HUNUA 4 WATERMAIN TRENCHLESS CROSSING OF STATE HIGHWAY 1

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ABSTRACT

To cater for future growth in Auckland Watercare Services Ltd is constructing the Hunua 4 Watermain. The watermain runs for 28 kilometers from South to Central Auckland. One of the highest risk crossings in the alignment is a crossing under the Southern Motorway (on SH1), New Zealand’s busiest road.

GHD and CH2MBeca (CBG) have delivered the design and monitoring as well as a quality role within a NEC3 contract framework. As the designer of the permanent works, CBG provided the preliminary watermain alignment, and geotechnical investigation. The final vertical alignment and temporary works design including tunnelling design were undertaken by the main contractor, the Fulton Hogan John Holland Joint Venture (FHJHJV) and its subcontractor Bothar Boring Ltd.

The ground conditions, high groundwater, the proximity of buried and above ground services, and SH1 combined to make this a high risk crossing.

One of the greatest risks to utilising trenchless technology is machine and equipment selection. The design process should always be collaborative between designers, geotechnical specialists and specialist contractors. An integrated planning and design approach was undertaken and minimised the risks for this crossing although some issues still arose during construction.

This paper details the planning process, outcomes and the lessons learned.

KEYWORDS

Trenchless, tunnel boring, large diameter pipeline, risk management, settlement monitoring, watermain.

1 INTRODUCTION

1.1 THE HUNUA 4 PROJECT

To cater for future growth in Auckland Watercare Services Ltd (Watercare) who manages Auckland’s water and wastewater, is currently constructing the Hunua 4 Watermain project (Hunua 4). The pipeline is a 1600 mm and 1900 mm diameter concrete lined steel (CLS) main that runs for 28 kilometers from Redoubt North Reservoir in Manukau Heights to Campbell Crescent in Epsom, Central Auckland.

The Hunua 4 pipeline is part of a larger system that supplies raw water from the Hunua Ranges and Waikato River sources to Ardmore water treatment plant, which in turn supplies approximately 70% of Auckland’s water demand. When completed Hunua 4 will work in parallel with two existing pipelines that deliver treated water from Redoubt Reservoir to Auckland City. The two existing pipelines are meeting the city’s current demand but the new pipeline is needed to cater for future growth and ensure security of supply.

Prior to the current works, Watercare teamed up with three other organisations to incorporate Hunua 4 into their projects. A 1.5 km section of Hunua 4 was installed as part of New Zealand Transport Agency's (NZTA) Manukau Harbour crossing, 150 m crossing of the North Island Main Trunk Railway was included as part of Kiwirail's urban rail upgrade and another 200 m section was included along the road frontage of a new residential subdivision. These enabling works were all constructed during already planned works netting large savings in disruption for the residents of South Auckland, and avoiding the need to create additional crossings unnecessarily.

Unlike the three enabling works projects described above, the Hunua 4 crossing of State Highway 1 (SH1) was built as part of the main construction contract by a trenchless construction method.

1.2 PARTIES INVOLVED

The Hunua 4 construction contract incorporates the NEC3 ECC conditions of contract. The parties to the contract are Watercare (Employer) and the Fulton Hogan John Holland Joint Venture (Contractor).

The trenchless section was installed by the FHJHJV's Australian sub-contractor, Bothar Boring Ltd. They provided all tunnelling equipment and oversaw the production of the reinforced concrete jacking pipes. Fulton Hogan Ltd designed and installed the launch and reception shafts to the design depths and dimensions provided by Bothar Boring Ltd.

1.3 HUNUA 4 ROUTE ALIGNMENT SELECTION

In South Auckland, Hunua 4 is designed to run westwards away from the other two major treated water pipeline routes to provide improved security of supply to areas of the city that border the Manukau Harbour, including Auckland Airport.

A second criterion for the pipeline routing was that Watercare preferred to route the pipeline along publicly owned road corridors in preference to private land. As a result of those criteria, the pipeline heads west and crosses SH1 near the Reagan Road highway overbridge (Figure 1).

Figure 2 shows plan and longitudinal section views of the trenchless crossing of SH1.

