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Application of micropiles for resisting hydrostatic uplift forces in deep trench

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ABSTRACT: Micropiles were successfully used to provide resistance to hydrostatic uplift forces and to reduce bending moments in the rail base slab due to shallow groundwater level for a water-tight 'tanked' train underpass at Union Station in Melbourne. Instead of installing conventional large bored piles to provide tension capacity to hold down the base slab, micropiles were used to provide a faster and cost effective construction program, and an environmentally sustainable solution which successfully minimised the shut-down time of the operating train line during construction, and reduced environmental and social impact to a densely populated area. The micropiles were not 'post-tensioned', and they were fully grouted along the pile length, thus a 'passive' system that would be mobilised with the base slab. The micropiles were up to 12m long comprised 57mm diameter high tensile strength threaded steel bar installed in 324mm diameter drilled hole. A possible uplift failure mechanism involving cone pullout of soil/rock mass was also assessed using finite element analysis. Analysis of a single micropile indicated a mechanism comprising shallow cone of soil/rock mass and shaft failure below the cone rather than the pull-out cone extending to the toe of the micropiles. However, in a group of micropiles with closer spacing, formation of a block of overlapping deeper cones was observed. This paper presents the details of the design of micropiles, results of the pull-out tests carried out and the finite element analysis.

KEYWORDS: Geotechnical Challenges, Micropiles, Uplift Tension, Excavation.

1 INTRODUCTION

As part of the train and road grade separation project called Level Crossing Removal Project (LXRP) in Melbourne, a new rail underpass 'trench' beneath Union Road and Mont Albert Road was constructed. The existing Surrey Hills and Mont Albert railway stations at both road and rail intersections were demolished and replaced by a new railway station called Union Station located between the two existing stations. Named UMA (i.e. Union Mont Albert), the project was delivered by the South East Program Alliance (SEPA). Golder Associates (now WSP) was engaged to provide geotechnical consultancy services to Laing O'Rourke, a member of SEPA.

A watertight permanent retention system for the proposed rail underpass was adopted by SEPA comprising bored soldier piles with shotcrete infill panels and base slab. Micropiles were installed beneath the base slab to primarily resist groundwater uplift pressures.

This paper discusses the design approach adopting the micropiles to resist hydrostatic uplift pressures imposed on the base slab instead of using the conventional large bored piles as tension piles. The major advantages of micropiles is that they can be quickly installed at the excavation base level and under limited headroom within the trench to reduce the shut-down time of the operating train line and roads, and to minimise environmental disturbance and financial impact to the densely populated residential and commercial areas, respectively.

2 PROJECT CONTEXT

2.1 *General subsurface and groundwater conditions*

The ground conditions within the proposed rail underpass alignment predominantly consist of an "old water course' of Quaternary age alluvium overlying residual soil derived from the complete weathering of siltstone, which is in turn underlain by highly weathered siltstone of the Silurian age Anderson Creek Formation. The three dimensional (3D) geological model showed the deepest alluvium at approximately 10.5m depth overlying highly weathered siltstone profile (see Figure 1).

The siltstone rock mass is characterised by its high-persistence bedding planes. The bedding of the siltstone is typically planar and smooth, with partings closely to widely spaced (60mm to 1000mm) and with variable amounts of ferruginous (iron) staining or clay (derived from decomposed siltstone) on defects. Two joint sets were typically observed within the rock mass with the joints being generally planar, with tight, rough, and clean surfaces. The joints were generally of low persistence (1m to 4m) and are roughly oriented orthogonal to bedding.

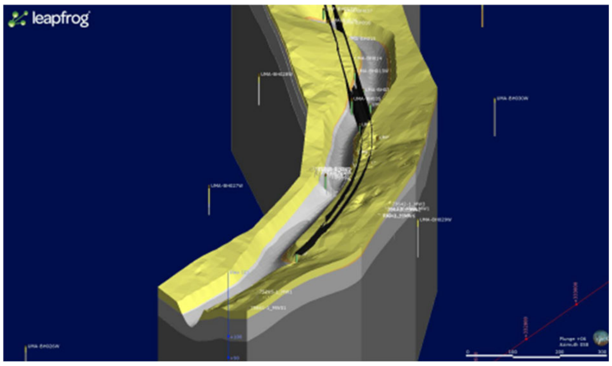

Figure 1. 3D geological model showing alignment and lateral extent of the old water course / alluvium channel underlain by weathered siltstone (yellow and grey).

