SUMMARY

Despite being an apparent simple structure, design of the Kirkbride Road Interchange trench 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 steel screw piles of low axial stiffness to restrain the base slab against buoyancy. The use of screw piles on an infrastructure trench project of this scale is the first of its kind in New Zealand. This paper covers the issues that stem from these complications and the innovations in design and construction that resulted.

INTRODUCTION

State Highway 20A (SH20A) is the primary route between Auckland International Airport (AIA) and the Auckland CBD. It also forms a strategic link between the western ring route (State Highway 20 and State Highway 16) and industrial zones in the vicinity of AIA and the greater Auckland area for the transportation of goods and services (see Figure 1).

The MHX Alliance was commissioned in 2010 to develop options for a new grade separated interchange. A trench to take SH20A under Kirkbride Road 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 causes the least visual and noise impact during construction.

Completed in August 2017, Kirkbride Road Interchange project provides a dual carriageway stretch of roadway with two lanes plus bus shoulder in each direction, with 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. The Alliance participants are Fletcher Construction Limited, Higgins Contractors Limited, NZTA and Beca Group Limited.
**DESIGN**

**Description of the Trench Structure**

The primary 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. However, 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 cross section through the trench underneath Kirkbride Road Bridge.

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

The D-walls are constructed in 7m wide panels with PVC waterbar across vertical keyed construction joints. A continuous capping beam along the top of the D-walls maintains panel alignment and concentrates 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 abutment beams at both ends with movement joint and settlement slab to transition to traditional road pavement beyond.

Stormwater and groundwater seepage is directed to the sides and middle of the trench, captured by catchpits and conveyed by concrete encased pipes to an 1800m³ stormwater sump located below the base slab. From there it is pumped to surface treatment ponds. To prevent seepage reaching the road surface, the road surfacing has a porous bottom layer below a sealing layer and OGPA surfacing, to convey seepage to the trench drainage.
Kirkbride Road Bridge is 29.2m wide, with three traffic lanes, 3.5m wide raised footpaths in both directions and a 1.6m wide raised median island. The bridge has a span of 30m, simply supported on rubber bearings and longitudinally restrained by the trench walls bearing against rubber bearings behind each of the twelve Super-T beams. An F-shape TL5 concrete barrier supports an architectural anti-throw screen on both edges.

The design also required provision of a watermain across the trench. The Watercare watermain is a 1.6m diameter 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 provides maintenance access.

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.
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.

Winter groundwater level is typically 1.0m below existing ground level, dropping by about 1.5m in summer, producing 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.

SCREW PILES

From the geological and hydrogeological investigations, it was clear to both design and construction that reinforced concrete bored piles and screw piles were the only real conceivable options for holding down the trench structure against buoyancy forces. In 2014, the Alliance Target Outturn Cost (TOC) team reached the conclusion that the advantages of screw pile construction significantly outweighed the selection of a bored pile solution, as shown in Table 1.