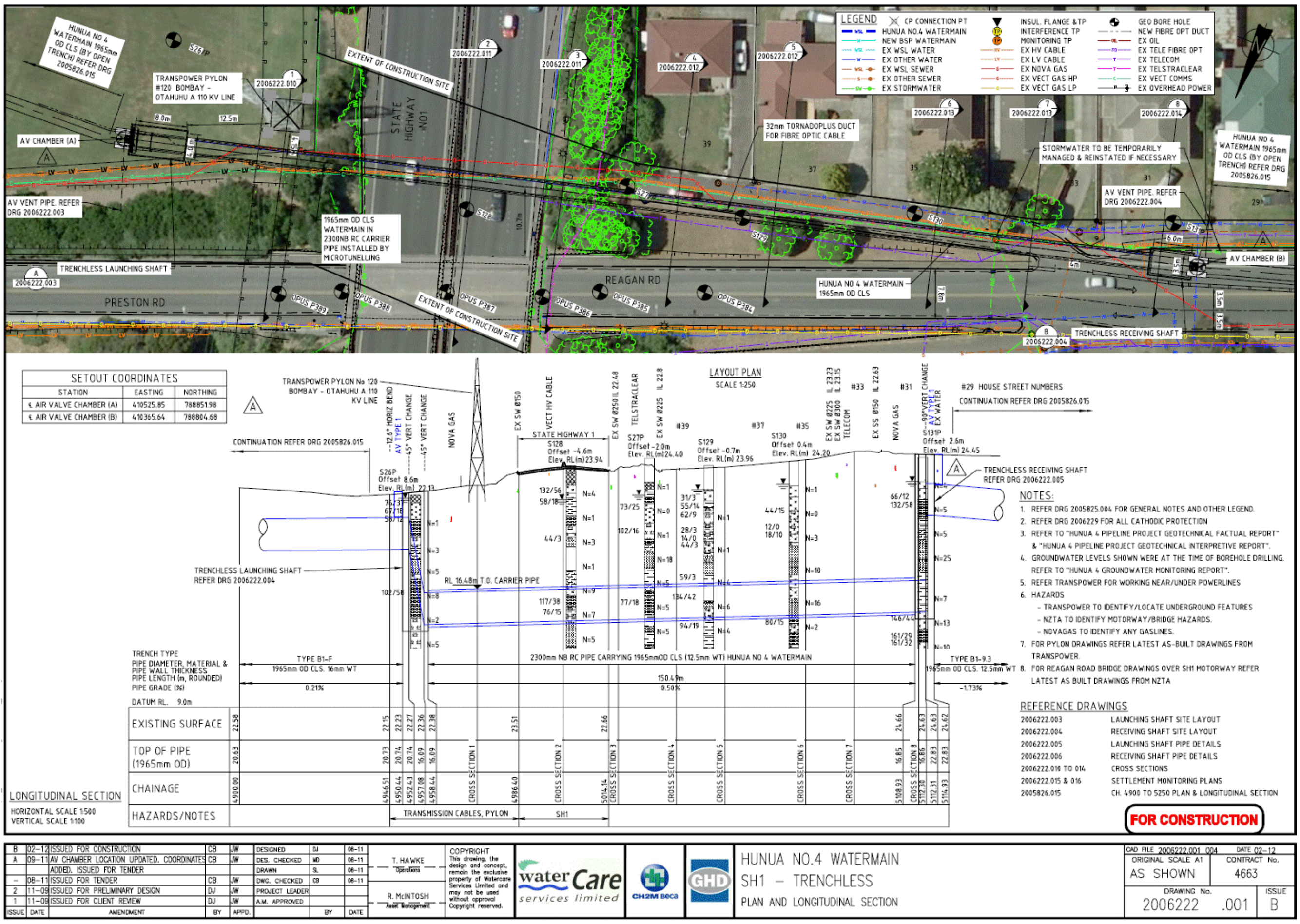
At this scale it can be seen that the trenchless section was aligned to be launched from the Manukau Sports Bowl parkland on the east side of SH1 passing under the highway and beneath a publicly owned right of way next to the west abutment of the Reagan Road overbridge, finishing in a reception shaft in Reagan Road.

The east end of the alignment is constricted between the east abutment of the Reagan Road overbridge and a Transpower pylon. At the west end the constrictions are the traffic on Reagan Road, the houses to the south of the road and numerous existing services.

The objective of the SH1 trenchless crossing was to complete the approximately 150 m long drive whilst providing no adverse effect on traffic, Transpower's Pylon or other services, and no effect on the overbridge or residential houses nearby.

Figure 1: SH1 Crossing Location

Figure 2: Hunua 4 SH1 Crossing Section



2 SETTING

2.1 TOPOGRAPHY

The alignment of the watermain runs from Manukau Heights west towards Auckland airport. Generally the pipe starts at around 115 m RL at Redoubt Reservoir, and follows existing roadways at a very gradually decreasing grade toward the airport, crossing under State Highway 1 at around 16 -18 m elevation above mean sea level. The pipe alignment continues on towards the airport then heads north crossing the Manukau Harbour then following urban streets north, rising rapidly to 90 m RL. The alignment traverses around Cornwall Park and terminates at Campbell Road, Onehunga at around 80 m RL.

Generally the ground level rises from around RL 22 m at the launch shaft of the SH1 crossing, on the Manukau sports bowl side, to 24 m at the retrieval shaft. However the Regan Road overbridge is founded on a 5 m high embankment parallel and immediately adjacent to the alignment.

2.2 GEOLOGY

The published geological history of the area is that of a low lying fluvial, lacustrine, and esturine deposition area, into which erosional and sporadic volcanic material were deposited to form the Puketoka Formation. Puketoka Formation contains silt/clay, sand and gravel with peats and rhyolite volcanic soils (Edbrooke, 2001).