The groundwater levels observed along the trench vary both spatially and temporally. The existing groundwater table appeared to be shallowest (less than approximately 1m deep) at the 'old water course' area, and deepest (up to approximately 9m deep) in the higher elevation upstream along the alignment.

2.2 *Retaining wall trench and base slab with micropiles*

The entire underpass structure was designed as a watertight 'tanked' structure. As the groundwater level was expected to be above the rail grade in most parts of the cutting, the underpass structure was required to resist the hydrostatic forces. As the base slab was relatively wide, 3 rows of micropiles were required (see Figure 2) to reduce the bending moments and shear forces in the slab and to provide required uplift resistance. The micropiles were not 'post-tensioned', and they were fully grouted along the pile length, thus are a 'passive' system that would be mobilised.

The bulk excavation of open cut and construction of the underpass was planned to be carried out during occupation of the train tracks (i.e., trains ceased to operate and intersecting roads closed). Due to the limited time duration of occupation for construction of the underpass, the retaining wall types preferred by SEPA were cantilevered soldier bored pile walls.

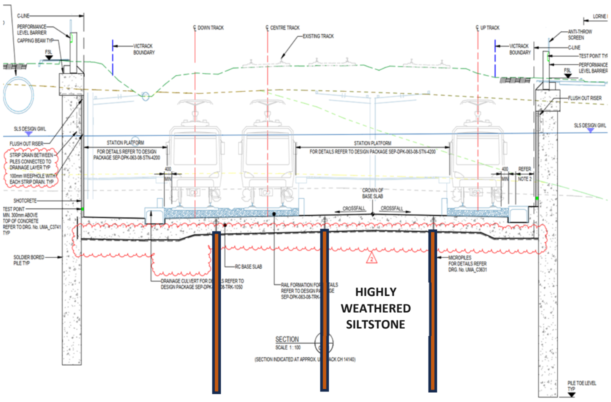

Figure 2. Cross section showing bored pile retaining walls connected with base slab and held down by micropiles

3 MICROPILES

Each micropile comprised a high tensile steel threaded bar and was grouted in a 324mm diameter hole over its full length as indicated in Figure 3. The micropiles were designed to satisfy durability requirements over its design life.

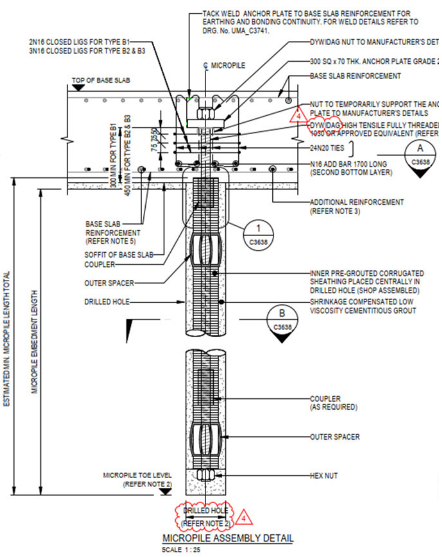

Figure 3. Typical details of micropiles (corrosion protection with corrugated PVC sheathing)

The design of the micropiles comprised the key assessments of (1) Geotechnical bond strength of micropile (i.e., the pile shaft resistance); (2) Possible "cone pull-out" from rock and soil mass, and (3) Structural capacity of steel bar.

This paper primarily discusses the assessment of the bond strength and cone pull out mechanism.

The structural capacity was assessed for the diameter of the bar and the steel strength. The bar was designed for corrosion resistance by using a 100mm corrugated PVC sheathing arrangement for long term durability. The maximum length of micropile was limited to 12m due to logistical constraints and the potential for "unzipping" effect. As the tensile resistance in a micropile will be mobilised gradually from the top, in a longer micropile, the upper part could go into residual (post peak resistance) and make the extra length less effective.

3.1 *Bond Strength*

For the assessment of ultimate geotechnical bond strength of micropiles, the unit shaft resistance values for various embedment materials as set out in Table 1 were adopted. Pre-construction 'pull-out' testing of sacrificial micropiles has proven a higher bond strength could be achieved, but the verified bond strengths were limited by jacking capacity rather than bond failure.

