

Advancing Cut and Cover Tunnelling in Auckland – Experience Gained from New Lynn Rail Trench

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ABSTRACT

KiwiRail has a goal of significantly improving passenger rail services to west Auckland, as part of its DART (Developing Auckland's Rail Transport) suite of projects. The New Lynn Rail Trench (NLRT) project was a significant component of this work. The NLRT project facilitates essential double-tracking of the Western Auckland line, and provides a new commuter transportation hub within a busy town centre.

The 1.5 km stretch of track through the New Lynn town centre has bisected the township for over 80 years. Double-tracking the line provided the opportunity to grade-separate road and rail, which would help to re-connect both sides of the township and enhance public amenity and safety.

KiwiRail employed an Early Contractor Involvement (ECI) procurement model engaging a consortium comprising Fletcher Construction, Beca Infrastructure and Synergine to develop a Design and Construct solution.

With weak soil conditions and high groundwater levels, a diaphragm wall construction method to form the trench walls was selected as the best solution. Further challenges to construction included working within a narrow rail corridor constrained by buildings along much of the boundary, and a requirement to keep the existing line open for rail services. Detailed construction planning was required to ensure rail and construction safety in a constrained space whilst building the trench walls, propping these walls during excavation, and constructing the base slab in the most efficient way possible.

This paper will detail the geotechnical conditions and the additional constraints that made this rail trench a challenging structure to design. It will also describe the final design and construction methodologies that were employed, that allowed very successful completion of the project.

Additionally, this paper will review the design and construction lessons applicable to the current Victoria Park Tunnel project, and indeed for all future cut and cover tunnel projects in the Auckland isthmus.

INTRODUCTION

With an ever-increasing population and a road network stretched to capacity, Auckland commuters will become increasingly dependant on rail services. With the existing single track through west Auckland at its maximum capacity, Auckland regional transport planners committed to double-tracking of the Western Line to increase the frequency of passenger train services.

KiwiRail's New Lynn Rail Trench (NLRT) project was created to provide double-tracking through the urban centre of New Lynn as part of the double-tracking of the entire Western Line. This project formed a critical part of KiwiRail's DART (Developing Auckland's Rail Transport) suite of projects.

The NLRT project required 1.5 km of double-tracking along the existing rail alignment through the heart of New Lynn commercial district. The existing single rail line was built in 1880 and the town centre grew around this line over time. This meant that before the project the rail line divided the commercial area, severing access between businesses and residences and crossing major urban roads and footpaths.

This division of road transport caused by the previous rail line can be seen at the Clark Rankin intersection (Figure 1). With double tracking, and the increased frequency of trains to ten minutes

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FIG 1 - Clark Rankin intersection.

each way, the degree of community severance and increased road congestion would have been unacceptable.

A rail trench to provide grade separation from road and pedestrian traffic was the logical solution. Providing grade separation using a trench meant that road and pedestrian traffic would become uninterrupted by rail, which would enhance safety and accessibility through the urban area, and allow further urban renewal and enhancement projects above the trench.

KiwiRail approached the design and construct civil engineering market in 2007, with Fletcher Construction and Beca Infrastructure winning the 'attributes only' bid to design and construct the works on a cost reimbursable basis. The financial structure included a fixed margin against a target outturn cost, and a pain/gain mechanism whereby innovation and better-than-budget performance was incentivised through sharing of any profits or loss.

The scope of work for the consortium was to design and construct the grade separated rail trench whilst maintaining rail passenger and freight services throughout construction. The three existing road crossing points were to be provided as bridges, along with the provision of two new crossings.

For the construction team, building a trench in this location presented considerable challenges. These included weak soil conditions, a high water table, and restricted width because of adjacent buildings. Given these conditions, a diaphragm wall construction method was selected to form the trench walls. Providing the best solution for these diaphragm walls required very detailed design and diligent construction.

DESIGN CONSIDERATIONS AND SOLUTION

Design considerations

The design of the rail trench initially incorporated provision for a two-storey building development over the central section of the trench. This drove the specimen design to incorporate significant vertical load carrying capacity, in addition to having to support train loads at the base of the trench and road bridges over the trench.

During the investigation phase a significant increase in depth to suitable bearing strata was identified and a value engineering exercise was undertaken to try to significantly reduce construction

costs. As part of this exercise the requirement to make provision for building development over the trench was removed, significantly reducing the demand for vertical load carrying capacity.

The final design required a 1000 m long trench graded from ground surface to 8 m below ground level. The trench width varied from 15 m at the ends to 22 m at the location of the centrally located station platform.

