AUCKLAND’S BIG PUSH

E. AYRE¹ AND D. COOK²

¹ McConnell Dowell Constructors LTD
² Firth Humes Group

ABSTRACT

The Artillery Drive Storm Water Tunnel is a 1km long 2.5m internal diameter stormwater pipeline enabling large scale housing development in the South of Auckland. This paper provides an overview of the project, reasoning for the alignment and microtunnelling machine selection, the development of temporary works, groundwater management and its effect on temporary works design, jacking load prediction versus reality, the application of ASNZS 4058:2007 in the design of VT cast jacking, interaction with critical infrastructure and engagement with stakeholders. Alignment selection and effective communication with affected parties played a major role in overcoming the challenges presented and delivering a successfully project for the client, constructor and the community.

INTRODUCTION

The Artillery Drive Stormwater Tunnel is the first of three Auckland Council projects that will enable the development of over 15,000 new homes at Takanini, South Auckland, see figure 1 below.

Currently the land designated for development is within a flood plain, and in order to minimise the flood hazards, the water must be allowed to drain from the area.

This paper covers the construction of the Artillery Drive Stormwater Tunnel, a 1.1 km long and 2.5 m internal diameter pipe installed utilizing mainly pipe jacking.
PROJECT OVERVIEW

Project History

The Artillery Drive Stormwater Project was originally recommended in the Papakura District Council Stormwater Catchment Management Plan in 2004, but never went to construction due to lack of funding. Following the amalgamation of several legacy councils and the formation of Auckland Council this project was re-established as a key project to enable growth. In 2013 Sinclair Knight Merz, now Jacobs, was engaged to carry out the design and in December 2015 McConnell Dowell was awarded the physical works following a competitive tendering process.

Pipe alignment selection

The scheme design required a pipe capable of transferring the 1 in 100 year rainfall event (23.7 m³/s) from the Upper McLennan Park Pond to the Pahurehure inlet. The Upper McLennan Park Pond is a stormwater treatment devise but also acts as a stormwater detention area which provides flood protection to the residential and commercial areas south of its dam structure.

The proposed stormwater pipeline had to cross the North Island Main Trunk (NIMT) railway line, the only rail service between the two largest cities in New Zealand (Auckland and the Capital Wellington) and a critical local public transport system.
The alignments that were considered reflected on one hand the need for cost savings (most direct route, options 1 and 3 in figure 2), on the other hand the desire to provide a solution that was easily consented (where possible in publicly owned land, option 2).

As installing the pipe within private land could have resulted in long lasting legal challenges, a decision was made by Auckland Council to select an alignment that would be fully within publicly owned land (road reserves, parks and a private property purchased to facilitate the works).

**Equipment selection**

The tender required to use of a slurry Tunnel Boring Machine (TBM) with a tunnelling head capable of excavating the Baseline Ground Conditions, as defined in the Geotechnical Baseline Report. The decision to specify a slurry TBM was made during the preliminary design phase and was influenced by the following:

i. A multitude of services (some in close proximity and / or critical) which had to be crossed or were running parallel to the proposed alignment.

ii. The close proximity of buildings and structures, some of which are susceptible to settlement.

iii. Varying ground conditions (East Coast Bays Formation Rock, East Coast Bays Formation Soil, Alluvium with bands of Gravel and Sand and the risk of running Sands.

iv. A paper summarising the findings of several tunnelling projects in Singapore, comparing the resulting settlements of Earth Pressure Balancing and Slurry TBM’s, suggesting a lower risk of settlement in Alluvium when utilising slurry TBM’s.

Whilst many factors contribute to the amount of settlement resulting from tunnelling, Auckland Council was willing to pay a premium to reduce the risk of damage to buildings, structures and infrastructure.
With hindsight the decision to specify a slurry TBM was critical to the success of this project as unexpected high groundwater flows were encountered within the East Coast Bays Formation Rock.

CONSTRUCTION PHASE

Concrete Pipe Design

With the pipe discharging into the marine environment, any exposed metalwork within the pipeline would need to be Grade 316L stainless steel. Due to the cost of the stainless steel the pipe design inevitably moved to a concrete in-wall joint with a rolling ring rubber seal. The concrete mix design incorporated 8% microsilica and reinforcement covers were those associated with marine grade pipes. The profile of the spigot end of the pipe presented some challenges in getting an adaptor ring design that could be economically fabricated and installed onto the back of the TBM. Instead an innovative solution of a dedicated concrete “pipe 0” was built that would fit the back of the TBM and adapt to the in-wall joint. This also allowed for a standard fitout of this pipe to suit the services in and out of the TBM. Pipe 0 was then reused for all the subsequent drives.