Table 1. Verified design parameters for assessment of geotechnical bond strength of micropiles

Embedment material	Su^ (kPa) or [UCS]# (MPa)	Ultimate unit shaft resistance (kPa)
Alluvium	75	50
Extremely Weathered (XW) Siltstone	[120]	80
Extremely to Highly Weathered (XW-HW) Siltstone	[600]	145*
Highly Weathered (HW) Siltstone	[1.0]	435*

*design values confirmed by 'pull-out' testing; ^undrained shear strength; #Unconfined Compressive Strength

The geotechnical strength (resistance) of the micropiles in ultimate limit state (ULS) was assessed in accordance with Australian Standard (AS) 2159-2009 based on the following criteria: $E_d \leq \phi_g R_{ug}$

Where E_d = design action effect, which considers the actions in accordance with AS1170.0, ϕ_g = geotechnical strength reduction factor, taken as 0.6 as per AS2159 with load testing of 2% of production micropiles; R_{ug} = ultimate geotechnical strength, assessed based on the assumed parameters in Table 1.

Regarding the design action effect 'E_d', AS1170.0 clause 4.2.3 indicates that a load factor of 1.2 should be considered for hydrostatic pressures 'F_{gw}' based on groundwater level as given in AS1170.1 clause 4.3, which states that a 1 in 50 Annual Exceedance Probability (AEP) groundwater level should be considered where groundwater information is available. Thus, the design action effect, $E_d = 1.2 \times E_{ds}$. Where E_{ds} = serviceability load for micropiles calculated using the finite element analysis program PLAXIS 2D, based on the design groundwater level.

3.2 *Cone pull-out*

The cone pull-out assessment methodology typically considered an "end-anchorage" at the base of the micropile to mobilise the soil / rock mass above and develop resistance. As illustrated in Figure 4a for the case of a single anchor, a simpler standard approach assumes that the uplift force in an anchor is resisted by the weight of a cone of rock mass (Littlejohn and Bruce, 1977). The anchor assumes an unbonded 'free length', h and bonded 'anchored' length, L. The authors consider this assumption to have been based on 'actively' post tensioned anchors.

The resistance for a cone pull-out was considered to be equal to the effective weight of an inverted cone of rock or soil having a base angle of either 60 degrees or 90 degrees measured from the bottom of the pile. The 60 degrees angle is recommended for soil, weak heavily fissured or weathered rock whereas 90 degrees is recommended for massive rock as shown in Figure 4a. For relatively close spacing of micropiles, potential overlap or interaction as shown conceptually in two dimensions in Figure 4b would occur where overlapped volume should be discounted.

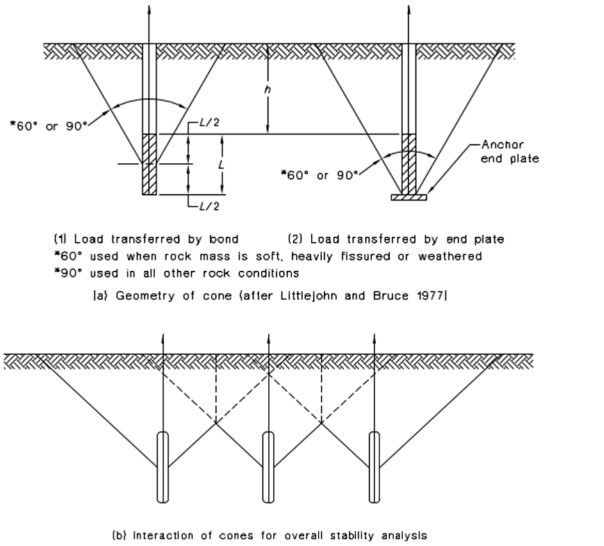

Figure 4. Cone geometries used in uplift capacity calculations for single (a) and group (b) tensioned anchors (after Littlejohn and Bruce, 1977, extract from AS4678-2002)

A load factor (γ_{ge}) of 0.7 (Table 6.4 of AS5100.2) was applied to the weight of the soil or rock in ultimate (ULS) resistance calculations where the weight of the soil or rock increases stability. Given the rock mass being mobilised is below the water table, a buoyant unit weight was also adopted. The uplift resistance provided by the rock mass is the total buoyant weight of the inverted cones of soil or rock mass taking into consideration the overlapping effect of adjacent cones. The shear and tensile resistance along the potential failure plane forming the cones are generally ignored. However, the estimated values of tensile or shear resistance (or cohesion) within the rock mass may be used either instead of, or in addition to, the resistance to uplift provided by the weight of the cone of rock as suggested by Brown (2015).

4 ANALYSIS AND RESULTS

Numerical analyses were also carried out using the finite element software PLAXIS 2D for the tension load derivation for three rows of micropiles to provide resistance against hydrostatic uplift pressure below the base slab as shown in Figure 5. The uplift resistance of a single micropile and group of micropiles in terms of bond strength and cone pull out was then assessed.

