Proceedings of the 11th ANZ Young Geotechnical Professionals Conference – 11YGPC
Queenstown, New Zealand
25 – 28 October 2016

SH20A to Airport: New Zealand's First Project adopting Screw Piles and Diaphragm Walls as a Trench Combination

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ABSTRACT

State Highway 20A (SH20A) is the primary route to and from Auckland's international and domestic airports and forms a strategic link between State Highway 20 (SH20), State Highway 16 (SH16), the airport, and the greater Auckland area. The SH20A to Airport project comprises a new motorway interchange that separates the local and motorway traffic. SH20A is to be trenched below Kirkbride Road. The trench is approx. 450 m long and 30 m wide, retaining up to 7.2 m height of soil, and accommodating two carriageways with allowance for a future rail corridor. Diaphragm walls (D-walls) and screw piles are adopted as the trench supports, a combination never before used in New Zealand. This paper focuses on the trenching works, outlining the geotechnical design aspects for the trench, and the challenges through the design and construction to-date.

Keywords: Screw Piles, Diaphragm Wall, Trench, SH20A, Kirkbride Road, MHX

1 INTRODUCTION

SH20A is the primary route to and from the airport and forms a strategic link between SH20, SH16, Auckland's international and domestic airports, and the greater Auckland area. The main physical work for this project includes SH20A and Kirkbride Road widening, and grade separation of the SH20A/Kirkbride Road intersection by trenching the SH20A alignment under Kirkbride Road. The trench is some 450 m long and 30 m wide, retaining up to 7.2 m height of soil, and accommodating two carriageways and a future rail corridor. D-walls have been adopted for retaining the trench sides, and screw piles are adopted for anchoring the trench base slab down and resisting hydrostatic uplift pressures on the slab. The SH20A to Airport project is New Zealand's first project using the combination of D-walls and screw piles.

This project was initiated in 2010 with detail design commenced in May 2014. The construction started in July 2015 with project completion in 2017 by the MHX Kirkbride Alliance, consisting of alliance partners the New Zealand Transport Agency (the Transport Agency), Beca Ltd, Fletcher Construction Company (FCC) Ltd, and Higgins Group Holding Ltd. At the time of preparing this paper, the majority of the D-wall panels and screw piles have been constructed and installed, and the trench excavation between the D-walls is about to begin.

2 SITE CONDITION

2.1 Geology

The site is underlain by Puketoka Formation of the Tauranga Group, comprising of mostly light grey to orange-brown pumiceous mud, sand and gravel with black, muddy peat and lignite. The northern side of the site is underlain by undifferentiated organic-rich alluvium (Kermode, 1992). The nearest active fault is the Waiora North fault, some 30 km to the east. It is an active normal fault, dipping 60 – 70 ° to the west or northwest (Wise, 1999) with an apparent vertical slip rate of about 0.1 mm/year. The fault is a potential source for large earthquakes.

2.2 Ground and Groundwater Conditions

Between 2010 and 2015, field investigations of 28 CPTs, 27 machine boreholes, and 47 hand augers were undertaken and tested to up to 45.2 mbgl. The typical ground profile interpreted is summarised in Table 1.

The trench structure is located within the upper 4 layers identified in Table 1 with the top of the base slab positioned within soft Peat (L4), the toe of the D-wall founded on Puketoka Formation soils (varies from L5 to L7), and the toe of the screw piles founded on dense to very dense Shallow Marine Deposits (L8). Figure 1 shows the exposed Ash (L3) and Peat (L4) during the earthworks excavation.

Table 1. Typical Ground Profile

Layer Unit		Soil Description	Depth to Top of Layer (m)
1	TOPSOIL		0 (RL14.5m)
2	FILL	Stiff to very stiff, clayey SILT, minor gravel, yellow brown mottled grey, moist, highly plastic	0.1 – 0.15
3	ASH/Recent Alluvium	Loose sandy SILT; Soft to firm pumiceous sandy/clayey SILT and silty CLAY	1.5 – 2.25
4	Estuarine Deposits	Soft amorphous PEAT; very soft to firm organic SILT with minor clay, minor fibrous organics and wood pieces	2.0 – 4.4
5a[a]			8.0 – 16.2
5	Puketoka Formation	Alluvium including organics beds – Loose to medium dense silty/clayey SAND and sandy SILT, pumiceous; firm to stiff SILT-CLAY	8.2 – 9.2
6		Alluvium – Medium dense to very dense silty SAND	17.9 – 22
7		Interbedded Alluvium – very stiff to hard pumiceous sandy/clayey SILT; dense to very dense silty SAND	23.5 – 24.5
8	Shallow Marine Deposits[b]	Dense to very dense silty SAND; hard sandy SILT; trace shells	30.8 – 33.6

[a] Interbedded Peat within Layer 5 typically across the site.
[b] The soil was initially understood as “Kaawa Formation”. However, the geological unit has been updated as “Shallow Marine Deposits” to reflect the fact of the absence of microfossils based on a paleontology studies of the “Kaawa Formation”.

