Use of screw piles for floatation resistance

L'utilisation de tas de vis pour la résistance de flottement

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ABSTRACT
Tugun Bypass Tunnel is part of the 7 km long Tugun Bypass connecting Southeast Queensland and Northeast New South Wales in Australia. The tunnel is under an extended section of the Gold Coast Airport runway and is built with diaphragm walls using the cut and cover method. The tunnel portals are joined with depressed ramps constructed with cast-in-situ U-shape reinforced concrete structures within the sheetpile-supported excavated areas. The site is underlain by alluvial and estuarine deposits up to 35m depth with the groundwater table close to the surface. Due to the high groundwater table, the ramps are constantly subjected to floatation. Steel screw piles with a single helix at the pile tip are used to resist the uplift force from floatation. The design of these screw piles requires the considerations of tensile capacity and long term displacement. A semi-empirical method was adopted for the calculation of the tensile capacity of the pile, which was then confirmed by full scale in-situ pull-out tests. Three-dimensional numerical modelling was undertaken to investigate the serviceability conditions of the ramp when it is subjected to floatation.

RÉSUMÉ
Le Tunnel de Pontage de Tugun fait partie du Pontage de Tugun de 7 kms de long raccordant Queensland du Sud-est et le Nouveau Pays de Galles Sud Nord-est en Australie. Le tunnel est sous une section prolongée de la piste d'envol et d'atterrissage d'Aéroport de Côte D'or et est construit avec les murs de diaphragme en utilisant la méthode de couverture et la coupe. Les portails tunnel sont rejoints avec les rampes déprimées construites avec les structures de béton armé d'U-forme cast-in-situ dans le sheetpile-soutenu a excavé des régions. Le site est sous-tendu par alluvial et estuarine dépose la profondeur de jusqu'à 35 m avec la table de nappe phréatique près de la surface. En raison de la haute table de nappe phréatique, les rampes sont constamment faites subir au flottement. Les tas de vis d'acier avec une hélice simple au bout de tas sont utilisés pour s'opposer à la force de tonus du flottement. Le design de ces tas de vis exige les considérations de capacité extensible et de déplacement à long terme. Une méthode semi-empirique a été adoptée pour le calcul de la capacité extensible du tas, qui a été alors confirmé par l'échelle complète dans - situ les épreuves de retrait. Le fait de modeler numérique en trois dimensions a été entrepris pour enquêter sur les conditions de praticabilité de la rampe quand il est fait subir au flottement.

Keywords : screw piles, flotation resistance

1 INTRODUCTION

The 7 km long Tugun Bypass is a four lane motorway located on the eastern coast of Queensland and New South Wales, Australia. The project alignment traverses hilly terrain in the north and floodplains in the south adjacent to the Gold Coast Airport. The key feature of the project is the tunnel of 334m in length with approach ramps to the portals of the tunnel. The tunnel had to be built concurrently with the construction of the extension of the runway over its top. The project also involved cuttings up to 30m depth, five bridges, four soil nail walls and large quantity of earthworks. Details of the project are presented by Hsi et al (2008).

In the area of the Tugun Bypass Tunnel, the ground water table is about 0.5m below the existing ground surface at RL0.2m. The 1 in 100 years ARI flood level is estimated to be 2.5m above the ground surface. Because of the high water levels, substantial uplift pressures will apply to the base of the ramps. Tension piles are therefore required to resist the buoyancy forces. With considerations of constructability, performance, economy and durability, steel screw piles were selected as the tension piles.

The steel screw pile is made up of a single helical-shaped circular plate welded to a steel circular hollow shaft with an outer shaft diameter of 219mm and a wall thickness of 8.2mm. The helix diameter chosen is either 600mm or 700mm with a plate thickness of 32mm. The length of the pile and the diameter of helix vary along the ramps depending on the design load requirement and the ground conditions. The uplift resistance of a screw pile is achieved primarily from end bearing of the soil above the helix. In order to achieve the required tensile capacity, the helix needs to be fully embedded in medium dense to dense sands at depth. The prediction of the screw pile capacity in sand is based on a semi-empirical method (Clemence, 1982; Mitsch and Clemence, 1985; Meyerhof and Adam, 1968) which incorporates a "breakout factor" that can be obtained from screw pile pull-out test results.