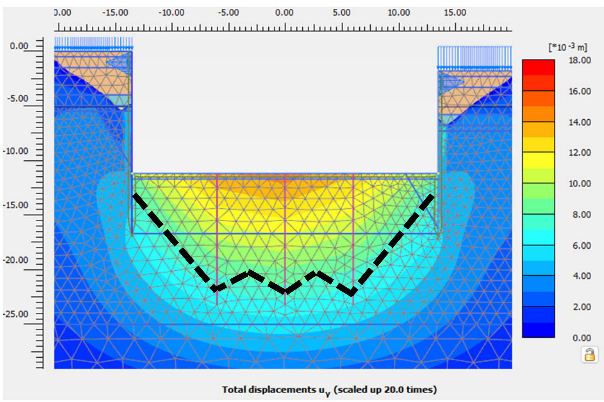

Figure 5. Contour of ground movement beneath base slab from PLAXIS 2D showing a group effect across 3 micropiles

4.1 *Tension force and bond strength failure*

The calculated maximum tension force from PLAXIS 2D analyses and the required micropile lengths are summarised in Table 2. The micropile lengths have considered the embedment stratigraphy directly over the highly weathered siltstone.

Table 2. Summary of tension load in micropiles at Design Section 11

Micropile Diameter/Length (mm) / (m)	Row/ Spacing (No.) / (m)	SLS, E_{ds} (kN/pile)	ULS, E_d (kN/pile)
325 / 11	3 / 6 x 5.5	1790	2148

4.2 *Cone pull-out mechanism*

As the micropiles were not post tensioned, the resistance of micropiles would only be initiated and mobilised as a 'passive' system when the uplifting groundwater pressure is imposed on the base slab. The group effect with a 'block' of similar movement contour across 3 piles has shown in Figure 5, which is inferred to be similar to that indicated in Littlejohn and Bruce (1977).

While cone pull out capacity has been assessed as per current standards and codes, a single micropile has been assessed using the 3-dimensional finite element program PLAXIS 3D to study the 'realistic' mechanism for a fully grouted, non post-tensioned micropile. The siltstone rock mass described in Section 2.1 has been modelled with a set of rock mass 'continuum' properties taking into consideration the bedding and non-persistent joint sets. The analysis showed that the predominant failure mechanism for a single micropile is failure along the shaft with formation of a 'shallow' rock cone near the surface as shown in Figure 6.

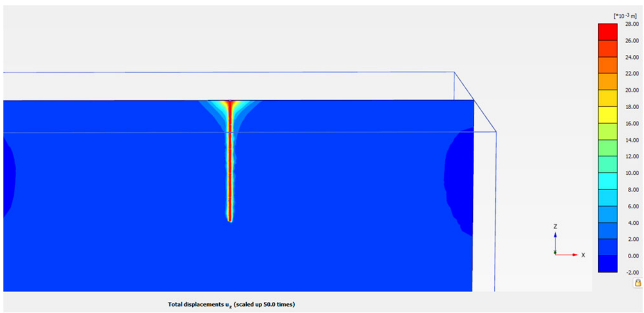

Figure 6. Single pile modelled in PLAXIS 3D showing predominant capacity is from shaft and relative larger mobilisation is on the upper pile section near surface forming a 'shallow' cone in siltstone.

A similar failure mechanism involving the combination of the shaft and a shallow cone at the top of the anchor was observed and inferred by Weerasinghe and Littlejohn (1997) as shown in Figure 7 for a straight shafted anchor grouted to the ground surface in a weak mudstone and subject to uplift.

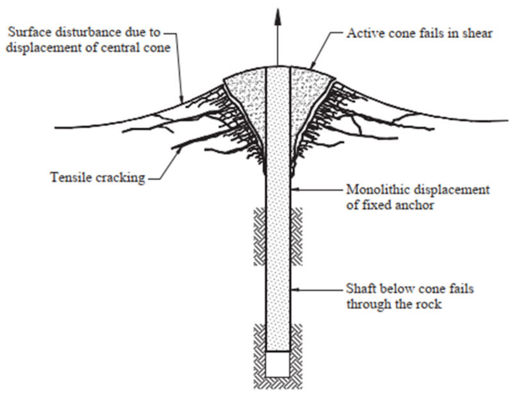

Figure 7. Failure by uplift of a fully grouted, straight-shafted, shallow, tensioned anchor in weak mudstone (Weerasinghe and Littlejohn, 1997).