Figure 1. Earthworks Excavation Exposing Ash (L3) and Peat (L4)

Three distinct water levels are observed from monitoring: a shallow water level in Peat ranging from RL14.5 m to RL 8.5 m from south to north, an intermediate water level some 2 m to 5 m lower in the Puketoka Formation, and a deeper water level at approx. RL 5 m in the underlying sands. For design purposes, these aquifers have been assumed as all connected to produce a hydrostatic groundwater table level close to the surface.

It is understood that short-term groundwater drawdown (due to construction) would not extend beyond the construction zone. For long-term groundwater drawdown, it has been conservatively estimated up to 5 m could occur immediately adjacent to the trench, rapidly reducing to less than 2 m within 10 m to 15 m of the trench walls, and reducing to 0.25 m around 65 m from the trench walls. However, with the removal of soil from within and adjacent to the trench, static settlements due to groundwater drawdown are expected to be negligible long term. Short term drawdown measured to date has been small and less than 60% of the estimated 50mm.

2.3 Seismicity

The site subsoil class is Class E, in accordance with NZS1170.5 (Standards New Zealand, 2004). The design Peak Ground Acceleration (PGA) for the ULS case is 0.21 g for 2500 years return period with a moment magnitude of 5.8 determined by New Zealand Bridge Manual 3rd Edition Amendment 0 (New Zealand Transport Agency, 2013). A PGA 1.5 times that adopted for the ULS case is adopted for the

Maximum Considered Event (MCE) case in accordance with the Bridge Manual, and the return period of 10,000 years was probabilistically derived from a Site Specific Seismic Hazard Assessment (MHX Kirkbride Alliance, 2015). This required and received approval from the Transport Agency Scope and standards for specific use on this project.

3 TRENCH DESIGN OVERVIEW

3.1 D-Walls

Concrete D-walls are the main trench retaining structures for the project, to limit the excavation footprint and the need for groundwater dewatering during wall excavation. Bentonite slurry is utilised to maintain the stability of the existing ground during the excavation. When the excavation is complete, a prefabricated reinforcement cage is inserted into the excavated D-wall location. Concrete is then tremmied into the excavation, progressively displacing the bentonite and forming the D-wall.

Each D-wall panel is 800 mm thick and 7 m wide, retaining a soil height of 2.5 m to 7.2 m. Most of the D-walls are to be permanently propped by concrete beams. Temporary props are to be utilised during trench excavation before the base slab is cast. The D-wall throughout the trench is categorised into five types based on the structural setting (refer to Figure 2):

- *Type 1 D-Wall* – propped by bridge structures, with toe at RL -10 m, retaining approx. 7.2 m height of soil
- *Type 2 D-Wall* – propped by concrete beams, with toe at RL -10 m, retaining approx. 6 m height of soil
- *Type 3 D-Wall* – propped by concrete beams, with stormwater sump below the road base slab, toe at RL -10 m, and retaining approx. 6.5 m height of soil
- *Type 4 D-Wall* – propped by concrete beams, with shallow toe at RL -3.5 m to RL -2.8 m and retaining approx. 6.5 m height of soil
- *Type 5 D-Wall* – cantilever, with toe at RL -1.4 m to -3 m and maximum retained height of 4 m.