There are significant peat / organic clay deposits in this area of South Auckland that contain lenses of sand and volcanic derived silts. These ground conditions are particularly difficult for trenchless crossings as they can contain wood and stumps which can block TBM advance, contain ground sensitive to disturbance that can flow with vibration resulting in volume loss at the face of the TBM. Additionally the peat layers in this ground can be highly compressible during dewatering resulting in significant surface settlements.

2.3 GROUND INVESTIGATIONS

Six historic machine boreholes, conducted in 1984 for the construction of the overbridge, were retrieved from archive and incorporated into the ground model. Six new machine boreholes to 15 m depth were drilled along the trenchless route with piezometers where possible and laboratory analysis undertaken to determine the soil properties .Figure 2 presents the location of the historic and recent investigations. The ground conditions encountered were in general accordance with the published geology (Edbrooke, 2001).

In total nine piezometers were installed on the trenchless alignment and adjacent to it, to determine groundwater profile and to monitor drawdown effects.

2.4 CONDITIONS ENCOUNTERED

Fill was encountered, and has been placed on the western side of the site generally around 0.5 m thickness, and up to 5 meters height for the overbridge approach embankments. Puketoka Formation alluvium consisting of weak dilatant silts, organic clays, peat and sands (Table 1).was found underlying the fill.

Table 1: Geotechnical Properties of Puketoka Formation Encountered

Unit	Puketoka Formation
Soil Types	Clays with varying silt and sand content, silt with minor clay, and peat. The colour of the soils ranged from pale to dark brown and (greenish) grey to black. Fine to coarse grained, poorly graded sand, with some to minor silt and clay content, dilatant. Green/white and grey.
Soil Condition	Ranging from soft to hard. Clays were generally highly plastic but occasionally slightly to moderately plastic. The silts were generally non to slightly plastic, but occasionally moderately plastic. Sands loose to medium dense

	The shear vane readings taken ranged from 12/0 kPa to 190/37 kPa and the SPT N values ranged from 1 to 50+.

2.4.1 GROUNDWATER

Generally groundwater was found to be around 2 m below the existing surface. Groundwater was monitored for around 12 months before construction and during the works. Permeability of the geologic units was tested where applicable, and assumed with known correlation to other projects in similar ground. The adopted permeability values were 1 x 10^{-7} m/s for Puketoka Formation Clay and Silt, and for the silty Sand. The later unit varies considerably depending on sand content, and when more sandy can be considerably more permeable. The Puketoka Formation Peat/Organic Silt had an adopted permeability of 1 x 10^{-9} m/s and was determined to have high initial water content (65%) in laboratory testing. The high water content of the organic material and the presence of high permeability sand lenses cane result in groundwater drawdown induced settlement.

2.5 EXISTING SERVICES

The position of existing services was a difficult constraint on the crossing vertical and horizontal alignment. Services immediately in the vicinity included a gas main, high voltage and low voltage underground, overhead power transmission cables and pylon, communication cables, stormwater, water and sanitary sewer. Additionally the overbridge abutments are piled. Figures 3 and 4 show the crossing alignment and proximity to services.

