare the test data to the comparative design performance data, which is the mid point of that distribution.

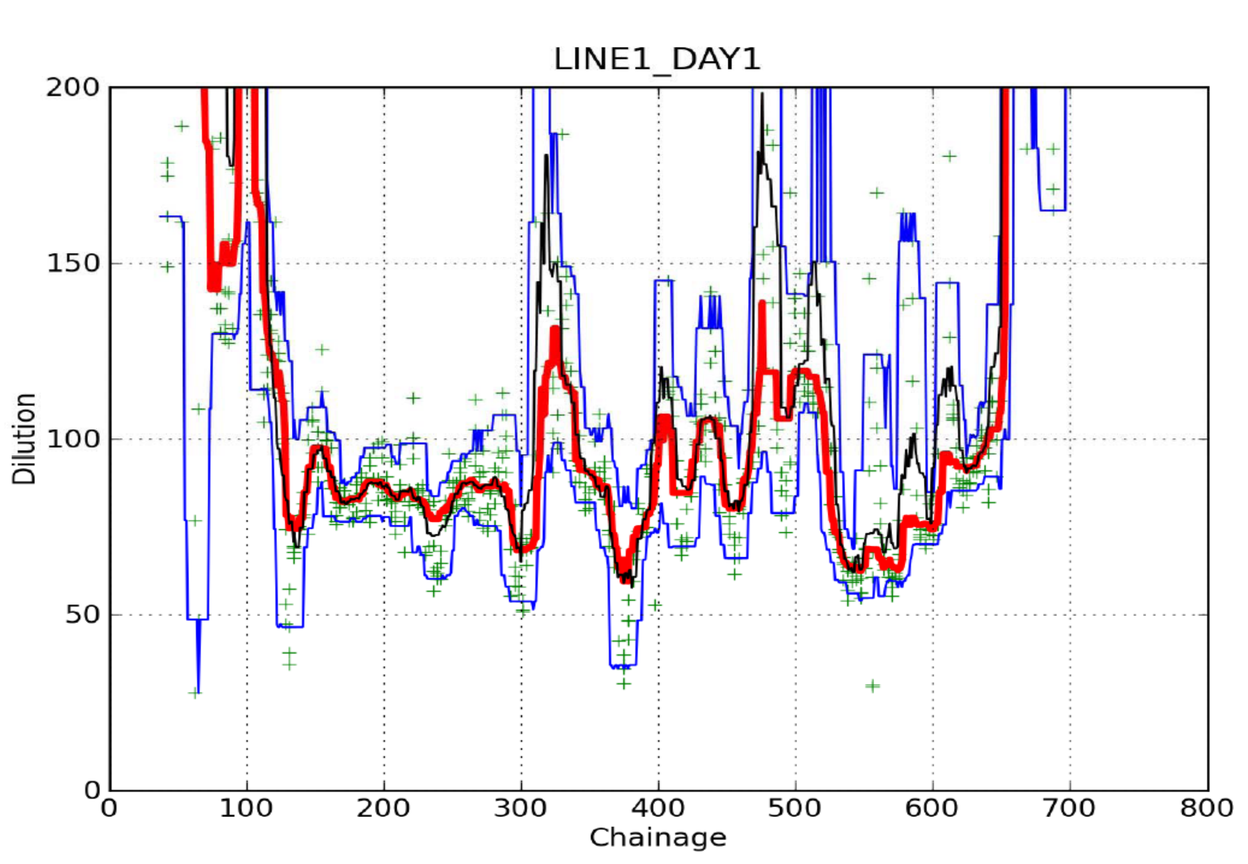
It was found that the dilution at the end of the near-field as represented by the median of all samples was above 61 times on both days. Table 1 presents the statistics of all data between Chainages 0 m and 360 m on Day 1 and Day 2.

Table 1: Dilution Statistics between Chainages 0 – 360m

	Median	10th %ile	90th %ile
Diffuser Line Day 1	78.78	151.08	57.84
Diffuser Line Day 2	70.1	153.95	53.99

Six separate transects were sampled away from the centreline of the diffusers. Line 1 represents a distance 200 m to the north of the diffuser (at the edge of the mixing zone). On the first day of testing, the dilutions as measured along that transect are shown on Figure 7 following. This shows that while individual measurements show dilutions less than 61 times, the median of the dilutions along the diffuser is greater than the required minimum of 61 times.

Figure 7: Measured dilutions on Line 1 – Day 1



Dilution tests were performed during the design quiescent conditions, which are the conditions for the worst outfall performance. The outfall diffuser was found to be performing as designed and provided dilutions in the ocean directly above the diffuser greater than the target dilution of 61 times.

A dive inspection showed the diffuser operating as expected.

7 CONCLUSIONS

The Christchurch Ocean Outfall Project has been a tremendous success, the origins of which go back to 1998 when the Council started looking at long term solutions for Christchurch's wastewater discharge. During the design and construction phase, there have been many challenges, culminating in the project being delivered approximately 18 months later than desired. The main reason for this is weather and sea state conditions that made progress difficult and dangerous. There have been many lessons learned through this project, both in terms of what worked very well, and what could have been done better.

The procurement process involving pre-selection of tenderers, an Interactive Tender Process and early consideration to project risks worked well. The Geotechnical Baseline Report and contract conditions around weather and sea state conditions provided clarity for all parties to assess extension of time and differing physical conditions claims. Risks were clearly assigned to the best party to either carry or manage that risk.

There were a number of aspects of the construction that proved to be extremely difficult, frustrating or dangerous. The connection between the microtunnel pipe and the HCPE pipe was always going to be difficult. MacDow had construction methodologies and procedures to undertake a wet recovery of the MTBM, however failure of a jacking system put the pipeline and construction programme at extreme risk. The tenacity of the contractors and dive teams must be applauded, as must the worker who undertook the remedial works from the inside of the pipe once it was dewatered.

The Health and Safety record for the project was disappointing. Undertaking construction in a marine environment is extremely risky. Delays due to weather and sea state prolong the exposure to these risks. The ability to control the work environment is limited in the ocean. Even with excellent polices, procedures and practices, the project did not achieve one of its fundamental milestones of zero lost time injuries.

Predicted sea state conditions prior to award were not in a form that provided a clear indication of the length and frequency of working windows. This would have aided in selecting construction procedures and risk mitigation measures that maximised working time and opportunities.

Throughout the project, the Partnering Relationship established between the Council, MacDow and the professional advisors enabled issues to be resolved without call on the disputes procedures. This is a project that has delivered an outstanding asset for the people of Christchurch City.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the efforts of many people who have contributed to the successful delivery of the outfall project. The team from McConnell Dowell Constructors Ltd deserve special mention for their commitment and resourcefulness during a long and difficult project. Thanks also go to the Christchurch City Council, URS New Zealand Ltd, OCEL Consultants, Powell Fenwick, Downer EDI and many other sub-contractors who played meaningful roles in this project.

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