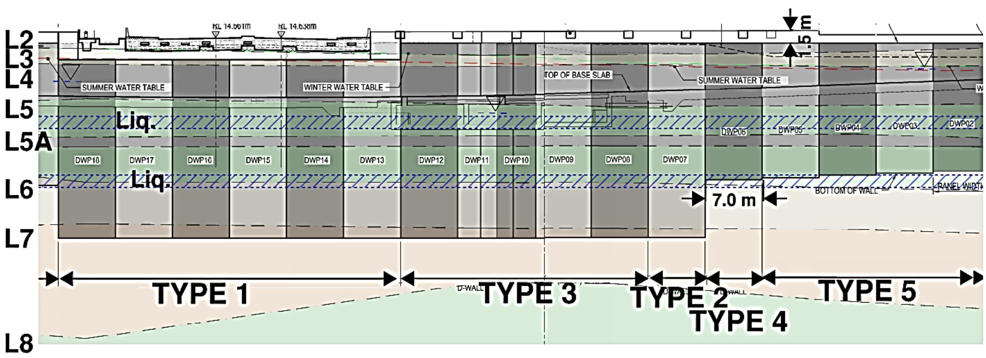


Figure 2. D-Wall Long Section (partial) and Ground Profile (screw pile not shown for clarity)

3.1.1 Liquefaction Analysis

Liquefaction potential and induced settlement have been assessed using the methods of Idriss and Boulanger (2008) and Zhang et al. (Zhang, Brachman and Robertson, 2002) in accordance with the Bridge Manual. Isolated part of the interbedded loose to medium dense silty SAND (L5 in Table 1 and Figure 1), is expected to liquefy in the ULS event at two horizons. The upper horizon is centred at an average of RL 4 m with an average thickness of approx. 1.5 m and the lower horizon is centred at an average of RL -3 m, also with an average thickness of about 1.5 m.

In an MCE event, liquefaction is also expected to occur broadly at two horizons. The upper horizon is as for the ULS event but the lower horizon is centred about a metre lower at an average of RL -4 m and has an average thickness of about 3 m.

As shown in Figure 2, Type 1, 2 and 3 walls are founded in very stiff to hard interbedded silty SAND (L7), while Type 4 and 5 walls are founded approximately at the interface of loose to medium dense sandy SILT (L5) and medium dense to very dense silty SAND (L6). The potential liquefaction induced settlements have been estimated and shown in Table 2.

Table 2. Estimated Liquefaction Induced Settlements under ULS and (MCE) cases

D-Wall Settlements (in mm)	Type 1	Type 2	Type 3	Type 4	Type 5
Above toe of D-wall	95[a] (150[a])			85[b] (100[b])	50-85[b] (60-100[b])
Below toe of D-wall	5 (30)			15 (80)	15-50 (80-120)
Adopted Wall Settl. for Slab Design	5-50 (10-90)			15-100 (20-180)	

[a] Up to half of the estimated settlements could induce settlement of the wall due to negative skin friction.
[b] Possible settlements due to negative skin friction
[c] Settlement under MCE case are shown in the brackets

3.1.2 D-Wall Analysis

Geotechnical analysis of the D-wall was carried out using Strength Reduction and 2-D Finite Element Methods in the computer program WALLAP Ver 6.05 by Geosolve Ltd. WALLAP was used to assess bending and shear demands, horizontal wall displacements, prop reactions (for both permanent and temporary props), and base slab propping reactions. The analysis took into account each stage of construction, long term static, seismic ULS, liquefaction, sequential seismic ULS (i.e. 2 x ULS case that was tested against collapse rather than controlling design), and MCE Cases. The program Plaxis 2D was used to cross-check the magnitude of demands and displacements determined by WALLAP.

The demands obtained from WALLAP were then adopted by Spacegass (a structural analysis program) to analyse the demands at the joints of the D-wall and the base slab, and of the base slab and screw piles.

3.2 Screw Piles

Screw piles are the 'Design and Construct' component of the project carried out by Piletech Ltd, a business unit of FCC. The piles have been designed to support the trench base slab as well as provide the hydrostatic pressure resistance. To date, more than half of the total 360 screw piles have been installed. Figure 3(a) shows the installation of screw piles on site. Piles have been installed with a mandrel from the existing ground level to some 39 m depth. The mandrel acts like a pile extension allowing the top of the pile to be pushed to the pile cut-off level (approx. the base slab level) from the existing ground level. The bore within the mandrel has been backfilled with sand before the mandrel was removed.

The geotechnical screw pile design, including pile helices configuration and foundation verification requirements, were carried out based on the test pile investigations in accordance with AS2159.