Figure 3: SH1 Crossing Services Section 1 – proximity to pylon foundation

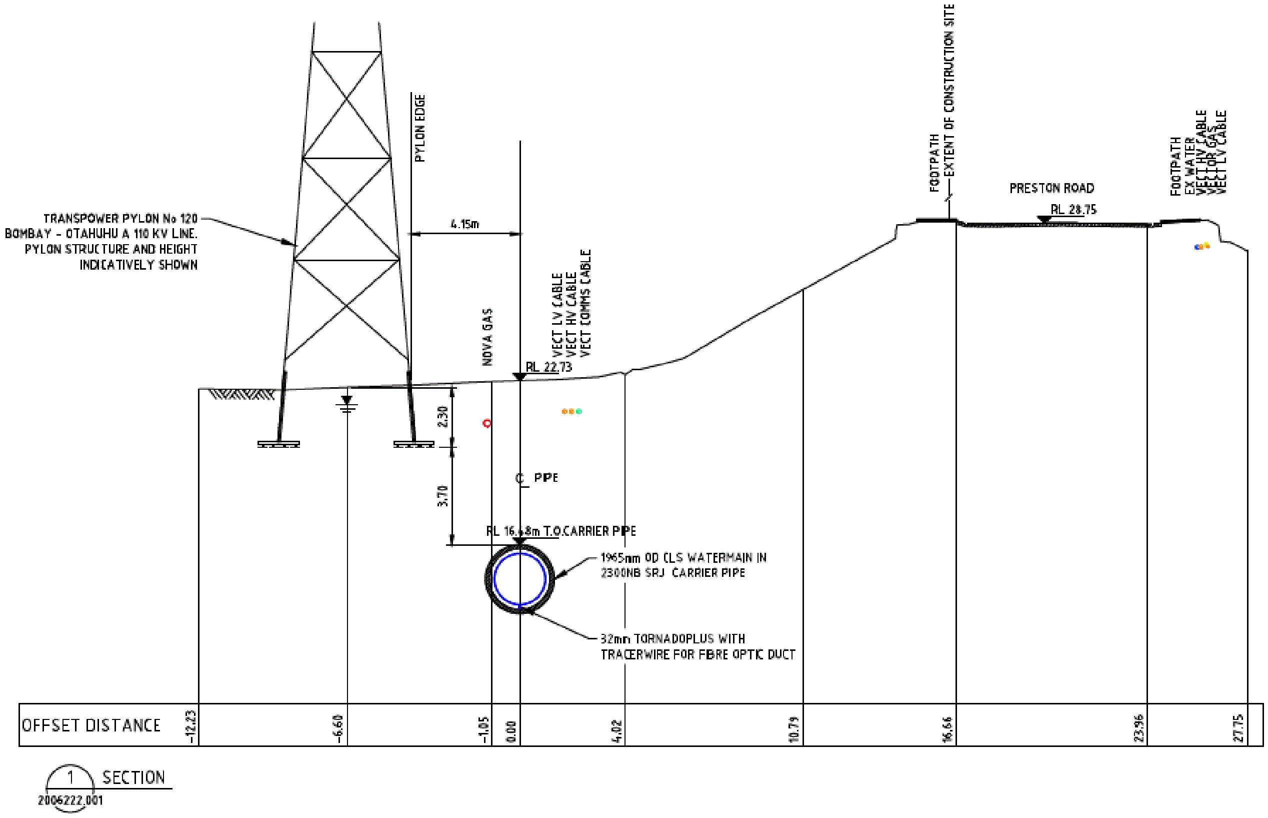
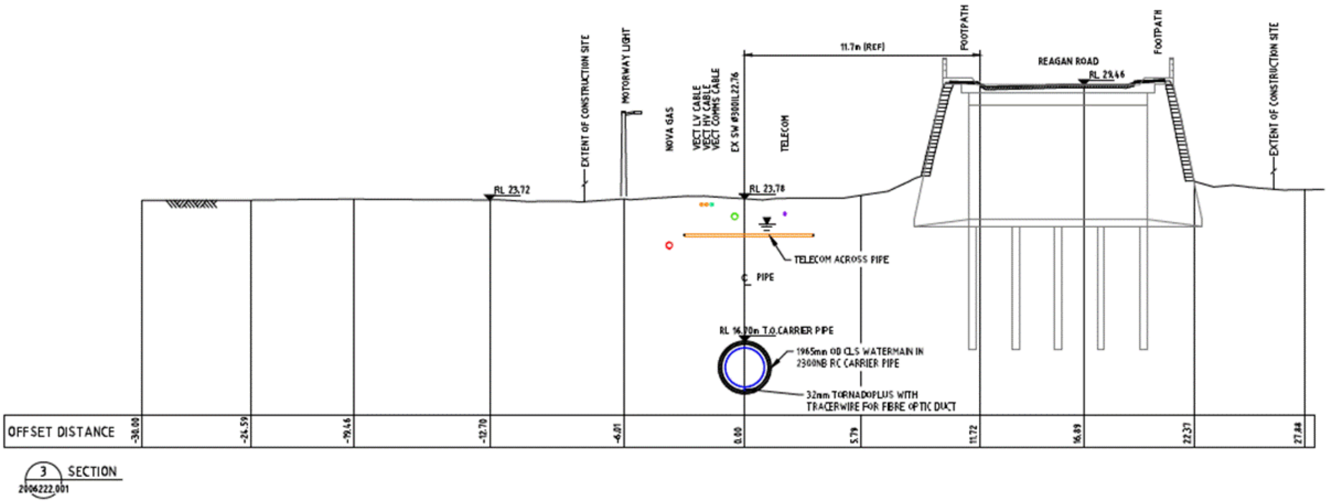


Figure 4: SH1 Crossing Services Section 2 – proximity to overbridge piles



3 DESIGN AND PLANNING

Design and planning of the SH1 crossing alignment via trenchless installation and determination of the design requirements for permanent structures were undertaken as part of the Hunua 4 pipeline permanent works design. Then the design of the temporary works and the trenchless installation, including the carrier pipe design was tendered.

Tenderers assessed the ground conditions, the watermain alignment prerequisites, and stakeholder requirements to determine:

- Tunnelling methodology and TBM selection / design,
- Design of the carrier pipe and final vertical and horizontal alignment, and
- All temporary works design.

A risk assessment process (Section 4) for each of the significant risks identified in the risk register was undertaken collaboratively with trigger levels set and contingency measures agreed. Settlement was one of the major risks identified and a careful planning and monitoring framework was established prior to construction at design stage as presented in section 3.2.

