The Kirkbride Road Interchange

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Abstract: Kirkbride Road is a busy urban road crossing State Highway 20A, the main route between Auckland City and Auckland International Airport. The existing crossing is a signalised at-grade intersection which contributes to local congestion and has a high accident rate, both of which were expected to worsen as traffic volumes increase. The MHX Alliance was commissioned in 2010 to develop options for a new grade separated interchange. Three significant alternatives were considered; a 550m long viaduct carrying SH20A over Kirkbride Road, a similar arrangement with a shortened bridge and earth embankments, and a trench to take SH20A under Kirkbride Road. The latter was selected because it achieves traffic flow and safety objectives with least impact on the local community, maintains local road and cycleway levels and access, maintains visual connectivity in this region of schools, parks and residential properties, and limits adverse effects on local businesses by relatively low land take. Lowering SH20A also has the least visual and noise impact during construction. Preliminary design and budget development of the trench option commenced in 2014 and additional design requirements were added – provision for a 1.6m diameter watermain across the trench at Kirkbride Road and widening the trench to allow future construction of two light rail tracks within the trench. The final arrangement is a 455m long, 29.25m wide open trench with current provision for 6 traffic lanes and future provision for 4 traffic lanes and 8m wide two-track rail corridor. Kirkbride Road Bridge is 29.2m wide with 6 traffic lanes and two 3.5m wide shared footpaths. Design was complicated by the presence of deep peat and silty soils with potential for seismically induced liquefaction, a high water table, acidic soil, large stormwater retention volume and use of low axial stiffness steel screw piles to restrain the base slab against buoyancy. This paper covers the design issues that stem from these complications and the innovations in design and construction that resulted.

Keywords: Trench, Screw Pile, Diaphragm Wall, Base Slab

1. Introduction

State Highway 20A (SH20A) is the primary route to the Auckland International airport from the Auckland CBD and is a major gateway to and from New Zealand. In addition to its function as a primary link to the Auckland International Airport (AIA), SH20A also forms a strategic link between the western ring route (State Highway 20 & State Highway 16) and industrial zones in the vicinity of AIA and the greater Auckland area for the transportation of goods and services.

When completed in late 2017, Kirkbride Road Interchange project will provide a dual carriageway stretch of roadway with two lanes plus bus shoulder in each direction, and provision for future implementation of light rail, over a total length of 2.8 km between Bader Drive in the north and Landing Drive roundabout in the south.

The $180M project is procured by the New Zealand Transport Agency (NZTA) through an Alliance Agreement. It was chosen because it provides the ability to manage the complexity of the design solutions and construction challenges whilst maintaining traffic flow on the primary route to AIA, the busiest airport and the second largest cargo port in New Zealand. The alliance participants are Fletcher Construction Limited, Higgins Contractors Limited, NZTA and Beca Group Limited. URS New Zealand Limited were the independent peer reviewer for the structural and geotechnical design works.

1.1. Project Description

Increased visitor numbers and the further development of the AIA industrial zones will increase traffic volumes along SH20A and its connecting arterial roadways. To meet these increased traffic demands the SH20A alignment will be upgraded to motorway standard between Bader Drive in the north and the Airport roundabout in the south. Refer to figure 1 for the project location.

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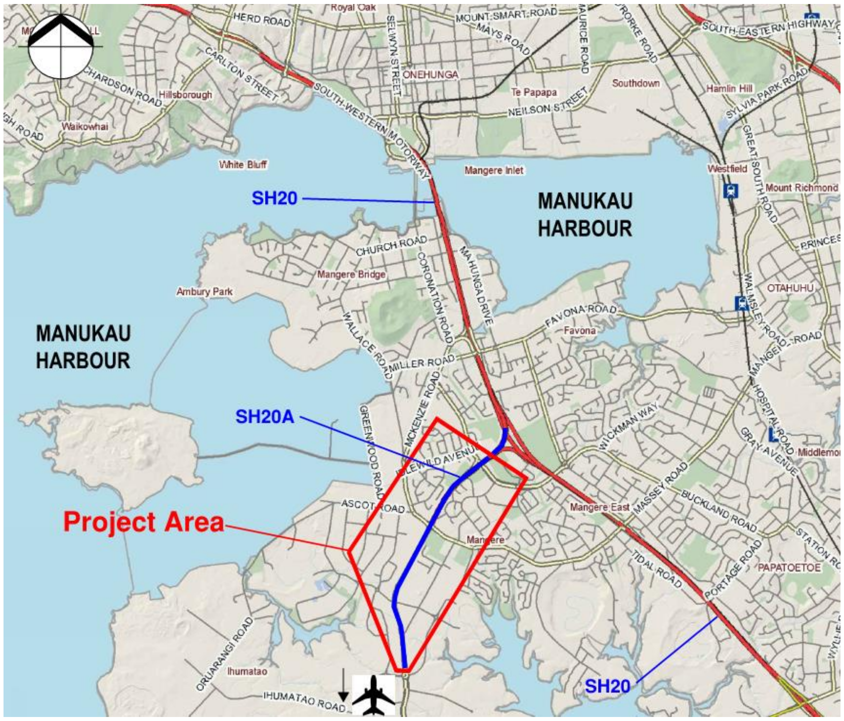