The pipes used for the Artillery drive jacking project were manufactured using VT pipe technology. This method allows for the manufacture of numerous pipes each day allowing for shorter delivery times. The method also allowed great accuracy in the internal dimension of the pipe and the wall thickness. Accuracy of wall thickness is important when considering the pipe jacking capacity. The pipe length for the project was selected at a non-traditional length of 3m, which reduces the number of joints in the pipeline and improves straight line jacking capacity. The selection of the non-standard pipe length involved the procurement of new internal and external moulds, complete with pallets and head rings. To achieve the tight programme requirements the moulds were manufactured in two factories, one being the USA and the other in Europe, and shipped to NZ.

Successful pipe manufacture involves very tight tolerances, so considerable care is required when manufacturing components from different locations. It was with considerable relief when all components arrived at the Humes Papakura factory and fitted together perfectly.

The manufacture of concrete pipes is described in AS/NZS4058. A pipe design is proven by type testing which involves load testing a minimum of 4 pipes to destruction (ultimate load) and 8 pipes to the proof crack load. The test load requirements for the 2500 internal diameter pipes is 161KN/m for the ultimate load and 242KN/m for proof load. One challenge for the Artillery Drive project is that final testing could not commence until the new 3m long moulds and manufacturing equipment arrived. Humes did however have 2.5 long moulds for the required diameter. This allowed some preparatory work of refining the concrete filling rate, vibration amplitude and frequency to achieve optimal compaction. Two pipes were manufactured and load tested to provide some comfort that the design would achieve the required load class before the final mould configuration arrived. When the moulds arrived and pipes were manufactured for types testing, the achieved loads comfortably exceed those required by AS/NZS4058.

From the expected ground conditions jacking load studies were undertaken to predict what the potential jacking loads would be, determine a pipe capacity and projected number of interjacks. Ultimately this developed a criteria for a 1000T capacity jacking pipe.

The jacking capacity of a concrete pipe is often described in terms of the straight push capacity of the pipe. As the pipe deflects the stress at the joint no longer remain uniform with a concentration of stresses occurring at the compression edge. To avoid failure, with due
consideration for fatigue, it is therefore necessary to define the load verses joint deflection capability of the pipe. Figure 3 illustrates a typical load joint deflection relationship. Most references are silent on how the squareness tolerances are incorporated into the load deflection relationship. For a pipe of this size AS/NZS4058 allows a maximum out of squareness of 7mm. This presented two challenges, how to measure these very tight tolerances accurately, and also how these measurements should be incorporated into the design load deflection recommendations.

Several option were examined for measuring squareness. Traditional methods of using a square were not practical given the size of the pipes (3m outside diameter) and the accuracy needed. The weight of the pipes (16 tonnes) also presented a challenge. The final innovative solutions was to build a dedicated testing station, comprising of a thick steel plate which was levelled to very fine tolerances and grouted in level at the casting factory. This formed the base of the testing rig. The 16 tonne pipes where stood on the socket end. As the socket end was at the bottom of the mould this was considered to be the true end. Laser levels were used to calculate the level of the two faces of the in-wall joint at 6 assigned locations around the circumference. In addition lasers were used to calculate the verticality of the pipe. These measurement then being input into a spreadsheet which converted the data into measurements of end squareness. The process proved to be able to measure the end squareness to fractions on a mm accuracy and was repeatable. It also provided data for each individual pipe which if preferable to a simple pass/fail test methodology.

For this project Humes were confident they could achieve end squareness values less than prescribed in AS/NZS 4058. This proved to be the case, though was very challenging. The advantage of improving the end squareness is that it allows for greater load at any given joint deflection. Exactly how the allowable joint tolerances should be incorporated into the pipe load deflection performance was the subject of considerable literature review. Most references were silent on the topic and there were opinions which turn out to be wrong. To evaluate the issue, a first principles approach was adopted, a spreadsheet developed, and end squareness tolerances changed to evaluate the consequences. The conclusion being that squareness tolerances could not simply be ignored by assuming they were accommodated by squashing of the packer, and these tolerances needed to be considered in the load/deflection calculations.

When a spigot is inserted into a socket of a pipe the seal ring is compressed. The compression of the ring is important as it determines the pressure rating (internal and external) of the joint. With jacking pipes consideration has to be made of the external pressures relating to the grouting process. To achieve the required seal compressions requires very tight tolerances of
the mating faces of spigot and socket. Normally the dimension of spigots and sockets are measured using go, no/go gauges, and swing gauges. Although a good quality control mechanism, the documentation only consists of a pass/fail. For this project the decision was made to measure and record the spigot and socket tolerances using a specifically procured digital vernier. This allowed the accurate measurement of the pipe along 3 sections and provided quantitative data. It allowed the allocation of tolerances to be assigned appropriately to either the spigot or socket based upon the review of manufacturing records while ensuring the require seal compressions were achieved.