<table>
<thead>
<tr>
<th>Item</th>
<th>Pile Options</th>
<th>Advantage of Screw Piles</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Productivity</strong></td>
<td>Screw Piles: 4-6 screw piles installed per day</td>
<td>Higher daily productivity</td>
</tr>
<tr>
<td></td>
<td>Bored Piles: 3-4 bored piles installed per day</td>
<td></td>
</tr>
<tr>
<td><strong>Construction Footprint</strong></td>
<td>Screw Piles: 1-2 excavator mount rigs, grapple excavator and merlo</td>
<td>Reduced plant spread and construction footprint</td>
</tr>
<tr>
<td></td>
<td>Bored Piles: 1-2 hydraulic rotary rigs, 1-2 crawler cranes, temporary steel</td>
<td>Less impact on other construction works</td>
</tr>
<tr>
<td></td>
<td>casings, vibro drivers, spoil excavator, spoil trucks, concrete trucks,</td>
<td></td>
</tr>
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<td></td>
<td>surge pile and a bentonite plant and distribution network</td>
<td></td>
</tr>
<tr>
<td><strong>Environmental</strong></td>
<td>Screw Piles: No spoil, no mess</td>
<td>No impact on environment</td>
</tr>
<tr>
<td></td>
<td>Bored Piles: Large volumes of spoil produced, possibly bentonite contamination and elevated hydrocarbon levels within the peat layer</td>
<td></td>
</tr>
<tr>
<td><strong>Pile cap to base slab treatment</strong></td>
<td>Screw Piles: Excess screw pile shaft is cut-off at required level using plasma torch</td>
<td>Simplified tie-in to base slab treatment</td>
</tr>
<tr>
<td></td>
<td>Bored Piles: Time-consuming mechanical break-back techniques, often resulting in damage to starter reinforcing steel</td>
<td>No damage to reinforcement</td>
</tr>
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</table>
The use of screw piles on a large-scale infrastructure trench project is the first of its kind in New Zealand. Previously, screw piles have been used as wall anchors on trench structures, but not under a base slab and not in a project of this scale.

Selecting screw piles over bored piles had a significant bearing on the design and construction of the structural elements of the trench that interfaced with the screw piles, namely the trench base slab, the connection of the slab to the D-walls and the trench stormwater and retention sump.

Quality Assurance of Screw Piles

With possible quality risks procuring steel from offshore sources, the Alliance developed a procurement process that incorporated rigorous quality assurance measures. In addition to the usual quality assurance testing requirements, the Alliance sent 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. Local testing was also undertaken on the steel once landed in New Zealand. These stringent measures ensured that the Alliance has complete confidence in the integrity of the screw piles which are a critical component of the structure but becomes inaccessible once incorporated.

CHALLENGING DESIGN ISSUES

Design of this apparently simple structure, consisting of base slab, walls and props, was complicated by several issues that required unusual design solutions.

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 90mm for the ULS seismic event and 180mm for the MCE seismic event.

It is expected that liquefaction of the deeper sand layer will cause water pressure below the base slab and the sump structure, to increase by about 30% to match total vertical ground pressure outside the trench walls.

Groundwater Retention

A hydrogeological investigation found that seepage of ground water into the trench must be kept below 5m³ 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 recommendations in NZS3106: Design of Concrete Structures for Storage of Liquids.

Although not required by NZS3106, it is recognised that restrained shrinkage does increase flexural crack widths, and the effect of restrained concrete shrinkage was included by adopting the procedure recommended for the serviceability design of concrete structures (Gilbert, 2001). Design shrinkage was calculated using a basic final drying shrinkage of 750 microstrain.
and assuming humidity of 80% during construction, and 90% after completion in recognition of the water pressure below the slab.

PVC waterbar and hydrophilic strips were installed in all base slab joints and the base slab was constructed in approximately 30m wide x 10m long panels to reduce the number of joints.

**Groundwater 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/m³ minimum binder content, max 0.45 w/c ratio, 65mm cover and continuous water curing for 7 days. On the base slab, this was achieved by applying Transguard 4000 to the upper surface.

Groundwater data was also used to assess the exposure category for the steel piles according to AS2159:2009, 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.

**Structural Behaviour of the Screw Piles**

Tension piles are needed to restrain the base slab against hydrostatic pressure, and prevent flotation of the trench structure. During and shortly after construction the piles are also needed to support slab gravity loads before hydrostatic pressure builds up.

The chosen screw pile has 245mm outside diameter shaft with 15mm wall thickness, and is typically anchored about 30m 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 develop 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 or compression = 1890kN
- SLS tension or compression = 1350kN

These design forces require a minimum geotechnical ultimate capacity of 2700kN in tension or compression, which was demonstrated by calculation for every pile using installation torque. The same force was adopted for ULS design of the shaft 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 of the slab around the anchorage is not shown).
Base Slab Design for Pile Elongation and Wall Settlement

Because of the low axial stiffness of the piles, the trench slab is expected to deflect upwards by 45mm to develop the SLS design tension in the piles. However vertical deflection of the slab is prevented at the D-wall connection and pile loads diminish close to the wall, reducing efficiency.