Figure 1: Project Location

The primary physical work involves the grade separation of the SH20A/Kirkbride Road intersection by trenching the SH20A alignment under Kirkbride Road, and the closure of the Montgomerie Road/SH20A intersection. This allows the existing traffic signals to be removed from SH20A in order to improve road user safety and improved journey times. The closure of the Montgomerie Road intersection will result in an increase of local road traffic using Kirkbride Road. Changes to the layout of Kirkbride Road and the adjacent local road intersections have been incorporated into the project to deal with the traffic flow increases.

The Kirkbride Road Interchange project is expected to deliver significant community benefits when completed. The trench achieves grade separation between the local Kirkbride road and SH20A. The removal of the existing intersection at Kirkbride road and SH20A significantly enhances safety and neighbourhood connectivity. This allows east/west cross movement for vehicles, pedestrian and cyclist within Mangere between the residential areas, schools, parks and commercial areas.

An urban design framework was developed for the project to reflect Auckland councils new Maori urban design principles. The project has adopted a significant amount of urban design treatment not only within the trench (prop art, bridge throw-screens, trench precast panels) but also outside the trench (noisewalls, landscaping, wetlands and swales). Input to the treatment of the urban design features were undertaken in consultation with the project's iwi advisor which contributes to cultural identity and awareness of the local community. The treatment enhances the visual appearance of the gateway corridor and gives it a sense of place.

A hallmark of any high performing team is the way in which the team quickly develops a shared understanding of the key drivers and constraints. The difficult balance between programme drivers and structural design limitations often become lost in translation between designer and constructor under traditional D&C arrangements. In an Alliance environment, this shared understanding and collaboration can lead to the development of innovative solution that deliver significant cost and programme savings.

2. Design

2.1. Description of the Trench Structure

The primary and current purpose of the trench is to take SH20A under Kirkbride Road, requiring sufficient width between side walls for two traffic lanes and a bus shoulder in each direction with concrete TL4 median and edge barriers. The 29.4m clear width was chosen to allow future reconfiguration for two light rail tracks on centreline of the trench and two traffic lanes in each direction. Figure 2 shows the typical cross section through the trench.

The trench structure descends between vertical retaining walls and passes 5m below the new Kirkbride Road Bridge and 1.6m diameter Hunua4 watermain. The retaining walls start as concrete L-wall supported by the trench base slab, then changes to embedded cantilever diaphragm wall then embedded and propped diaphragm wall as the trench deepens. Props are prestressed concrete beams spaced at 7m centres plus the Kirkbride Road bridge.

The diaphragm walls are constructed in 7m wide panels with PVC waterbar across vertical keyed construction joints. A continuous capping beam along the top of the diaphragm walls (D-walls) maintains panel alignment and draws lateral wall load to the props. The excavated face of the D-walls is concealed by architectural precast concrete panels.

The base of the trench has a full width concrete slab, restrained against buoyancy pressure by 360 steel screw-piles. The slab is continuous over the full length of the trench with buried abutments at both ends with movement joint and settlement slab to transition to traditional road pavement beyond.

Stormwater and groundwater that seeps through the trench structure is directed to the sides and middle of the trench, captured by catchpits and conveyed by concrete encased pipes to the 1800m^3 stormwater sump located below the base slab, from where it is pumped to surface treatment ponds. Road surfacing has a triple-layer design to prevent seepage contaminating the road surface by trapping it in a porous bottom layer from where it drains to sides of the trench.

Kirkbride Rd Bridge is 29.2m wide with three traffic lanes and 3.5m wide raised footpath in both directions and 1.6m wide raised median island. The bridge has a span of 30m, is simply supported on rubber bearings and longitudinally restrained by the trench walls with rubber bearing behind each of the twelve Super-T beams. F-shape TL5 concrete barriers support architectural anti-throw screens on both sides.

The Watercare watermain is a self-supporting concrete-lined steel pipe spanning 30.5m between the D-walls, located between Kirkbride Road bridge and a separate Super-T beam that together support a maintenance walkway.