The major driver for a trenchless crossing was that SH1 is New Zealand's busiest road, with limited diversion options. NZTA would not accept open trenched construction due to the significant disruption to motorists.

3.1 TUNNELLING METHODOLOGY SELECTION

The objective of the tunnelling was to cause the minimum amount of settlement at the ground surface, however it was accepted that given the poorly consolidated ground some settlement was likely.

An Earth Pressure Balance (EPB) Tunnel Boring Machine (TBM) was considered most suitable for the ground conditions, whilst controlling face pressure to limit settlement. An EPB machine can be either closed face or a partially closed face machine with the capability of adjusting the pressure in its front compartment to match the outside pressure of ground water. Such technology limits groundwater drawdown and therefore limits consolidation settlement.

Photograph 1: SH1 Crossing 2.75 m diameter EPB TBM

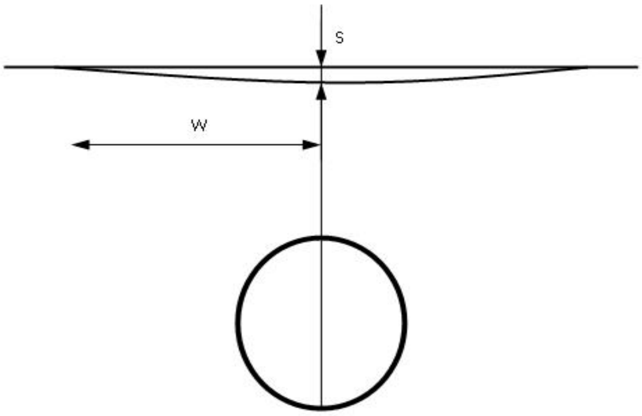
3.2 SETTLEMENT

Two key sources of ground settlement include dewatering creating **consolidation settlement** and direct ground movement or **collapse settlement** from tunnelling. The settlement from dewatering depends on the composition of the ground, degree of consolidation, permeability, and amount of groundwater movement. With closed or partially closed face TBMs, especially with EPB type, ground settlements resulting from groundwater drawdown can be minimised.

3.2.1 CONSOLIDATION SETTLEMENT

Ground settlement from groundwater drawdown related to tunnelling typically results in a wider spread cone of depression (Figure 5 Width (W)) and happens gradually over a longer period of time than other types of settlement. Careful monitoring of changes in groundwater level can provide a good indication and warning of the potential for ground settlements and mitigation measures can then be applied.

Figure 5: Schematic of Tunnelling Induced Settlement



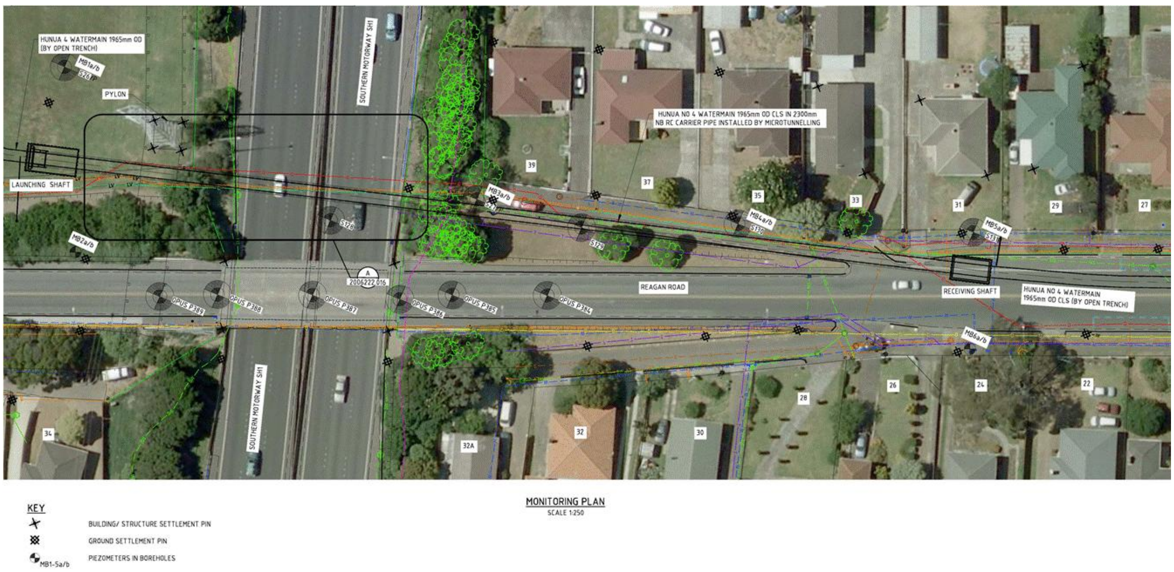
3.2.2 COLLAPSE SETTLEMENT

Settlement derived from ground movement associated with tunnelling has two components; **face volume loss** and **tail volume loss.** Face volume loss can be limited by controlling TBM progress in conjunction with balancing the groundwater pressure. The tail volume loss results from the void creation between the outer edge of the TBM and the outside diameter of the carrier pipe. Additional loss can also be created if the TBM requires steering for alignment control. The overcut void for this drive was 56 mm, but some alignment control was required that may have created additional displacement.