The trench structure was required to support: the up-to 8 m deep excavation and associated lateral loads; the vertical downward loads from trains; the traffic and bridge loads across the trench; and the vertical upward loads from soil rebound and long-term buoyancy due to groundwater.

Thorough geotechnical and hydrogeological analysis was required to identify design loadings for the proposed structure, and to establish potential effects on neighbouring properties (to fulfil resource consent requirements).

Geology

The site is located on a relatively narrow low-lying area of the Auckland Isthmus, with ground levels typically between 5 and 15 m above mean sea level. The geology of the site consists of Tauranga Group Alluvium (TGA), underlain at depth by Waitemata Group East Coast Bays Formation (ECBF) sandstone and siltstone.

The alluvium consists of primarily cohesive soils, silts and clays, with lenses of amorphous organic peat deposits. Isolated lenses of ‘sandy’ deposits were encountered in some boreholes. The strengths of these deposits ranged from localised soft peat layers to firm to stiff silts and clays.

Groundwater levels were measured for a period of 12 months prior to the proposed construction and indicated seasonal variation in levels between 1.0 and 3.0 m below ground level.

The ECBF comprised residual soils of similar strength to the slightly stronger overlying alluvium, grading to variably interbedded sandstone and siltstone forming the bedrock in the area. Water levels recorded in piezometers screened within the ECBF indicated deeper levels than the overlying alluvium, typically 8 to 12 m below ground level. A generalised soil profile along the alignment is presented in Figure 2.

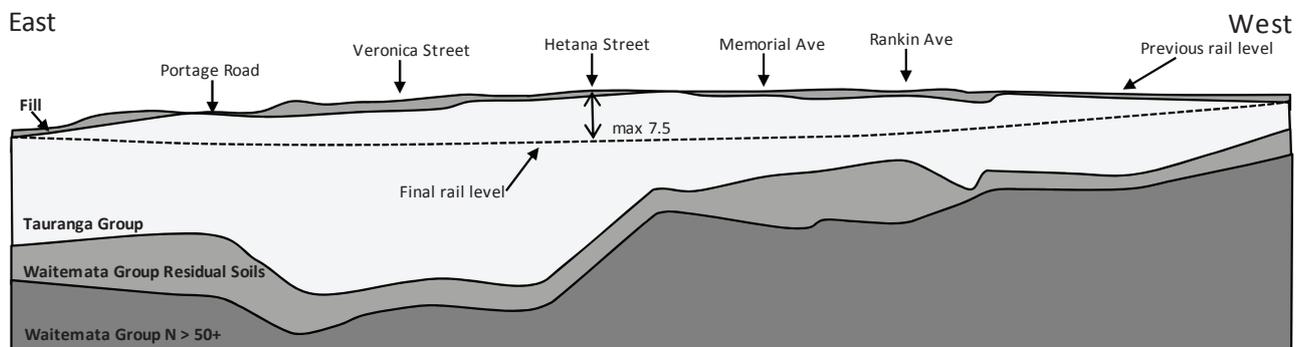


FIG 2 - Generalised soil profile on trench alignment.

Hydrogeology

The area of the rail trench is in close proximity to an estuarine branch of the Waitemata Harbour, and is bound on the eastern and western ends by incised tidal streams at around RL 2 m. This is consistent with the level of groundwater measured in the ECBF across the site and considered representative of the regional water table. The dominant groundwater flow is from south to north, with levels falling towards the harbour.

Groundwater levels in the overlying TGA are typically higher to the west, at RL12 m, and fall in a north easterly direction to around RL8 m. Groundwater is dominated by the generally low hydraulic conductivity of this material.

Groundwater modelling

Groundwater modelling was undertaken to assess the potential effects of the proposed rail trench on neighbouring properties. This modelling is the subject of a specific research paper by S France (2010).

Both 2D and 3D modelling was undertaken, and this indicated a maximum drawdown of 0.3 m immediately north (down-gradient) of the trench walls, with measurable drawdown extending for less than 40 m to 50 m. A similar magnitude of groundwater elevation (damming) was predicted on the southern side, due to the low permeability diaphragm walls reducing through-flow. However this degree of change is within the recorded changes in groundwater level due to seasonal effects, identified as being 0.5 m – 1.5 m during monitoring.