The weight of the pipes provided some manufacturing challenges. A forklift capable of moving these very large pipes was secured for the project. A special curing slab was constructed for the project. This slab was sufficient thick to allow the passage of the very large forklift, and recycled the water to cure the pipes for seven days.

**TBM and Tunneling Equipment**

**TBM**

The TBM used for the project was a Herrenknecht AVN2500. This TBM had previously been used by McConnell Dowell for a project overseas where it was up skinned to 3.4m, so with a 315kw main drive it had ample torque capability for the expected conditions. This was matched to a soft ground cutter head (fig 4 below) as the ground conditions in the Geotechnical Baseline Report indicated an upper bound rock strength of 13.5 MPa.

![TBM Launch](image)

**Figure 4 - TBM Launch**

On surface, the control cabin was the central point for TBM operations, also controlling the operation of the automatic bentonite lubrication system. Positional control of the machine was achieved through the use of the Herrenknecht MWD II fibre optic gyro theodolite, and a Hydrostatic water level system.

The setup required only minor changes to meet the New Zealand regulations, with the oil being changed to a high flashpoint low flammability type, and an additional gas monitoring system was installed in the machine.
Separation Plant

To separate the spoil a Herrenknecht desander was procured with a nominal flow capacity of 500m$^3$ per hour, and an S4 centrifuge from Piggot Shaft Drilling to aid with fines separation.

The local geology consisted mainly of an interbedded sandstone and mudstone (both intact and weathered), where the individual grains in the sandstone were themselves mudstone. XRD analysis had identified total clay contents of up to 60% geologically long sections is shown in fig 5. It was identified that a long chain PHPA polymer would be the best material to aid the flocculation and a close relationship was developed with a local chemicals manufacturer to fine tune the molecular weight and charge of the polymer to best suit the local conditions and processes.

By choosing to start in the middle of the four drives and work towards the ends, the separation plant would not have to be moved during the durations of the project, only requiring 2 additional booster pumps to send the slurry back through the completed tunnels to the plant. With this arrangement a much smaller tunnelling footprint was achieved at the remote tunnelling sites.

Power Supply

The requirements around design and implementation of power supplies have evolved significantly under the new regulations with the requirement for a certified design to be undertaken and an Electrical Engineering Principle control plan prepared.

The local lines authority was engaged to supply and install a new substation adjacent to the main site compound so as to provide 1.5MVa at 400 volts directly to the main site switchboard. This removed the need to have any high voltage equipment and the safety concerns that go with those kinds of installations. Next a local firm was engaged to undertake an earthing study, looking at the power and other site installations to determine and protect against rises in earth potential and minimise any step or touch voltage risks. In conjunction an electrical design firm was contracted to provide the site electrical design, including analysis of installed equipment, cable sizing and arrangements, and protection analysis.
Shaft Design

From the geotechnical information available the main jacking shafts were to be sunk mostly in alluvial and residual soils, with the base of two of the 3 drive shafts founded in rock. The base design was around ground support in the form of sheet piles or Deep Soil Mix (DSM) piles to the rock head, then excavation into the rock with spot bolting and mesh where required.

During the excavation of the first shaft, ground water was seen piping up the sheet pile clutches early in the excavation, with the volume increasing as the depth increased. Eventually this water started to push through the remaining silts and clays in the bottom of the shaft, and with the excavator working in the material significant challenges were created around ensuring the water discharge quality into the local storm water network was acceptable. Once down to formation level in the rock, the water inflow rate was far higher than would have been expected for the typical conditions in the ECBF and had become regionally significant at 8l/s, impacting on groundwater monitoring piezometers up to 600m away.

The shaft design was modified to allow for anchoring of the base slab and fully sealing the excavation so as to allow the deep groundwater levels to recover. All of the boreholes for the project were monitoring the deep groundwater in the rock, and we were seeing significant continuity across the project footprint. To better assess the impacts, a number of the deep boreholes had additional shallower piezometers installed to allow the water in the alluvial material to be monitored. In the alluvial material only minor variation was seen, this was attributable to weather effects (dry periods and rainfall) as opposed to what the deeper ground water levels were doing.

The arrangement for the deepest shaft was originally going to be DSM piles down to bedrock, then bolts and mesh down into the rock, however the improved understanding of the area hydrological conditions indicated that this would have an unacceptable effect on groundwater drawdown considering how long this shaft was intended to be open for. To find an acceptable solution, Auckland Council and McConnell Dowell, along with an independent designer and hydrogeologist collaboratively developed a range of possible solutions, concentrating on developing a robust low risk solution that was acceptable to all parties.