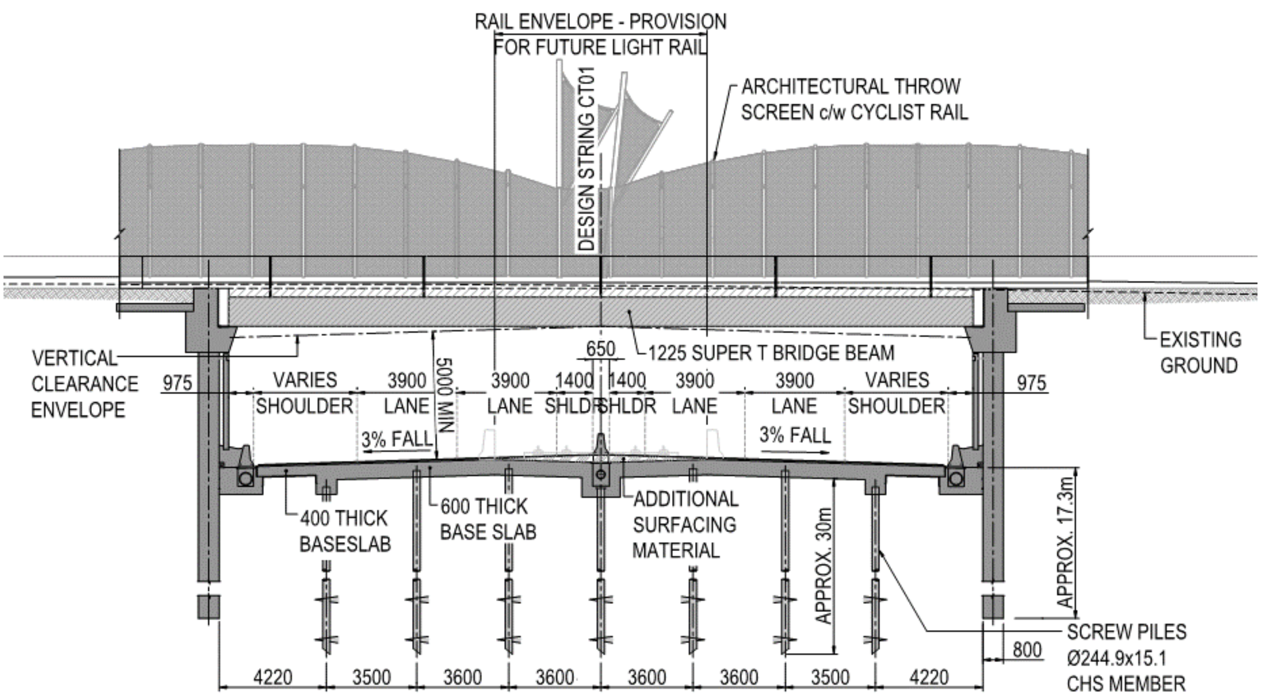

Figure 2: Cross Section at Kirkbride Road Bridge

2.2. Site Geology

The trench structure is located in ground consisting of fill overlying alluvium and includes a 4-7m thick peat layer located 2-3m below existing ground level. The soil layer beneath the peat consists of interbedded firm clayey silts and loose silty sands. Figure 3 shows the geological formation at the site.