With a number of buildings in close proximity (3 to 5 m) to the rail trench, settlement analyses were based on the conservative assumption that the TGA sediments were not preconsolidated. These analyses indicate that on this basis maximum settlements of around 25 mm might be possible immediately adjacent to the trench, reducing to <15 mm within a distance of around 15 m. However, these predicted settlements were based on full consolidation of these soils and therefore were reported as maximum credible effects. Analysis based on a degree of pre-consolidation indicated maximum settlements of in the order of 5 mm were more likely.

Groundwater seepage (through the excavated floor) was estimated to be of the order 0.1 m³/d per metre length of floor.

Retaining wall design

With removal of the need to accommodate building loads above the trench, the trench walls were designed solely as retaining structures, and piles below the base slab were designed to resist vertical loads. Design of the retaining walls was undertaken using 1D limit equilibrium methods and 2D finite element models to determine both design loadings on the trench structural elements and to establish deflection and settlement effects that could affect adjacent buildings.

The structural demands of the trench walls were determined using the 1D limit equilibrium methods at a total of 13 sections along the walls, with the outputs providing upper bound loadings suitable for use in limit state design methods. Wall deflections were also determined at the critical, deepest section at Ch 19300 m; these were correlated using 2D finite element analysis, which indicated similar relative performance.

These analyses indicated the range of lateral wall movements above the base slab level to be in the order of 20 to 40 mm at the wall face, for these upper bound loading conditions. The graph in Figure 3 presents the deflected shape for Ch 19300 m. As can be seen the modelling predicts just over 40 mm at base slab level, although this is dependent on full active loads developing on the wall prior to constructing the base slab.

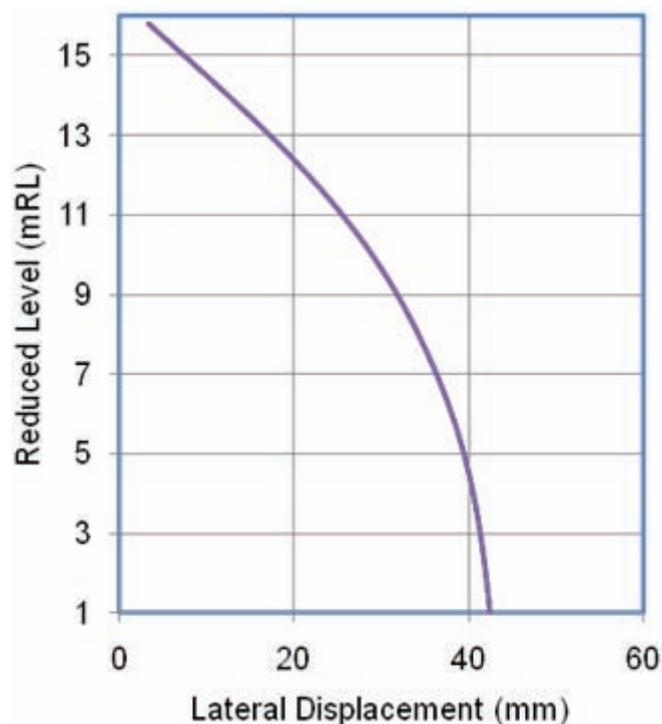


FIG 3 - Calculated lateral displacement at Ch 19300 m.

Sensitivity analyses using higher stiffness values were used to assess their effects on deflections and were found to suggest a ± 25 per cent change for the range of values that were technically possible.

Vertical settlement effects behind the walls due to wall deflections were assessed using empirical methods, which indicated maximum vertical movements in the order of 20 to 30 mm.

With typical distances of 3 to 5 m between buildings and trench walls, estimated settlement effects from wall deflections were assessed to be worse for the deeper excavations. This was predicted despite these sections of wall being propped both during excavation and permanently; permanent propping at the surface, with up to two levels of temporary propping (these being replaced by propping by the base slab upon completion of the trench structure).

Combined effects

Analysis of the effects of the trench on adjacent properties was required for the Resource Consent application. The combined effects of both groundwater drawdown and wall deflection needed to be considered. For the consenting process it was important to establish that the maximum potential effects were ‘less than minor’, so that a non-notified consent could be considered by the consenting authority.

Potential sources of settlement during construction were assessed to be a function of:

- soil relaxation during diaphragm wall excavation,
- deflection of wall elements during excavation, and
- groundwater drawdown effects.

The combined effects were based on upper bound estimates for soil relaxation and wall deflections, and full consolidation effects of both maximum credible and expected drawdown estimates.

The effects of diaphragm wall excavation were calculated to be typically less than ten to 20 per cent of those due to wall deflection.