The screw piles also need to satisfy the serviceability requirement of the ramp which is allowed to move upwards up to 25mm. The prediction of the movement of the ramp supported on screw piles when subjected to floatation can be assessed numerically taking into account the effects of soil-groundwater-structure interaction. The finite element program PLAXIS 3D Foundation was used for such modeling.

Full scale in-situ screw pile pull-out tests were undertaken to investigate the pile tensile capacity and to obtain the required installation and design parameters. These tests were also used to confirm the serviceability performance of the screw piles.

2 DESIGN CONSIDERATIONS

Design of screw piles for tensile resistance needs to consider the uplift tensile capacity as well as the serviceability performance in terms of displacement. The tensile capacity can

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be calculated based on the method described by Clemence (1982), Mitsch and Clemence (1985) and Meyerhof and Adam (1968), which includes considerations of end bearing contributed by soils above the helix and skin friction around the shaft.

Based on the ground conditions along the ramps and the required uplift capacity, the screw piles to be used are categorised as Deep Anchor in Sand, i.e. $H/D > (H/D)_{cr}$. The uplift capacity for a deep anchor in sand consists of the shaft frictional resistance and the helix bearing capacity, as shown in Figure 1. A cylindrical shear model was proposed by Mitsch and Clemence (1985) for the determination of the ultimate tensile capacity of a screw pile. For a deep single helix screw pile in sand, the ultimate bearing capacity can be expressed as,

$$Q_t = \sigma_v' A_H F_q^* + 0.5 \pi d H_{eff}^2 \gamma' K_u \tan\phi' \quad (1)$$

where,

- Q_t = ultimate screw pile uplift capacity (kN)
- A_H = area of the helix (m^2)
- F_q^* = breakout factor
- d = outer diameter of the shaft (m)
- H_{eff} = effective shaft length as shown in Figure 1 (m)
- γ' = effective unit weight of soil (kN/m^3)
- K_u = coefficient of lateral earth pressure in sand when subjected to uplift
- ϕ' = angle of internal friction of soil (degree)
- H = total embedment depth (m)
- D = diameter of the helix (m)
- H_{cr} = critical embedment depth (m)

The embedment ratio (H/D) is defined as the total embedment depth H divided by the helix diameter D. The critical embedment ratio $(H/D)_{cr}$ (or H_{cr}/D) for a circular anchor is suggested by Meyerhof and Adam (1968), as shown in Table 1, and the coefficient of lateral earth pressure is recommended by Mitsch and Clemence (1985), as shown in Table 2. Equation 1 applies when $H/D > (H/D)_{cr}$.

Table 1. Critical Embedment Ratio $(H/D)_{cr}$ for Circular Anchor (after Meyerhof and Adam,1968)

Friction Angle ϕ'	25°	30°	35°	40°	45°
Depth $(H/D)_{cr}$	3	4	5	7	9

Table 2. Recommended Uplift Coefficients K_u for Helical Anchors (after Mitsch and Clemence, 1985)

Friction Angle ϕ'	25°	30°	35°	40°	45°
Recommended Coefficients for Helical Anchors K_u	0.70	0.90	1.50	2.35	3.20

In Equation 1, the breakout factor F_q^* is related to the friction angle of sand above the helix. Based on Mitsch and Clemence's (1985) theory, Das (1990) explicitly demonstrated the relations between the breakout factor and sand friction angle. For friction angle of 28° to 43°, the breakout factor can vary from approximately 20 to 200. It can be seen that any minor change in friction angle of sand can cause significant variation of the screw pile uplift capacity. Therefore it is important to justify the breakout factor for the assessment of the pile capacity.

Upward displacement of the ramp structure supported on screw piles can be predicted using a numerical means which takes into account soil-structure-groundwater interaction. This approach is discussed in Section 4 below.