3.2.1 Test piles

Three piles with different number and configurations of helices were installed on site and full scale load tested in tension in 2014. The configuration of helices of Test Pile 1 (TP1) was selected with an upper helix of 650 mm diameter and lower helix of 500 mm diameter at 1.6 m spacing. It was founded at 39 mbgl and reached the ultimate test load of 2700 kN without geotechnical failure. A further 7 test piles (including one that failed to reach the geotechnical ultimate capacity of 2700 kN) were installed and tested also in tension in 2015 based on the same configuration as TP1. These 6 pile tests have confirmed a minimum tensile ultimate geotechnical capacity of 2700 kN. Piletech advised that the compressive capacity would not be less than its tensile capacity based on dense to very dense nature of the Shallow Marine Deposits. Pile demands are also mitigated by distribution of compressive loads to the base slab sharing some vertical loads.

Soil creep at the toe of the pile (100 years) has been estimated from 7 of the test piles. Deflections were obtained under a constant load of 1650 kN over a 12-hour holding period, and extrapolated and extended conservatively up to 100 years. Deflections due to soil creep have been combined with initial deflections for long term pile toe deflections. The total deflections have been tabulated in Table 3, and the 95th percentile has been considered in the base slab design.

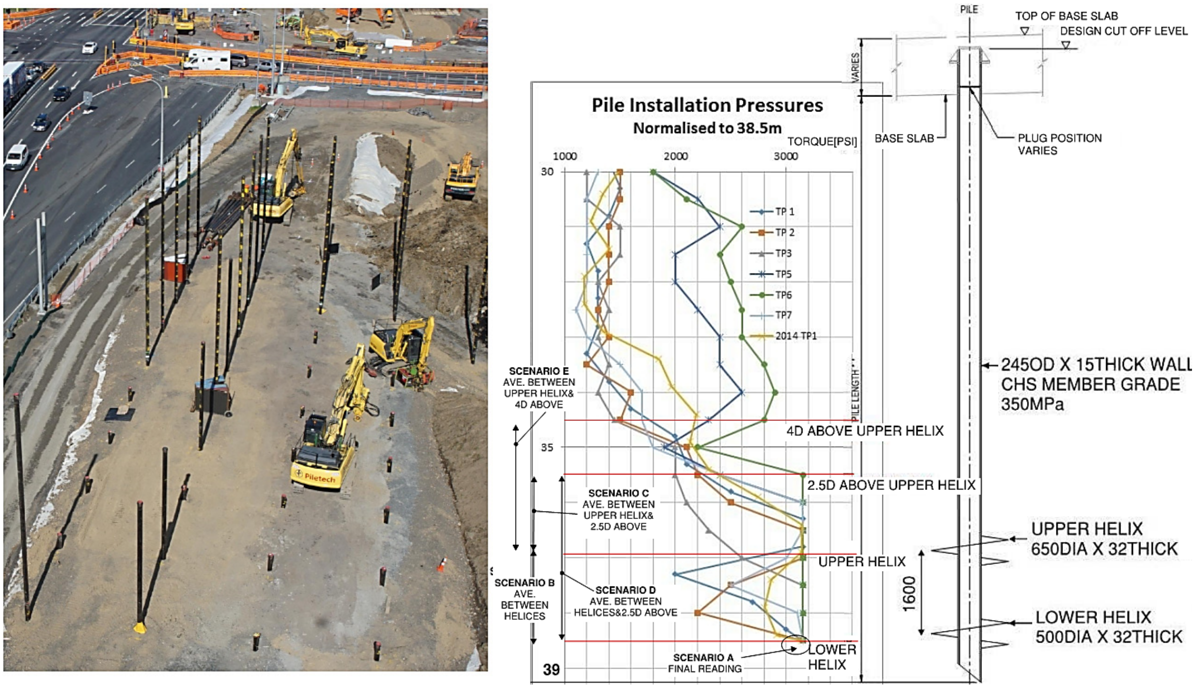
Table 3. Summary of Pile Toe Creep from Test Piles

Test Pile	Initial Deflection[a] A (mm)	Short Term Creep[b] B (mm)	Long Term Creep C (mm)	Total Creep B+C (mm)	Total Deflection A+B+C (mm)
2014 TP1	11	3.25	5.75	9	20
2014 TP2	11	3.25	7.35	10.6	21.6
2015 TP1	6.8	4.6	7.9	12.5	19.3
2015 TP2	4.0	5.5	7.35	12.9	16.85
2015 TP3	10.5	3.1	5.5	8.6	19.1
2015 TP5	13.6	4.3	7.5	11.8	25.4
2015 TP6	18.9	3.0	8.25	11.25	30.15
				95th Percentile	28.7