Pre-tensioning the piles was considered to solve this problem, but not adopted because preload force would rapidly dissipate as the soft peat below the slab consolidated.

Therefore to maximise pile efficiency across the width of the trench, a flexible base slab design was adopted to increase the vertical deflection near the sides of the trench. The slab is generally 600mm thick with a 2.5m wide, 400mm thick section on both sides of the trench and a partial hinge at the wall connection (see Figure 5).

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, 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.

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 lengthen under buoyancy induced tension, requiring the sump structure to be vertically independent of the trench walls. The 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 was resolved by providing a parallel slab arrangement on the sides of the sump; a 400mm thick upper flexible slab (i.e. motorway slab) to connect the sump to the trench walls and allow vertical movement similar to the trench slab, and a lower 450mm thick slab (i.e. sump roof) to seal the sump. This arrangement is shown in Figure 6.

![Figure 6: Part Trench Showing Flexible Wall Connection](image)

A 200mm horizontal void between the parallel slabs allows potential wall settlement to occur without the two slabs coming into contact. 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 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 where remaining waterproof under sustained hydrostatic pressure. This was achieved by installing a removable waterproof rubber-bellow between the slabs, secured to steel frames cast into each slab.

- The sump is de-watered by stainless steel pipes into a wet-well constructed on one 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.

**Trench Props**

The trench props were modified 900mm deep x 1150mm wide prestressed Single Hollow Core beams with an elastomeric bearing at each end to transmit compression between wall and prop. The Hollow Core system allowed for reduced weight for ease of handling. De-bonded linkage bars were provided to prevent a prop dropping in the event of severe beam damage by vehicle impact or earthquake shaking.

The props were designed for a factored axial compression of 4100kN from normal loads and 4300kN in an extreme event where an adjacent prop was lost. 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 trench were connected to form a horizontal Vierendeel truss and designed for 500kN vehicle impact as required by the New Zealand Transport Agency Bridge Manual. All other props were designed for a 50kN impact.

**CHALLENGING CONSTRUCTION ISSUES**

Construction challenges required innovative solutions to be developed for the project

**Horizontal Void Former**

The formation of the horizontal void between the sump parallel slabs (see Figure 6) was made from 200mm thick cardboard waffle formers called SupaVOID (see Figure 7).

Initially, the construction team preferred forming the void with polystyrene, due to its ability to support the self-weight of the 400mm thick wet concrete of the upper flexible slab without risk of losing strength when it gets wet. However, polystyrene is an unfavourable material because it would be inaccessible to remove post pour.

Agreement with the design team was reached to use cardboard primarily due to its ability to leave a void once the cardboard comes into contact with water.

![Figure 7: SupaVOID (hidden) with waterproofing membrane on top of it](image)

**Access Manhole Rubber Bellows**

Seven access manholes with 610mm x 914mm clear opening provided in the shoulders of the upper flexible slab are the only access points into the sump when the trench is in operation.
Since the access manholes pass through the 200mm thick void, the typical steel circular manholes were not a viable solution. The manhole needed to cope with the relative movement of the upper flexible slab and the lower slab, generated by seasonal groundwater pressure and vertical settlement of the D-walls in a seismic event, and remain waterproof.

A rubber bellow system, bolted between the two slabs, was selected. Practical considerations of removing the rubber bellow for operation and maintenance in the future meant clamping brackets that support the bellows were detailed to be easily removable from inside the opening.

![Figure 8: Frame and rubber bellows in the lower slab (i.e. sump roof) (L). Frame at the upper flexible slab above the 200mm void (R).](image)

**CONCLUSION**

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.

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**REFERENCES**


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