The estimated net pressure acting below the base of the ramp is based on the assumptions where the ground water table is at RL0.5m in normal conditions and at RL2.7m in a 1 in 100 year flood event. The resistant forces include the weight of the ramp structure, the friction between the ramp side walls and the soil, and the tensile capacity of the screw piles. Two failure modes of the screw piles are considered: individual anchor failure and group anchor failure. The former failure mode depends on the ultimate capacity of a single anchor, and the latter depends on the mobilised mass of soil above the group of anchor helices. The ultimate capacity of a single anchor is calculated based on the semi-empirical method as discussed above with a geotechnical strength reduction factor of 0.4 adopted for the determination of the working load capacity of the anchor. For the group anchor failure mode, it is assumed that a cone above a helix is formed along a slip surface with an angle (to vertical) of half of the soil friction angle. These cones would intercept each other at some height above the helix. The total mobilised soil mass is the sum of all cones and the block above the intercepted surface. A tensile load factor of 1.3 is adopted. The minimum anchor depth required to achieve satisfactory resistance against buoyancy can then be determined.

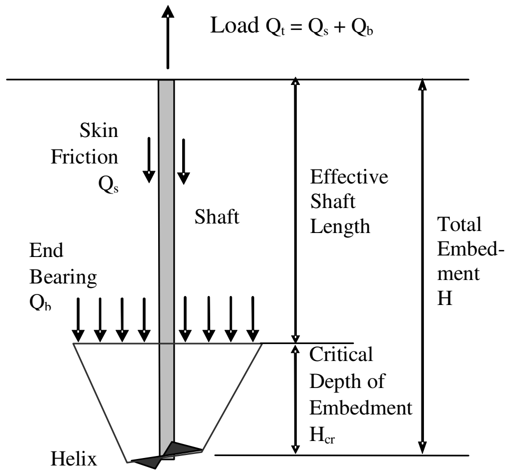

Figure 1. Resistant Forces of Screw Pile

In addition to meeting the required tensile capacity of the screw piles, they also need to satisfy the serviceability requirement which is based on a deflection limit criterion of the ramp structure defined to be in the order of 25mm. The deflection includes the upward movements of the structure when the water table returns to its normal level after dewatering for the ramp construction is terminated, the 1 in 100 years flood event occurs, the steel thickness reduces after corrosion loss (with a corrosion allowance of 0.04mm/year) and the soil creeps in the long term.

3 PULL-OUT TESTS

Six full scale in-situ pull-out tests of screw piles have been carried out to observe the field performance and derive design and installation parameters. The conditions of these tests are summarised in Table 3.

Note that the subsurface conditions described above are the layers of soil (top down) between the ground surface and the helix where consistencies of sands and corresponding thicknesses in meters shown in brackets are given. RL1 is the existing ground surface level, RL2 is the measured groundwater level within the dewatered area, RL3 is the test formation level, and RL4 is the level of helix. The helix diameter is 700mm for Test Nos. 1 and 2 and 600mm for Test Nos. 3 to 6, and the shaft diameter is 219mm.

During installation of the test piles the hydraulic torque was recorded for every 250mm advance of the screw pile and the revolutions per 500mm were visually recorded. Static tensile loads were applied via a jack against four reaction screw piles. Each test includes three runs of tensile loads:

1. First Cycle Run – loading incrementally up to 100% of the predicted serviceability load and held for 6 hours;
2. Second Cycle Run – loading incrementally up to 150% of the predicted serviceability load and held for 1 hour;
3. Failure Test Run – increasing load incrementally until the total deflection of pile head exceeding 15% of the helix diameter.

The predicted serviceability loads used for the pull-out tests are shown in Table 4.