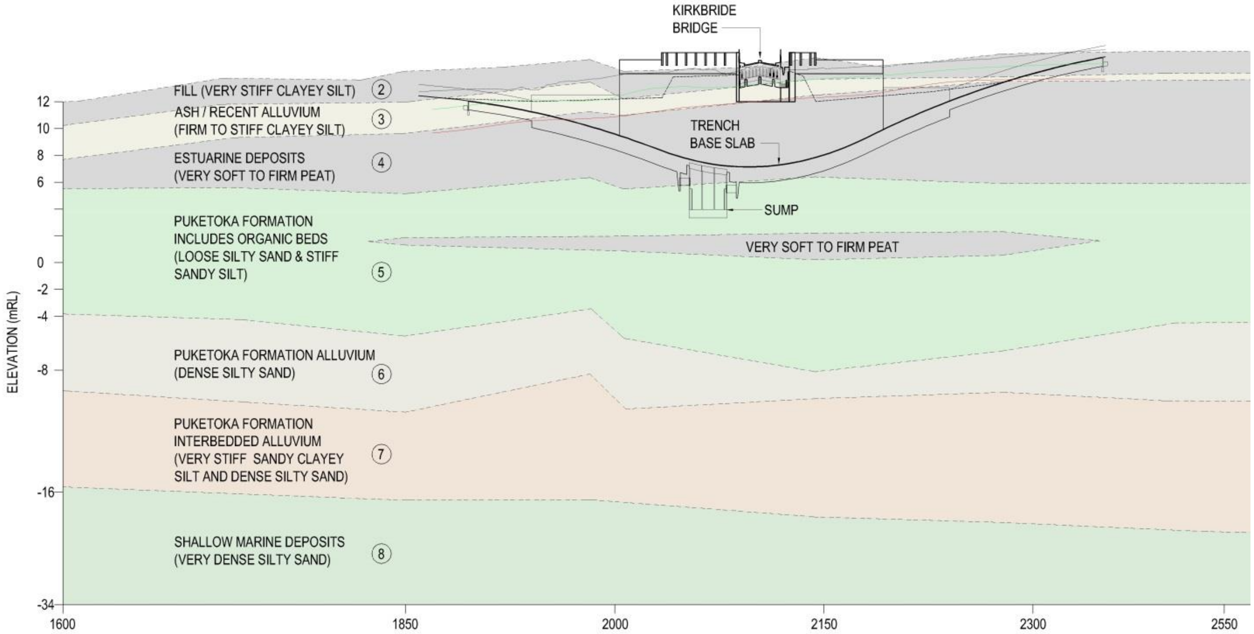

Figure 3: Site Geological Formation

2.3. Site Hydrogeology

The site is less than 14m above mean sea level and about 1.5km southeast of the Manukau Harbour and is bounded by incised tidal streams. Two perched water systems were identified, both suggesting groundwater flow to the north-northwest toward Manukau Harbour.

Winter groundwater level is typically 1.0m below existing ground level, dropping by about 1.5m in summer. This produces sustained hydrostatic pressure of about 65kPa below the lowest portion of base slab and 90kPa below the sump. Pressure at base slab level is near zero during construction but is expected to recover within 6-12 months of completion, thus requiring trench design to consider zero and full hydrostatic pressure for stability and strength.

2.4. Significant Design Issues

The trench is visually a simple structure consisting of base slab, walls and props, but design was complicated by several issues that required unusual design solutions:

2.4.1. Soil Liquefaction during Seismic Events

During ULS and MCE seismic events, liquefaction is expected to occur in semi-contiguous layers of loose sand which vary in lateral position and elevation. The upper layer is at an average depth of 10m below existing ground level and has an average thickness of about 1.5m. The lower layer is at an average depth of 17m below existing ground level and has an average thickness of about 1.0m. These layers are expected to consolidate following liquefaction, causing settlement of the soil layers above and generating negative skin friction on the trench walls, resulting in wall settlement of up to 100mm for the ULS seismic event and 180mm for the MCE seismic event.

It is assumed that liquefaction of the deeper sand layer will cause water pressure below the base slab to increase by about 30% to match total vertical ground pressure outside the trench walls. Wall settlement is expected to occur as hydrostatic pressure returns to normal after the event and the liquefied sand layers consolidate.

2.4.2. Groundwater retention

A hydrogeological investigation found that seepage of ground water into the trench must be kept below 5m^3 per day to prevent lowering of the water table and settlement of adjacent structures.

To this end structural crack width under SLS loads is limited to 0.2mm for direct tension and 0.3mm for flexure, based on NZS3106: Design of Concrete Structures for Storage of Liquids.

Although not required by NZS 3106 (1) The effect of restrained concrete shrinkage on flexural crack width was included by adopting the procedure recommended by Gilbert (2), which is slightly more conservative than pure flexural crack width calculation according to NZS 3101 (3). Design shrinkage was calculated using a basic final drying shrinkage of 750 microstrain and humidity of 80% during construction, becoming 90% after completion in recognition of the water pressure below the slab.

PVC waterbar and hydrophilic strips are installed in all base slab joints, but to further reduce risk of joint leakage, the base slab is constructed in approximately 30m x 10m panels to reduce the number of joints.

2.4.3. Ground Water Quality

Groundwater quality in the vicinity of the trench was tested and found to be slightly acidic with pH between 5.3 and 6.6, just falling into exposure category XA2 according to NZS 3101. According to NZ Transport Agency Bridge Manual and NZS 3101 this exposure requires use of concrete containing 30% fly ash, 370kg/m^3 minimum binder content, max 0.45 w/c ratio, 65mm cover and continuous water curing for 7 days.

Ground water data was also used to assess the exposure category for the steel piles according to AS 2159 (4), and found to be “mild to non-aggressive”. A 1.5mm sacrificial corrosion allowance was therefore applied to the external face of the piles over the full length, except that an allowance of 3.25mm was applied near the top of the piles in accordance with BD 42/00 (5).