Figure 4 presents the total combined effects of dewatering, diaphragm wall construction and trench excavation based on the methods outlined above, at Ch 19300 m. These represent the range of movements possible if:

- lateral earth pressures for drained conditions are fully mobilised prior to base slab construction,
- lower bound soil lateral stiffnesses are realised in mobilising passive pressures,
- full drawdown effects are realised, and
- full consolidation of alluvial deposits occurs prior to groundwater level recovery after construction.

The above estimates of combined effects (Figure 4) were presented for consent, and the permissible effects and alarm levels established as conditions of consent were:

- groundwater drawdown: 0.6 m alert level and 1.0 m alarm level,
- settlement of buildings: 10 mm alert level and 15 mm alarm level,
- settlement of ground: 15 mm alert level and 25 mm alarm level, and
- angular distortion: 1:1000 alert level and 1:500 alarm level.

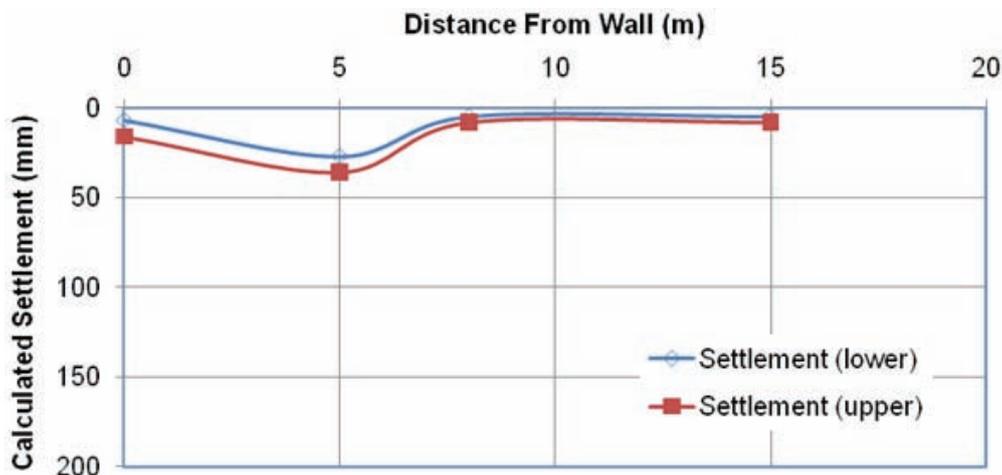


FIG 4 - Calculated settlement at Ch 19300 m.

CONSTRUCTION METHODOLOGY

Key constraints and sequencing

The typical constraints of working in a dense urban environment, such as movement of adjacent pedestrians and traffic, noise, vibration and site access, all had a major impact on the chosen construction methodology and sequencing. Additionally, at New Lynn the restricted width of the work site and the presence of a live railway provided significant additional challenges over and above typical urban construction projects.

Prior to commencement of the project the rail track was located at grade in the centre of a 20 m wide corridor, leaving two narrow working areas either side of the line. To maximise the separation between live rail services and construction activities, and also to maximise the remaining width of corridor available for construction activities, a sequence of track slews was planned. This process is explained in the following.

- Step 1 During the enabling works phase, a temporary at-grade alignment and associated passenger platform was constructed against the southern boundary of the corridor. This provided a maximum 14 m width of corridor for the construction of the northern wall.
- Step 2 On completion of the northern wall, including the capping beam, H piles, and necessary rail fit out (including a second temporary platform), the operative train track was 'slewed' onto the newly-constructed north wall. Work on the south wall and the remainder of the trench could then proceed.
- Step 3 Work within the new walls included installation of permanent precast concrete props, bulk excavation of the trench including temporary propping, construction of the reinforced concrete base slab, and then removing the temporary props.
- Step 4 With the trench completed the operative track could be 'slewed' into its final alignment within the newly constructed trench.

Figure 5 shows the construction sequence and adopts the same 'Step' nomenclature.