The final solution was a secant pile arrangement, with the piles extending 6m past the base slab level. This had the advantage of controlling the groundwater effectively, had very low uncertainty in the cost to implement, and also was the quickest to install and progress minimising the impact on the project programme.

Tunneling Activities

The tunnelling operation ran smoothly, with advance rates between 100 and 180mm/min in the alluvial and residual soils. A maximum of 7 pipes were installed in a 10 hours shift, with the project running dayshift only due to the location within a residential area and the construction noise limit standards. The use of the VT pipe manufacturing technology meant that pipe supply could keep up with the pace of installation.

Jacking loads

With the early drives being relatively short, some observational effects in differing lubrication regimes was seen. The first drive, 160m long, was driven primarily through sandy clays and sandy silts. While below the groundwater table, this material was known to be relatively stable. No bentonite was applied to the annulus during this drive, jacking loads increased relatively linearly with length, albeit with significant peaks for breakout. TBM broke through into the reception shaft with jacking loads in the region of 350T, see drive 1 on figure 6 below
Due to the way the machine broke in, the water in the annulus of the tunnel largely drained out. This loss of annular fluid caused the jacking loads to increase to 600T by the time the machine and pipe zero were fully recovered circa 36 hours later.

During the second drive, 200m long, a short flow able fill breakthrough eye was added into the reception eye to enable the machine to mine much closer to the breakthrough point without loss of slurry circulation. The geology of the drive was similar to drive 1 for the 1st half of the drive, then moving into the alluvial for the remainder. At about 80m into the drive the jacking loads started increasing rapidly, and bentonite lubrication was applied to the annulus from this point. It can be seen from the graph below that the bentonite did eventually control the jacking loads and start to reduce them.

For Drive 3, it was known that we would clip the alluvial material early in the drive, so bentonite was applied to the annulus right from the start. This resulted in a much slower increase in jacking loads, and very little difference between breakout load and mining load during the drive.

![Figure 6 – Jacking Loads vs. Chainage](image)

**Crossing of North Island Main Trunk Railway Line**

For the crossing of the NIMT rail line, advance meetings were held with the rail operator to explain the methodology, how it would react to the ground conditions and the monitoring regimes that would be put into place to check for deformation of the line as we tunnelled underneath. A series of monitoring arrays were established on the rails themselves and at prescribed offsets as shown in figure 7 below.
To minimise impact to the rail line these points were setup so that they could all be monitored from outside the rail corridor. A trigger action response plan was developed and distributed to the parties involved to ensure that the lines of communication were well understood during the process. As the TBM passed through the area the points were monitored after every 3m of advance. There was no measurable movement of the monitoring points during the rail crossing by the TBM, and hence no impact on train movements.

LESSONS LEARNT

The contract requirement to utilize a slurry TBM – Auckland Council’s approach in general is to promote innovation by providing flexibility in its Contracts. However in this particular case Auckland Council saw a need to de-risk the project by specifying the type of TBM to be used. A decision that proved vital to the success of the project. Clients and their advisors should not hesitate taking the lead in decisions which are considered critical to the success of a project.

High groundwater inflow – Groundwater flows in East Coast Bays Formation rock are limited to fractures and in general these fractures are “thin” and “heal” over time. The rock encountered in this project varied in strength from 1 to 18 MPa and the fractures in the harder rock appeared to be well developed, connected over a wide area and carry unexpected high volumes of flow. Whilst the geotechnical investigation identified fractured rock and unusual high permeability it was not clearly identified as a risk, resulting in significant additional cost to the project. East Coast Bays Formation rock exceeding 5 to 10 MPa should be investigated with care if excavations well below the groundwater table are required.

Jacking pipe with in-wall joint – Whilst a concrete in-wall joint pipe is less forgiving when it comes to sharp deviations in the alignment (potential damage to the pipe) it provides a very cost effective and long term suitable solution in aggressive environments. Sharp alignment deviations can be avoided with the appropriate equipment selection and regular control surveys of the alignment.

Changes to temporary works to reflect unforeseen ground conditions – Whilst the decision to change the temporary works design part way through the contract resulted in significant project delays and associated cost, it proved to be an overall best for project solution by preventing potentially large-scale property and infrastructure damage as well as delays triggered by exceeding consent conditions.
CONCLUSION

Delivering effective infrastructure is critical in the development of a growing city such as Auckland. Trenchless solutions profile a low impact means to install and upgrade infrastructure within a built up urban environment. Critical to the success of these project is good planning, a robust approach to risk management and health and safety and effective communication and collaboration between the Client, Constructor and pipe supplier, to ensure the inevitable challenges encountered are met and dealt with appropriately to produce the best solution for all involved.