However, as the base slab is connected to a group of reasonably closely spaced micropiles, PLAXIS 2D analysis showed that the likely failure mechanism with the group of micropiles could be a block of rock mass of overlapping cones extending close to the toe of the micropiles as indicated in Figure 5. The micropiles showed a 'mobilised' shaft force utilisation of approximately 30% of the design bond strength.

5 TESTING AND CONSTRUCTION

5.1 *Testing*

The testing of micropiles was carried out in two phases, i.e., pull -out testing on sacrificial piles and production piles prior to and during construction, respectively.

Pull-out testing was carried out on 4 no. sacrificial micropiles. The pull-out tests PO-4 and PO-5 successfully verified the design ultimate bond strength between the micropile grout and HW siltstone in Table 1 and shown in Figure 8. The test micropiles had 7m and 3m, free and bonded lengths respectively with a test setup as shown in Figure 9.

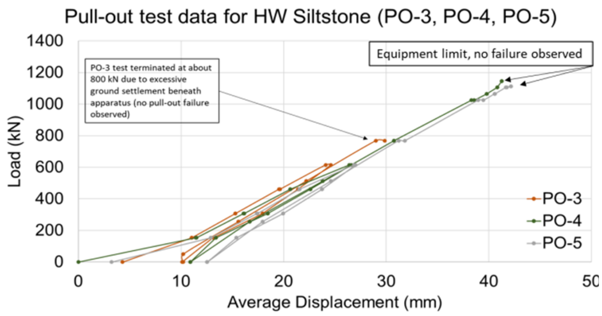

Figure 8. Pull out test showing a load displacement curve to maximum design ultimate load verifying bond strength in highly weathered siltstone.

The production testing of micropiles comprised static tension load testing in accordance with the requirements set out in Section 8 of AS2159. The loading program (including creep cycle durations) presented in Table A2 of AS2159 was adopted. 2% (10 no.) of the total number of production micropiles were also tested during construction.

The proof load testing was carried out to a maximum test load of 1.2 times the design load. The "proof testing" of the micropiles has successfully met the acceptance criteria as per AS2159. The load-displacement plots of the proof load tests were similar to the initial linear portion of the plots in Figure 8.

5.1 *Construction challenges and performance*

The occupation of the train tracks was 93 days (or 13 weeks), i.e. excavation of the trench (213,000m^3 of material) and the installation of 472 no. micropiles (including proof testing of micropiles) was completed in 4 weeks which allowed a site hand over to the structural team to commence the base slab and other station construction works. Micropile installation was completed earlier, validating the adoption of this system and leading to the successful completion of the overall station works within the occupation period.

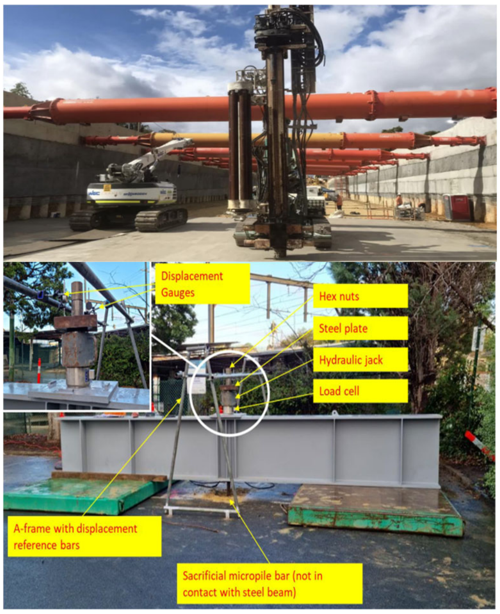

Figure 9. Micropile drilling rig at base of excavation level and pull out testing setup on sacrificial micropiles.

6 CONCLUSIONS AND FUTURE WORK

The design and performance evaluation of the micropile system implemented in the UMA underpass project at Union Station demonstrates a robust approach to resisting hydrostatic uplift pressures using a passive, fully grouted micropile system. While conventional methods often assume a dominant cone pull-out failure mode extending from the end of the anchor toe or shaft, the numerical analysis results indicate that uplift resistance is more accurately described as a composite mechanism involving both shallow cone formation and shaft resistance for a single micropile condition, but tending to a rock mass block of overlapping deeper cones extending close to the toe when the spacing of micropiles is closer. Future design practices should account for such hybrid behaviours, especially in weathered rock environments and closely spaced pile groups.

7 ACKNOWLEDGEMENTS

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