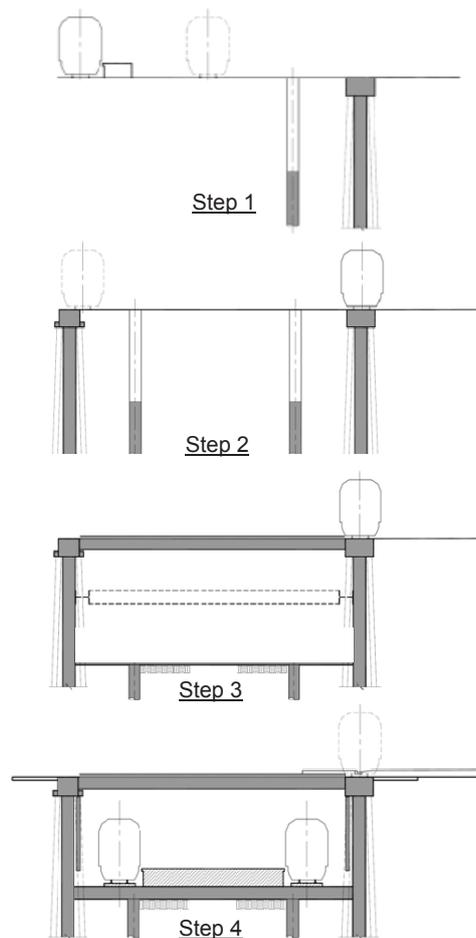


FIG 5 - General construction sequence.

Diaphragm wall construction

We selected a diaphragm wall technique for constructing earth and groundwater retention structures that involved the use of a bentonite clay slurry to support the excavation for wall panels. The technique was originally developed for the oil well drilling industry, and was adapted for use in civil engineering over 50 years ago. The use of slurry enables discrete slots to be excavated and remain stable such that reinforcement can be inserted and concrete placed via tremie; the basic methodology is detailed in Figure 6. The technique has been little-used in New Zealand until now.

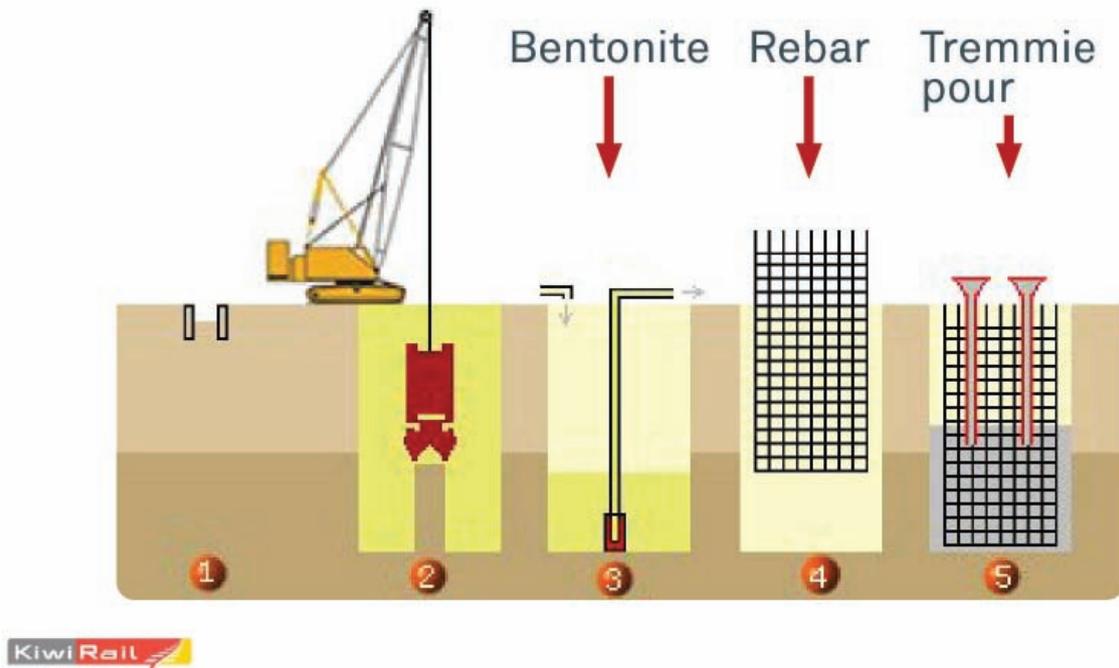


FIG 6 - Basic diaphragm wall construction methodology.

Important considerations during diaphragm wall construction

During wall construction, stability of the trench is a major concern from a safety, settlement and productivity point of view. This was rigorously assessed given the prevailing soil and groundwater conditions, coupled with the applied loads from construction plant adjacent to the trench. The elevation of the bentonite above the groundwater is a key parameter in the maintenance of trench stability during construction, requiring careful management of both the quality and level of the bentonite during panel construction.

Currently no New Zealand standard specification for diaphragm walls and bentonite support fluids exists, so the methodology was developed based on the *ICE Specification for Piles and Embedded Retaining Walls* (2007). Whilst we were able to call upon experience from previous piling projects, we also provided additional training of site teams to address the specific issues relating to this diaphragm wall construction technique.