Table 3. Summary of Screw Pile Conditions

Test No	Subsurface Conditions	RL1 (m)	RL2 (m)	RL3 (m)	RL4 (m)
1	L & VL (3); MD (3.7)	0.50	-2.07	-1.47	-8.15
2	L & VL (3); MD (6.2)	1.48	-2.07	-1.47	-10.62
3	VL & L (4.5); MD (7.9)	0.74	-3.25	-1.45	-13.80
4	VL (3.3); VD & MD (11.6); L (3); MD (1.1)	1.85	-1.20	0.70	-18.30
5	MD (1.3); VD (5.5); MD (4)	1.22	-3.25	-1.50	-12.25
6	MD & VD (3); L (2.6); MD (6.6)	1.11	-8.50	-6.80	-19.00

Table 4. Predicted Serviceability Loads

Test No	1	2	3	4	5	6
Serviceability Load (kN)	250	380	350	650	425	430

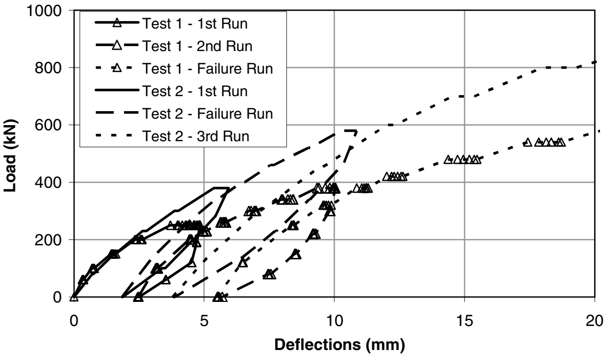

Figure 2. Screw Pile Tests – Cycle Test Runs

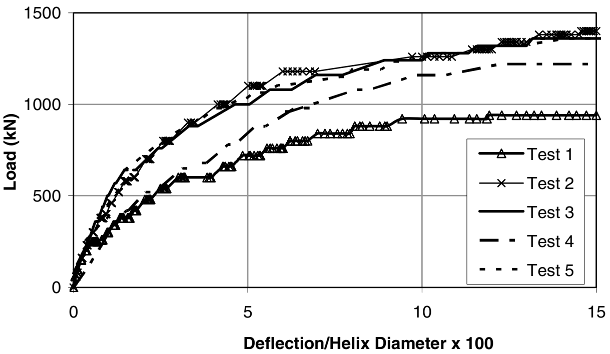

Figure 3. Screw Pile Tests – Failure Test Runs

The level of test formation was within an excavated and dewatered area. During the period of testing, the groundwater table adjacent to the test pile was recorded. Figure 2 shows results of the three cycle runs for Test Nos. 1 and 2 and Figure 3 shows the results of failure runs for Test Nos. 1 to 5.

Test No. 6 was undertaken for the investigation of long term creep movement of the screw pile. The test was carried out in several runs under the predicted serviceability load of 430kN. When the serviceability load is reached, it is held for durations of 500, 900 and 1600 minutes for Test Runs 1, 4 and 5. The deflections of the pile versus elapsed time were recorded, as shown in Figure 4 where the scale of time is in logarithm. Further extrapolation is undertaken by drawing straight lines through the last several deflection measurements to derive the predicted long term creep movement in 100 years. The figure shows that the predicted creep movement in 100 years under the serviceability load is in the order of 2.2 to 3.8mm measured from the time when the serviceability load is reached. After the 5 runs for creep tests, the pile was then loaded to failure, i.e. the deflection exceeds 15% of the helix diameter.

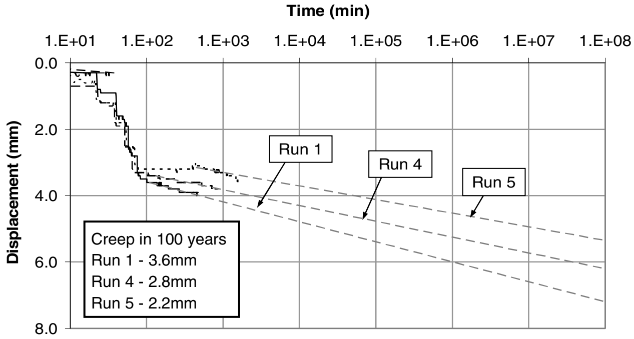

Figure 4. Screw Pile Test No. 6 – Creep Test

The ultimate capacity of the screw pile is defined as the test load corresponding to a deflection of the pile reaching 10% of the helix diameter. Based on the results of the Failure Test Runs, the observed ultimate tensile capacities are shown in Table 5. These values can then be used in Equation 1 to calculate the breakout factors F_q^*. The back calculated F_q^* values are shown in Table 5. The SPT values and the measured torques at the pile toe levels are also shown in Table 5.