[a] Load zero to 1650kN (P_s, in accordance with AS2159)
[b] Creep tested under a constant load of 1650kN for 12 hours
[c] Creep estimated based on the tested short term creep, for 12 hours to 100 years under a constant load of 1650kN

3.2.2 Pile Foundation Verification

CPT and SPT test results have been compared against the torque obtained from the test pile installation. Based on the comparison, sandy soil of SPT N = 50+ would generate a torque of some 2000 psi during the screw pile installation. During each pile installation, 5 test scenarios (A to E as shown in Figure 3(b)) are to be checked against target pressures. Our screw pile design assumption can then be validated. Scenario A is to test the geotechnical compressive capacity, while Scenarios B to E are to be considered for the tensile capacity.



(a) (b)

Figure 3. (a) Screw Pile Installation in April 2016; and (b) Test Pile Installation Pressures

3.3 Base Slab

The concrete reinforced base slab supported by screw piles is typically 600 mm thick between the trench D-walls. The base slab is to be connected to an edge beam on both sides of the trench. The edge beam is 1.2 m thick and 1.6 m wide accommodating the drainage pipe and catchpits along the trench. Between the edge beam and base slab, a ‘soft slab’ of 400 mm thick and 2 m wide is to be ‘pin-connected’ to the edge beam and provide the load transition between the base slab and D-wall.

4 CHALLENGES

4.1 Differential Settlement between D-Wall and Base Slab

Some of the D-walls are founded on Puketoka silty SAND and sandy SILT (L5) which are potentially liquefiable. The base slab is supported by screw piles founded on very dense Shallow Marine Deposits (L8) where liquefaction is not expected. In the seismic event especially in combination with hydrostatic uplift pressures, the D-wall is expected to settle. The 'hinge' (described in Section 3.3) between the 'soft slab' and edge beam would enable movement to prevent the main slab body and the nearest screw piles from being over-stressed. This 'soft slab' would provide more flexibility to accommodate the load demand, therefore reducing the connection demand at the joints between the D-wall and edge beam. After a major earthquake, the slab at 2 m to 3 m depth adjacent to the D-wall is expected to need to be repaired.

4.2 Use of Screw Piles

The available CPT results have been used for initial assessment to establish top of soil with an SPT N = 50+. However, linear interpolation between CPTs and BHs did not account for variations in ground conditions encountered during pile installation. So far, 44 % of the installed piles are up to 10 % longer than expected, and 4.2 % of the "longer piles" are founded in locally weaker soils than expected, which require further structural checks of the impact of the resulting reduction in ultimate capacity and pile vertical stiffness.

The increase in pile length results in an increase in the elastic displacement of the pile. The increase in elastic displacement from differential founding levels of the screw piles could over-stress the base slab. Therefore, before the mandrel is disengaged, reviewing the piling record and providing the feedback to the structural team promptly is important. The structural team then review the change and identify appropriate remedial options, such as using additional base slab reinforcement to redistribute load from the soft piles to the surrounding piles to reduce the impact on the base slab.

4.3 Cost Implication and Construction Sequences

The cost of fabricating temporary props is in the order of $150,000 each. Therefore prior to construction a value engineering exercise was undertaken to potentially reduce the required number of temporary props. This included examining the construction sequence and identifying potential impacts on the design, with new prop arrangements and rationalising the construction sequence in terms of temporary prop installation and removal staging in relation to the trench excavation. Temporary prop optimisation design has been undertaken deliberately with the involvement of Constructors, Structural and Geotechnical Designers working collaboratively. To date the outcome has been cost savings in the vicinity of $500,000 to $750,000 by lessening the number of temporary props to be fabricated.

5 CONCLUSION

The SH20A to Airport project has been successfully designed and is currently under construction. The trench has been designed with the innovations of using a D-wall and screw pile combination as the trench support, and 'soft slabs' to accommodate large load demands due to potential liquefaction induced settlements between the D-wall and base slab. The collaborative environment of an effective Alliance has allowed an innovative construction methodology, rationalised design (for example temporary prop design), rapid response to unexpected ground conditions and consequent project cost savings.

For future screw pile and base slab design, increasing the tolerance for pile lengthening shall be considered for encountering potentially significant variations in ground conditions.

6 ACKNOWLEDGEMENTS

The author would like to acknowledge the Transport Agency and MHX Kirkbride Alliance for permission to publish this paper. The author would also like to thank Sian France and Grant Newby of Beca Ltd for their help throughout the project and the review of this paper.

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