The direct settlement from tunnelling is typically greatest above the centerline of the pipe. The width of the cone of depression (Figure 5 Width (W)) depends primarily on the depth of the pipe and on the total volume losses, however typically the deeper the tunnelling bore, the less the settlement at the surface is. Typically with the vertical alignment designed at a depth of 2.5 – 3.0 times the pipe diameter, and with an EPB TBM the direct settlement resulting from tunnelling can be expected to be minimised. The SH1 drive depth was 2.25 – 2.75 times the pipe diameter.

A settlement monitoring array was developed in the Settlement Monitoring Plan consisting of isolated points around critical structures and an array over the bore length. The settlement monitoring of the SH1 carriageway was performed remotely after an initial progressive temporary lane closure to install the reflective marker pins.

Figure 6: Settlement Monitoring Control Points



4 RISK ASSESSMENT

The major risks identified were the potential effects on:

- a transmission pylon,
- SH1,
- the Reagan Road / Preston Road overbridge,
- the private properties adjacent to Reagan Road.

A risk assessment process for each of the significant risks identified in the risk register was undertaken agreed summary of which is presented below.

4.1 TRANSPOWER PYLON

One of the significant risks identified in the risk register was induced settlement on the Transpower Pylon, part of the Bombay to Otahuhu 110KV transmission line. The pylon is founded on a slab on grade, without piles.

After investigation, and following discussion with Transpower, scheduled strengthening of the pylon foundation was brought forward as a contingency and completed prior to tunnelling commencing. The launch shaft was then positioned 12.5 m from the Pylon to comply with the NZ electrical code of practice, with critical differential settlement values set in consultation with Transpower and with NZTA for SH1. Those critical settlement values became trigger values for any contingency measures to be actioned. The critical settlement values were related to the groundwater levels. In principle, if groundwater levels were kept within the seasonal variations then settlement from groundwater drawdown should have no impact on the structures within the area of groundwater drawdown.

4.2 STATE HIGHWAY 1

NZTA required a Service Agreement to be signed between NZTA and Watercare. The Agreement was subject to various conditions including a bond, and final approval was required after NZTA's review of ground conditions, the detailed design and the detailed methodology. The agreement set the maximum allowable settlement of the carriageway to 15 mm. NZTA also required the watermain to be inside a carrier pipe and to cross at a right angle to SH1 (or as close as possible).

The design team situated the launching shaft 28 m from SH1. The design intent was to monitor settlement as the bore progressed towards SH1 to gain confidence that limits would not be exceeded and adjust tunnelling technique as required.

A design process was undertaken to estimate the tunnelling induced settlements, and maximum ground deformation, with a contingency plan developed in the event that this occurred. Initial settlement modelling was undertaken, determining an increase in vertical stress resulting from a drawdown of groundwater. The initial modelling showed maximum predicted water inflows of up to 0.1 m^3/sec during shaft excavation. Potential for settlement of the ground surface resulting from collapse settlement related to tunnelling was calculated for the SH1 crossing. Calculations were based on various assumptions and were generally conservative. Settlement calculations were carried out for face volume loss of 1% and 4%. 1% being good performance on balancing the pressures (Table 2) and 4% indicating poor performance on balancing the pressures at the front face.

A worst case overcut of 50 mm was assumed with a full closure of the annual gap at the tail.

Table 2: Maximum Predicted Settlement Calculation

1% volume loss, face loss only	5-6mm
1% volume loss, face and tail loss combined	40-50mm
4% volume loss, face loss only	20-25mm
4% volume loss, face and tail loss combined	55-65mm

Given the 15 mm limit it was concluded in the Settlement Monitoring Plan that both control of groundwater draw down and operator control of face pressure and volume loss would be critical to ensuring a successful bore. For any settlement of the SH1 carriageway, the mitigation measures were firstly to impose a temporary speed restriction and secondly to remediate the asphalt carriageway layer. 65 mm settlement was considered a worst case scenario.

4.3 REAGAN RD / PRESTON RD OVERBRIDGE

The Hunua 4 alignment was aligned 10.7 m from the bridge pier at the closest point. Abutments and the central support pier of the overbridge are founded on 360 mm diameter concrete piles 6 m deep. Ground settlements caused by the tunnelling were considered unlikely to affect the bridge foundation. The shafts were located closer to the abutment embankments and as a precautionary measure there were four settlement monitoring pins installed in addition to the groundwater monitoring points.