2.4.4. Base Slab Piles

Tension piles are needed to restrain the base slab against buoyancy, and prevent floatation of the trench structure, but during and shortly after construction the piles are also needed to support slab gravity loads until hydrostatic pressure returns.

Steel screw piles were selected in preference to bored piles because of the rapid rate at which they can be installed and expected difficulty securing sufficient bored pile rigs to install the required number of piles within the available time.

The chosen screw pile has 245mm outside diameter shaft with 15mm wall thickness, and is typically anchored about 20m below base slab level into very dense silty sand. Reliable tensile capacity and axial stiffness of the piles was determined by carrying out 10 pile load tests and the data used to simultaneously develope the design basis for piles and base slab to suit the relatively low axial stiffness of the piles. Based on the test data and preliminary base slab design, the following pile performance criteria were adopted:

- Pile head deflection under a sustained tensile force of 1650kN must not exceed 55mm; this includes calculated elastic shaft elongation after deducting long-term corrosion allowance and expected soil deformation at the anchorage helix including long-term creep derived from the pile load test data.
- Variation in shaft length of adjacent piles should not exceed 10%

The pile spacing was calculated to limit design forces in the piles to the following loads:

- ULS tension (same for compression) = 1890kN
- SLS tension (same for compression) = 1350kN

These design forces require a minimum geotechnical ultimate capacity of 2700kN in tension or compression, which is demonstrated by calculation for every pile using pile installation torque. The same force was adopted for ULS design of the anchorage into the base slab, to minimise risk of brittle system failure. This required design of a novel cruciform plate anchorage device that relies on dowel action to transfer load to the concrete; the anchorage device is shown in Figure 4 (note that shear reinforcing around the anchorage is not show).

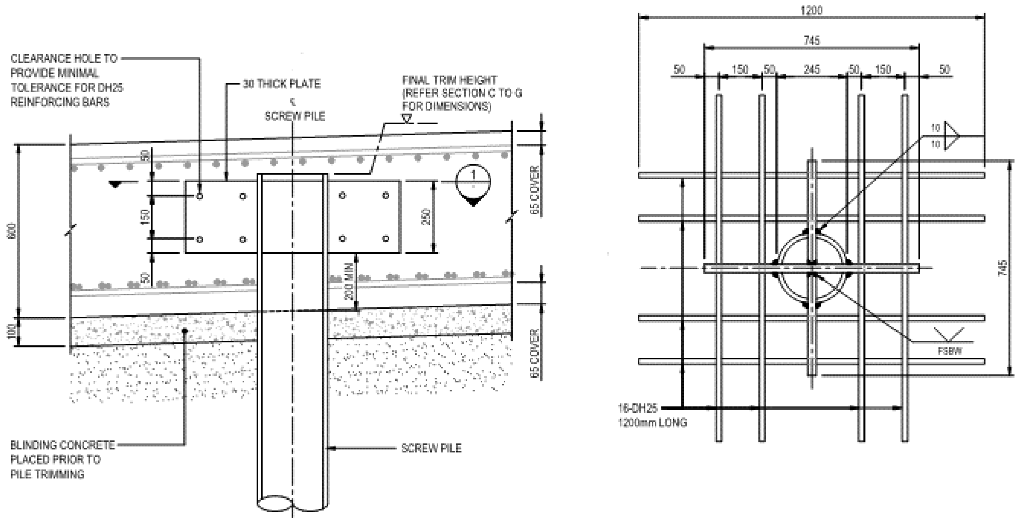

Figure 4: Base Slab Pile Anchorage

2.4.5. Base Slab Design for Pile Elongation and Wall settlement

Because of the low axial stiffness of the piles, the trench slab has to deflect upwards by 45mm to develop the SLS design tension in the piles. However vertical deflection is prevented at the D-wall connection which is required to be waterproof and transfer horizontal compression load between the wall and slab. Therefore pile load diminishes the closer it is to the sides of the trench, reducing efficiency.

Pre-tensioning the piles was considered as a potential solution to this problem, but found to be unworkable because preload force would rapidly dissipate as the soft peat below the slab is consolidated by the applied prestress force.