Between the discrete diaphragm wall panels is a cold formed joint. This joint is formed using a steel profile or stopend that is placed at the end of each panel prior to placing the reinforcement and concrete; this effectively acts as a form. The stopend is removed when the adjacent panel is excavated. The profile of the stopend was developed with the structural designers to meet shear requirements across the joint and to enable the incorporation of a waterbar across the cold joint between the diaphragm wall panels.

The supply and conditioning of used bentonite for reuse can influence productivity. The linear nature of the site and limited locations for storage and treatment stations necessitated construction of a significant bentonite reticulation system. This system was built to provide multiple delivery and return pipelines and pumps, and on-site there were dedicated personnel managing the system. To meet the necessary capacity the system required use of large diameter pipes and scheduled cleaning procedures. Additionally, with both clean and dirty bentonite needing to be pumped up to 500 m, intermediate pumps were installed.

Space constraints meant that the reinforcement cages could not be fabricated within the narrow rail corridor. Instead they were fabricated on a vacant site nearby, and transported to site in two pieces. Each piece was lifted and lowered into the slurry-filled trench using specialised lifting beams. As the structural design required the reinforcement in a panel to be continuous, the two pieces were then lifted and spliced together, before being lowered into the slurry filled trench for the final time, as a completed and continuous cage.

Due to the depth and variability of the Waitemata Group deposits the diaphragm wall was effectively 'floating' in the soft Tauranga Alluvial deposits and not founded in the weak rock.

RESULTS FOLLOWING CONSTRUCTION

Monitoring

Because of the assessed potential effects, a groundwater monitoring program was developed using inclinometers adjacent to the trench walls to measure wall deflections, and settlement monitoring using building and ground settlement pins.

In addition, at two locations both pressuremeters and piezometers were installed on the structure to determine actual loadings on the structure.

The majority of piezometers (17 out of 21) were installed in the shallow TGA: 11 were located to the north of the rail corridor and six to the south. The remaining four piezometers were screened within the Waitemata Group.

Baseline monitoring began on 27 June 2009 prior to commencement of bulk excavation between the diaphragm walls, and continued for almost five months. The monitoring ended on 30 June 2010, approximately six months after the final base slab was poured.

Groundwater monitoring results

Of the 17 piezometers screened in the TGA only six showed distinct trends of decreasing water levels that could be attributed to the excavation.

Drawdown (from the average groundwater level) was typically 0.3 to 0.4 m.

Four of the six piezometers in which drawdown occurred were located along the same cross-section (Ch 19470). This coincided with the location where a discreet sandy horizon was identified laterally between multiple boreholes. It is possible that the presence of this sandy horizon contributed to the drawdown in this area.

In all other TGA piezometers water levels were observed to change in response to rainfall. In four of these piezometers there was a slight overall downward trend in water levels, although this was small at generally <0.2 m over the period of the project.

Groundwater seepage

Groundwater seepage was minimal throughout the project, with some minor inflows occurring through the joint between the base slab and the walls. Visible groundwater seepage into the excavation was recorded at only one location (Ch 19350), and damp patches in the diaphragm walls were noted at a few locations. Inflows into the excavation were so minor that the only requirement for active pumping was to remove rainfall run-off entering the excavation.

Settlement and deflection monitoring

Survey of settlement-monitoring pins on the ground and buildings prior to construction recorded insignificant background movements, in the order of ± 1 to 3 mm. However, at one location a settlement of almost 6 mm was recorded prior to any excavation commencing.

There was no indication that any additional settlement due to the excavation occurred, with ongoing monitoring during construction showing changes in elevation of up to ± 3 mm.

Differential settlements, measured along transverse arrays across surrounding roads, were generally better than 1:7000, with isolated results in the range of 1:2000.

Inclinometer monitoring was used to identify lateral movements during diaphragm wall construction and trench excavation. Results indicate that where movement was recorded it was always less than estimated.

Maximum lateral displacements were recorded as 3 mm during diaphragm wall excavation and up to 13 mm at completion of excavation and removal of temporary props. These are typically between 30 and 40 per cent of the calculated values and are roughly 60 to 70 per cent of the deflections given by sensitivity analyses using upper bound soil stiffness values.

The graph in Figure 7 presents the actual measured deflection at the deepest section of the trench using inclinometers installed at approximately 1.0 m behind the diaphragm wall.