4.4 ACCESS WAY TO AND PROPERTIES AT 33-39 REAGAN RD

The second part of the tunnelling drive was under the road access way to the properties at 33-39 Reagan Rd. The ground settlements were not expected to be greater than under the SH1 carriageway and the consequences of any settlement were considered to be less. Settlement limits were specified in the resource consent for the project, and any reinstatement if required would be relatively easy to undertake. Settlement monitoring of the properties was undertaken.

5 CONSTRUCTION

5.1 CONTRACTOR'S METHODOLOGY AND TOLERANCES

Bothar Boring Ltd decided to use a partially closed faced 2,756 mm diameter earth pressure balance (EPB) tunnel boring machine (TBM), as they estimated the ground conditions to be stable enough to not require a fully closed face TBM. The pipe jacking utilised a 2.7 m outside diameter, 2.25 m inside diameter carrier pipe with a custom coupling without a strapping band, resulting in a minimised overcut.

A pre-construction meeting between Watercare, NZTA, FHJHJV, and CBG allowed Bothar Boring Ltd to present their trenchless methodology and for the parties to discuss the key risks (settlement of SH1, pylon, and settlement effecting existing services and local houses). The Resource Consent conditions for the project already incorporated differential settlement limits for the pylon and local houses, so discussion centered on acceptable settlement limits for the highway. It was concluded that the 'ride quality' for motorists would be compromised if the trenchless construction caused pavement settlements in excess of 15 mm and Bothar Boring Ltd considered this was achievable as long as ground conditions matched the investigation findings.

The line and level tolerances of the installed carrier pipe were agreed with Bothar Boring Ltd to be less than 50 mm deviation from the design alignment. This allowed them to avoid issues with the installation of the welded steel watermain (intended to be installed in a straight line vertically and horizontally).

The alignment of the tunnel was checked by survey, after the pipe jacking was completed and before the installation of the steel watermain. The maximum deviation was less than 50 mm over the 150 m drive length. This level of accuracy was an important achievement and the most prominent deviation in the tunnel was a 'bump' in the tunnel base about 50 m from the launch shaft.

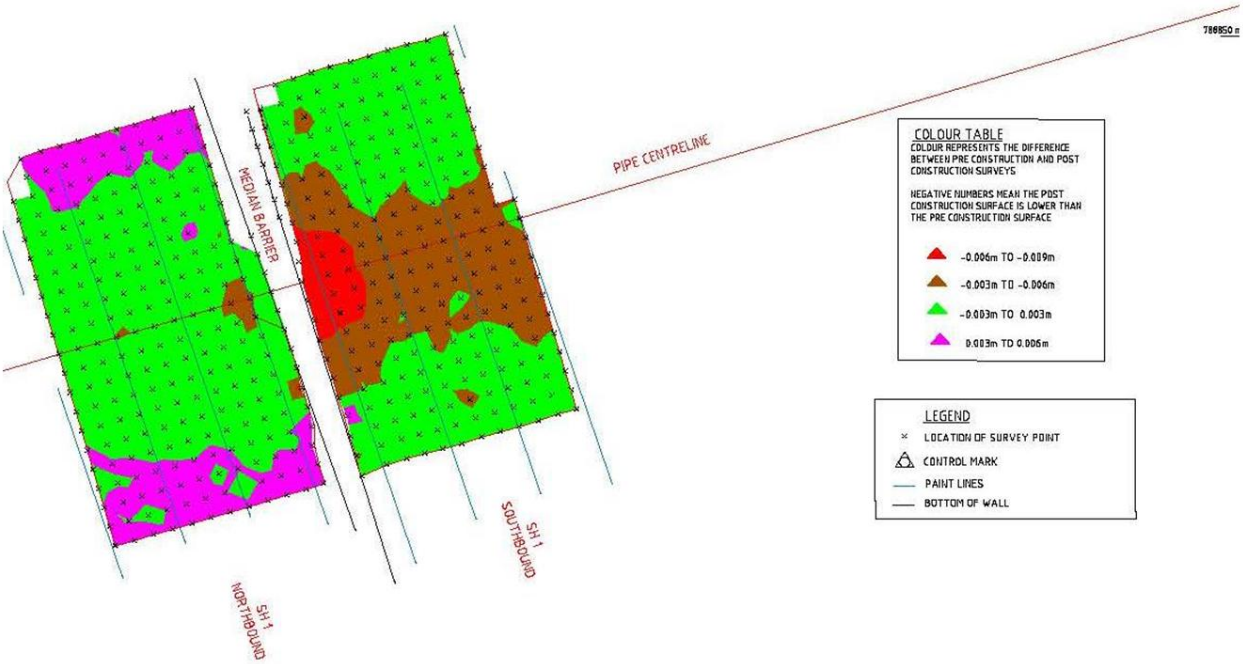
The steel watermain pipes were each 6 m length and each pipe was welded to the previous pipe before insertion into the carrier pipe. To avoid damage to the soft polyethylene coating of the steel pipeline each pipe had a steel weld-band near the welded joint (also at 6 m centers). The weld band was tightened on to the pipe and had two wheels at '5 o'clock and 7 o'clock positions'. This allowed the steel pipe run along the base of the carrier pipe like a railway carriage. At the 'bump' in the carrier pipe the wheels under the steel watermain became slowly 'airborne' until the deflection in the steel pipeline allowed them to 'land' approximately 15 m further along the carrier pipe base. Had the bump been any larger the carrier wheels would have failed under the 1 tonne per meter length weight of the steel pipe, the polyethylene coating would have been damaged.