Therefore to maximise pile efficiency across the width of the trench, a flexible base slab design was adopted to increase the vertical deflection that takes place near the sides of the trench. This is generally 600mm thick with 2.5m wide, 400mm thick section on both sides of the trench and a partial pin-joint at the wall connection

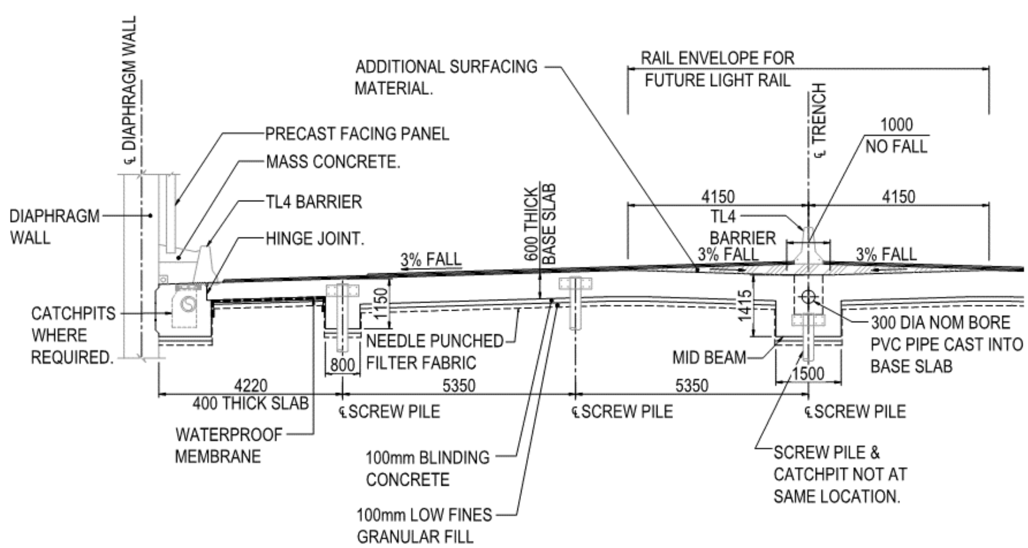

Figure 5: Base Slab Detail between Diaphragm Walls

This solution achieves reasonable pile efficiency across the width of the trench but results in high curvature demand in the 400mm thick edge slab, requiring high reinforcing content to comply with adopted crack width limits.

Upward deflection of the slab is expected to increase during a seismic event as groundwater pressure inside the trench increases for reasons explained earlier, but will return to normal as water pressure normalises. As that happens, the liquefied sand layers are expected to consolidate, causing the walls to settle relative to the base slab, permanently increasing flexural demand in the 400mm thick edge slab. Repairable damage is acceptable in this event.

2.4.6. Sump Structure Vertical Displacement

The problem of efficiently sharing load between piles is greater at the trench sump. Here too it is necessary to allow the piles to elongate under buoyancy induced tension, requiring the sump structure to be vertically independent of the trench walls. Difference in stiffness between the box-like sump structure and trench slab creates a problem at the connection between the two, particularly near the trench walls where slab curvature is greatest. This is resolved by providing a parallel slab arrangement on both sides of the sump; an upper flexible slab in the same plane as the trench base slab to connect the sump to the trench walls and allow the vertically movement similar to the trench slab, and a lower slab to seal the sump. This arrangement is shown in Figure 6.

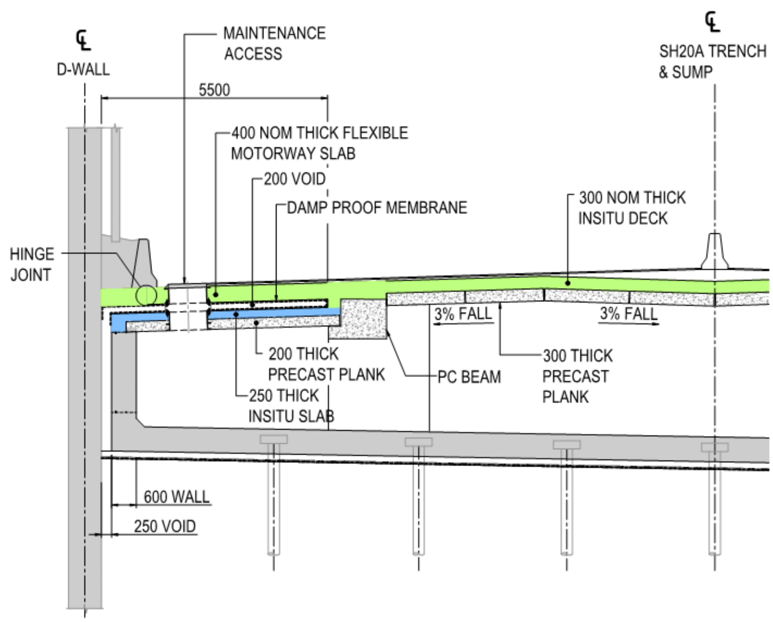

Figure 6: Part Trench showing Flexible Wall Connection

A 200mm horizontal void is provided between the parallel slabs to allow potential wall settlement to occur without the two slabs coming into contact, and a 250mm vertical gap is provided between the sump wall and the trench wall to allow relative vertical movement and ease construction.