Pressuremeter cells and piezometers installed behind the walls indicated wall pressures in the order of 30 to 60 per cent of the unfactored design pressures.

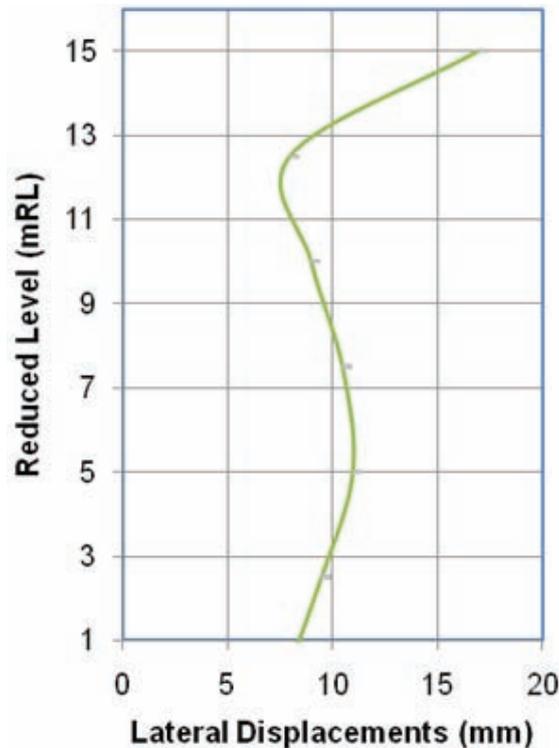


FIG 7 - Measured lateral displacement at Ch 19300 m.

Settlement effects and influence on neighbours

The recorded groundwater effects were generally of a similar order to that predicted in the analysis.

However, settlement effects due to both groundwater drawdown and wall deflections were found to be significantly less than those predicted. As such, the effects on neighbouring buildings were found to be typically less than half of the trigger levels set for consenting purposes. No damage to any surrounding structure was reported that could be attributed to the effects of excavation and associated groundwater drawdown.

Neighbouring structures are shown in Figure 8.

Summary of monitoring results

Almost all monitoring results indicate actual lateral displacements significantly below those predicted and planned for. This is seen as a positive result, as actual displacements stayed well within an acceptable range, and mitigation measures were in place should the larger designed displacements occur.

These positive results are likely to be primarily a result of the cohesive nature of the majority of the retained soils and their associated low permeability. These attributes resulted in negligible consolidation occurring during the period in which drawdown occurred and soils remaining in the undrained state, resulting in loads less than those predicted being applied to the structure.

In a similar way, translating the laboratory-measured soil stiffness values into the actual behaviour of the soils, both in compression at the toe of the wall and under relaxation behind the wall, is difficult to quantify. In this instance, the laboratory results are not the strongest driver of structural form.



FIG 8 - New Lynn rail trench and neighbours.

While modelling and empirical methods have been effective in predicting the shape of the lateral deflection of structural forms, the magnitude of deflections are typically over-estimated due to conservative assessment of the soil loadings and stiffness. These conservative assessments should remain as standard practice as they ensure establishment of an upper bound of effects.

The methods for establishing the associated vertical movements also lead to conservative predictions. This is because the prediction methods are dependent on highly simplified soil models which do not account for the non-isotropic nature of the actual soil. This typically results in over-estimation of the extent of vertical effects.

Project outcomes

The NLRT project adopted a diaphragm wall construction technique that has been rarely used in New Zealand, and implemented it successfully in a highly-constrained urban environment. Furthermore, the project team gained experience that will further the use of this technique for building cut and cover tunnels.

Settlements, deflections and impact on adjacent buildings were generally all significantly smaller than predicted estimates. Safety was maintained for rail traffic, road traffic, pedestrians, and construction workers.

The construction methodology proved successful, with no significant delay caused to rail operations throughout the project. Our program for slew of the existing rail line worked successfully during the very tight time frames available for track closure.

The project was delivered to the client to the specified quality, and the first train in the trench was provided for on time (see Figure 9).

Rail trench construction milestones were met several weeks ahead of programmed forecasts, with rigorous productivity monitoring throughout. Furthermore, productivity gains produced budget gains through time related costs, which were shared with the client, KiwiRail.

LESSONS LEARNED AND FUTURE APPLICATIONS

The practical application of the diaphragm wall technology on the NLRT project has led to numerous learnings related to design and methodology, and the performance of the end product. Many of these



FIG 9 - First train in the trench.

learnings have already been taken by the project team to their current assignment: construction of a cut and cover tunnel for the Victoria Park Tunnel (VPT) project.