The Contractors' methodology closely matched the methodology envisaged by the CBG designers. Fulton Hogan Ltd installed the launch and reception shafts and Bothar Boring had configured the launch shaft without a soft eye seal around the TBM. A pre breakthrough check, undertaken prior to cutting an annulus in the sheet piles determined that the ground was competent enough to continue and ground water influx manageable and Bothar Boring Ltd launched the TBM on 3 April 2013.

The Resource Consent conditions for the project incorporated differential settlement limits for the pylon and local houses and a 15 mm limit was agreed as the maximum settlement for the surface of SH1. Fortunately none of these limits were exceeded but the surveys showed an area of approximately 50 m wide and 150 m long centered on the tunnel alignment had settled up to 10 mm (Figure 7). A settlement of 5 mm occurred over the majority of the monitoring area and meant that the differential settlements were within tolerable limits.

Following installation of the jacking pipe compensation grouting was undertaken through grout ports. Generally the annulus around the pipe held open for the first half of the drive and accepted grout, and had relaxed and closed over approximately the second half of the drive with very little grout take.

Figure 7: Settlement Results for SH1 showing induced settlement over the tunnel alignment



Photographs 2-6: Construction Photos of the completed bore and the general construction setup.

5.1.1 LESSONS LEARNT

The alignment for the crossing was difficult and tight, but was designed and executed well, accommodating the ground conditions and minimising effects on surrounding infrastructure, resulting in a successful bore.

Even with a successful trenchless installation there are always lessons to be learnt, and things in hindsight that could have been improved.

5.1.2 ANNULUS CONTROL

Bothar Boring Ltd had envisaged that the annulus, if not significantly overcut, would be able to be sealed against ground and water influx with sandbags and packing. Groundwater influx was significant, and the steady flow migrated fine soil material into the launch shaft. Several attempts were made to grout and pack the annulus to restrict the flow (Photograph 7), which were partially successful; however around the 9th of April and for a few days following, a series of "bursts" of disturbed ground entered the shaft. This influx of soil resulted in a settlement crater immediately in front of the launch shaft as see in Photograph 4. The progression of the slump was monitored and the hole eventually backfilled with site concrete. In hindsight, the lesson learnt would be to install a soft eye to control local ground influx near the shaft.

Photograph 7: Annulus Packing – welded plate and sand bag to seal the annulus from ground ingress

Photograph 4: Settlement hole.

5.1.3 TUNNEL ALIGNMENT

Small deviations in carrier pipe alignment, whilst not detrimental to the success of the bore can have consequences for production pipe installation. The lesson learnt is that, in poorer tunnelling conditions than those experienced on this bore, or as the result of other construction issues, deviations of greater than 50 mm in the carrier pipe could occur. This is difficult to predict prior to construction, and could lead to difficulties in installing a steel pipeline in the carrier pipe with an undamaged polyethylene coating. The conclusion is that the tunnelling conditions can potentially impact on the selection of pipe material and the installation method, and in poor ground contingencies for carrier pipe deflection effecting production pipe installation should be considered.

5.1.4 SURFACE SETTLEMENT

Some permanent settlement is almost inevitable as a result of trenchless construction, even in reasonable tunnelling conditions like those experienced in the Hunua 4 crossing under SH1. Therefore settlement limits should be agreed with stakeholders that match the known tunnelling conditions and contingency plans devised in the advent of 'poorer than expected' ground conditions.

6 CONCLUSIONS

Even with a successful trenchless installation there are always lessons to be learnt, and things in hindsight that could have been improved.

Carrier pipe deviation uncertainty can affect the production pipe material selection and certainty of installation.

All trenchless activity requires good investigation, design and planning to determine a suitable construction methodology.

The trenchless planning process should always be collaborative between designers, geotechnical specialists, specialist contractors, and stakeholders.

An integrated planning and design approach allowed the risks to be minimised for this bore.

ACKNOWLEDGEMENTS

We wish to acknowledge Fulton Hogan John Holland Joint Venture and Bothar Boring Limited for successful construction of the crossing.

We wish to thank NZTA / the Auckland Motorway Alliance, Transpower, and the other service asset owners for their cooperation and assistance.

REFERENCES

Edbrooke, S.W. (compiler) 2001: Geology of the Auckland area. Institute of Geological and Nuclear Sciences. 1:250 000 geological map 3