Buoyancy pressure below the sump is about 35% higher than the adjoining trench slab, and higher pile density is needed below the sump to control relative deflection between the sump and adjoining slab.

Relative vertical movement between the sump and trench wall introduces a number of detailing difficulties, such as:

- Maintenance access from road level into the sump is provided through penetrations located within the parallel slab section, requiring special details that allow relative movement between the two slabs and remain waterproof under sustained hydrostatic pressure. This was achieved by installing a removable rubber-bellow type seal between the slabs, secured to steel frames cast into each slab. – see fig.??

- The sump is de-watered by stainless steel pipes into a wet-well constructed into the side of the trench, from where water is pumped to ground level for treatment. These pipes must also cope with relative vertical movement between the sump and wet-well and allow removal for maintenance.

2.4.7. Trench Props

The trench props are modified 900mm deep x 1100mm wide prestressed Single Hollow Core beams with an elastomeric bearings at each end to transmit compression between wall and prop. De-bonded linkage bars are provided to prevent a prop dropping in the event of severe damage through vehicle impact or earthquake shaking.

The props are designed for a factored axial compression of 4100kN from normal loads and an extreme event where an adjacent prop has been lost causing an unfactored axial load of 4300kN. Slenderness effects were considered according to NZS3101.

Because it would have been uneconomical to provide 6.0m vertical clearance to the props, a special pair of props at both ends of the propped section are arranged as a horizontal Vierendeel truss and designed for 500kN vehicle impact as required by the New Zealand Transport Agency Bridge Manual (6). All other props are designed for a 50kN impact.

3. Construction

3.1. Screw Piles

With reinforced concrete bored piles and screw piles considered to be the only real conceivable options for holding down the trench against the buoyancy forces it would be subjected to, the Alliance Target Outturn Cost (TOC) team was challenged with making a decision that would have significant bearing on both the design and construction of the trench structure.

There were many factors considered in the evaluation of the two options. A simple pros/cons comparison of each solution made the decision more obvious. The advantages in favour of screw piles far outweighed those identified for a bored solution, these included:

- Higher daily productivity – 4-6 screw piles/day against 3-4 bored piles/day
- Reduced plant spread and construction footprint. Screw piles consisted of 1-2 excavator mount rigs, grapple excavator and merlo. By comparison, a bored pile would have required 1-2 hydraulic rotary rigs, 1-2 crawler cranes, temporary steel casings, vibro drivers, spoil excavator, spoil trucks, concrete trucks, surge pile and a bentonite plant and distribution network.
- Screw pile method produces no spoil, no mess. Conversely, bored piles produce large volumes of spoil, including bentonite contamination and elevated hydrocarbon levels within the peat layer.
- Simplified treatment at pile breakdown – excess screw pile shaft is trimmed to cut-off level using plasma. Bored pile over-pour would require time consuming mechanical break-back techniques, often resulting in damage to starter reinforcing steel. The subsequent excavation of the trench also benefitted from this simplified break-back methodology, as it meant excess pile shaft could be quickly and easily trimmed down to provide unobstructed access for bulk excavation plant.

With known quality risks associated with procurement of steel from offshore sources, the Alliance developed a robust procurement process that incorporated rigorous quality assurance measures. In addition to the usual quality assurance testing requirements dictated by the project specification, the Alliance elected to send representatives to the production mill in China to inspect the facility and witness testing. Independent material testing was also undertaken in China under Alliance commission. In addition to overseas compliance testing, local testing was also undertaken on the steel tubes once landed in New Zealand. These stringent measures ensured that the Alliance had complete confidence in the integrity of the screw piles. This is a critical component of the structure that would become inaccessible once incorporated into the base slab.

3.2. Diaphragm Walls

Although well-developed methodologies already existed for D-wall construction, some aspects of the design presented opportunities for improvements to longstanding shortcomings. A simple and effective innovation on the project was the development of formwork sleeve protection for the D-walls which contributed to significant time savings in the concrete over-pour break down. Construction of the D-wall is undertaken with temporary guide walls and standard practice is to over pour the top of the D-walls followed by break down of the over pour to receive the capping beam reinforcement. The breakdown of the D-wall (typically done with rock breakers or similar) is a costly and time-consuming operation, often resulting in damage to reinforcing starter bars and ensuing remedial works. The formwork sleeve method simply involved installing timber plywood on either side of the rows of starter bars. When the concrete is still green a short width excavator bucket is used to remove the excess concrete. This was particularly effective in the removal of concrete from the inclined capping beams adjacent to the Kirkbride road bridge structure.