The learnings most significantly cover the application of design methods and parameter selection specifically for Auckland soil conditions. The learned knowledge also helps the project team to develop a diaphragm wall methodology that takes into account New Zealand standards, practices and available materials. It is this developing methodology that will be of benefit to all diaphragm wall projects throughout Auckland.

While all learnings cannot be covered with the confines of this paper, we highlight the following. These items show important knowledge gained from the NLRT project, and subsequent practices that have already been adopted at the VPT project:

- Modelling methods have been adopted based on correlation of results from NLRT: for example 2D FEM has been used to establish soil structure interaction, rather than 1D methods to provide just soil loadings for structural modelling.
- Parameter selection for both the TGA and ECBF soils have been re-calibrated based on results of the NLRT monitoring program.
- In the absence of New Zealand standards, specifications or codes of practice for diaphragm wall construction, research of international best practice and formation of an internationally experienced team resulted in the development of a full quality control management system covering all aspects of the methodology. This has been used and further developed with great success at the current VPT project.
- Development of methods to establish and model anisotropic behaviour of soils need to be progressed to improve predictions of lateral extents of excavation effects.
- Excavation of panels through soft alluvial soils adjacent to settlement-sensitive structures using bentonite support has been achieved with minimal ground displacement. This has provided the project team with the confidence to excavate adjacent to settlement-sensitive services at the VPT project (where there are soft uncontrolled reclamation fills and marine deposits).
- The diaphragm wall productivity achieved at NLRT confirmed that the primary plant selection using rope-suspended grabs and associated support plant was effective in the ground conditions from a program, cost and quality perspective.

- Site space constraints necessitated off-site cage fabrication, requiring that cages needed to be transportable on the road. For the 7 m long panels the cages needed full height splices over the trench which took considerable time. We learnt from this, and at the VPT project fabricated parts of cages are assembled on beds and lowered into the trench in one piece; therefore cage splicing is off the critical path.
- The stopends which form the joint between individual diaphragm wall panels were specifically made for the project. Due to structural design requirements these were not as stiff as initially intended, and some damage occurred to the stopends when they were removed. At VPT project the same stopends have been used, as this was the most cost effective solution, but a stiffer design has been developed for future projects as required.
- Excavation rates and methods were refined to optimise productivity using hydraulic rope suspended grabs in the TGA. An excavation trial was also performed in the Waitemata Group siltstones and sandstones to assess the dig-ability and digging rates in these deposits. The data from these experiences was directly applied to the cost and program assessment for the retaining walls at the VPT project.
- The use of bentonite in a narrow urban environment adjacent to public roads, rail, and pedestrians required a high level of control of the bentonite and spoil to minimise environmental impact. Some key environmental control measures included bunding and spoil transfer to separate site and off-site spoil transportation. This was very successful and has been adopted at the VPT project.
- The cover to some of the diaphragm wall reinforcement cages was minimal due to the soft ground and density of spacer blocks. At the VPT project the number of spacer blocks has been increased to reduce bearing on the walls of the trench and improve cover.
- At NLRT the connection detail between the diaphragm wall and base slab is a cold joint with no waterbar or waterstop. Careful joint preparation and post grouting has been planned and programmed at the VPT project in case it is required based upon the performance of the connections at NLRT.

CONCLUSIONS

The design and construction of the NLRT project has been successfully delivered to program and budget using diaphragm walls as the primary retention system.

A trench in this location presented considerable challenges for the project team. These included weak soil conditions, a high water table, and restricted width because of adjacent buildings.

Despite there being no New Zealand standards, specifications or codes of practice for diaphragm wall construction, the team learned from international best practice and developed a robust, efficient and safe design and construction methodology.

Considerable measuring and monitoring was performed to check settlement and displacement of the surrounding ground, particularly in relation to neighbouring buildings. The field measurements indicated that actual movements were significantly less than those designed-for, and no disruption was caused to the built structure or surrounding buildings.

Important lessons were learnt in designing and constructing the diaphragm walls, and the overall trench itself. From these lessons, the project team gained significant knowledge and experience regarding diaphragm wall construction methodology in Auckland ground conditions and within constricted urban environments.

This knowledge, and the resulting methodologies, have already been applied and further developed by the team at the Victoria Park Tunnel project.

With this locally-developed design and construction experience, the project team have established a very sound methodology for constructing diaphragm walls in Auckland. It will be very beneficial if this knowledge can be applied to future cut and cover tunnel projects in the region.

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