The construction of z-shaped D-walls at the wet well location is another innovative solution used on the project. This consisted of a D-wall section having three distinct sides. The fabrication of reinforcement cages to this profile to be installed in the excavation had the potential to present tolerance issues during installation. This was not the case and the z-shaped D-walls were installed successfully.

3.3. Bridge Staging

Another innovative solution that delivered significant cost and programme savings was the delay to the construction of the raised footpath on the south side of the bridge to accommodate three lanes of traffic on the bridge upon completion of the southern half D-wall. This refined structural mechanism facilitated earlier than anticipated D-Wall enabling works for the northern half of the bridge. This allowed an earlier start for the final phase of D-Wall and thus a significantly reduced stand down period for the resource intensive operation. Moreover, the earlier demobilisation of the D-Wall operation freed up much needed space on site with the releasing of large areas previously being occupied by the bentonite plant and reinforcing cage laydown areas.

This innovative solution was made possible through close collaboration between designers and constructors and thorough risk analysis to ensure shared understanding between the Alliance partners. This option had previously been considered during the early TOC phase but it was discounted due to structural limitations of the edge girder – due to it supporting a significant throwscreen option. This serves as a reminder to designer and constructors to regularly challenge and review decisions made at the design development stage. In this case, the urban design components of the bridge parapets had been value engineered out through a separate process, thereby reducing dead load demands on the edge girder.

By establishing a fertile medium for collaborative team thinking, the Alliance delivery team continued to develop innovative solutions that drew on the experience of not only the Alliance members, but also utilised resources from the home companies.

Perhaps the single biggest innovation gain was realised through a jointly developed temporary retention solution for the sump excavation, which allowed the removal of three of the highest loaded temporary steel props from the deepest excavation level in the trench. The original design have screw piles incorporated into the trench base slab - by adding supplementary temporary screw piles, the soldier pile and steel plate lagging temporary works design incorporated two key elements of the permanent works for a cost effective and efficient solution. Firstly, utilising a row of the already installed tension piles, deflection was able to be limited by adding shorter temporary piles at mid points between the permanent piles. With overlapping 16mm thick steel plate lagging vibro driven into the ground behind the soldier piles, the second element of the permanent works that contributed to limiting deflection was utilising the motorway base slab as tie back restraint for the tops of the screw piles. With limits placed on the deflection of the permanent screw piles due to their required stiffness capacity when working in tension, the high water table and weak silty sand layer that was being retained added further complexity to the solution. To meet deflection criteria, effective drainage of the temporary retaining wall was required, this was achieved through stone columns being constructed on the retained face with relief weep holes drilled through the steel lagging at regular spacings.

3.4. Temporary Traffic Staging

With the grade separation situated on a critical transport link between NZ’s largest international airport and NZ’s largest city, the Alliance identified during the TOC phase that maintaining traffic flows would be of utmost priority for key stakeholders in the region. Extensive traffic modelling of not only the Kirkbride Road / SH20a intersection, but also the surrounding intersections that are also incorporated within the project extents were conducted at an early stage. These models provided the basis for developing traffic management plans that provided the working space for construction activities whilst mitigating effects on road users. This included air passengers travelling through the site and the critical reliability of journey time that is principal to this group of end users. The selected scheme provided the optimal balance in terms of least disruption whilst maximising working space.

Another key constraint was the Alliance’s commitment to complete the portion of Hunua 4 pipeline that crosses the project main trench alignment in December 2015. With other portions of the large-scale Watercare project already underway within the Mangere area, this enabled one of the Alliance’s key stakeholders to fulfil their promise to the local community to ‘only dig once’ in the area. With the Hunua 4 watermain being supported on the bridge abutments which in turn sit on the D-walls, the order of construction sequencing had to be well coordinated around the intersection. The staging had to take into account the relocation of vehicular/pedestrian traffic, utilities and the construction works. This added to the complexity of working around this intersection.

4. Conclusions

This paper has briefly described the significant features of the Kirkbride Road Interchange trench that posed a number of design and construction challenges, and required innovative solutions to achieve a cost effective design that meets the project requirements.

5. Acknowledgement

The authors wish to thank the New Zealand Transport Agency for permission to publish this paper and also the contribution of the large team which have contributed to this project.

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