Table 5. Summary of Screw Pile Test Results

Test No	SPT Value	Torque (kN-m)	Ultimate Capacity (kN)	Back Calculated F_q^*
1	29	70	920	40.3
2	27	85	1250	39.2
3	22	92	1240	37.3
4	13	95	1150	20.5
5	21	105	1250	37.1
6	19	125	1270	36.4

4 NUMERICAL MODELLING

A typical screw pile arrangement on a cross section is shown in Figure 5. A 3D finite element model using PLAXIS 3D Foundation was set up to simulate this section. The model includes the subsurface soils, groundwater, screw piles and ramp structure.

The ramp walls are approximated to be 1m thick uniformly, and ramp base is assumed to be 1.2m uniform thickness. The ground level is at RL0.5m and the average level of the underside of the ramp slab is at RL-6.5m.

In the model, 3 rows of screw piles are simulated with a pile length of 12m. The pile spacing ranges between 2.34m and 3m in the transverse direction and 2.43m in the longitudinal direction. The initial shaft and helix diameters/thicknesses are assumed to be 219mm/8.2mm and 600mm/32mm respectively. The corrosion thickness in 100 years is assumed to be 4mm. The shaft is modelled as an equivalent solid circular pile with equivalent modulus and density.

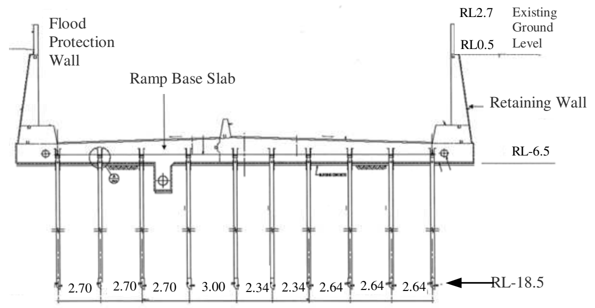

Figure 5. Typical Screw Pile Arrangement

The construction sequence modelled includes the lowering of the groundwater table below the base of the ramp, excavation of soil, installation of screw piles, construction of the ramp structure, return of the groundwater table below the base of the ramp to the ground surface, rise of the water table to 2.5m above the ground surface to simulate the design flood event and change of the shaft and helix thicknesses to allow for 4mm corrosion. Half of the ramp is analysed considering symmetry of the structure against the centre line.

The Hardening Soil Model of PLAXIS was adopted to simulate the soil behaviour subjected to unloading and reloading. The adopted key soil parameters are given in Table 6.

Table 6. Soil Parameters assumed for Numerical Modelling

Top RL (m)	Consistency	γ_{sat} (kN/m^3)	ϕ'	E_{50}^{ref} & E_{oed}^{ref} (MPa)	E_{ur}^{ref} (MPa)
0.5	VL	18	30^o	10	30
-3	L	18	32^o	30	90
-4.8	MD	19	34^o	50	150
-11	D	20	36^o	80	240
-17	MD	19	34^o	50	150
-18.5	Rock	-	-	-	-

Note that γ_{sat} is saturated unit weight; ϕ' is drained friction angle; E_{50}^{ref} is secant Young's modulus at a reference pressure of 100 kPa; E_{oed}^{ref} is tangent Young's modulus for primary odometer loading at a reference pressure of 100 kPa; E_{ur}^{ref} is unloading/reloading Young's modulus at a reference pressure of 100 kPa; cohesion c'=0 kPa; Poisson's ratio ν'=0.3 (drained) and ν'_{ur}=0.2 (unloading/reloading).

The deformed 3D finite element mesh with a water level at RL2.7 is shown in Figure 6. The arrangement of the modeled screw piles are shown in Figure 7 where the soil elements are not shown. It was found that the maximum displacement of the ramp under the design flood event (flood level at RL2.7m) occurred at the centre of the ramp. The calculated maximum upward deflection is about 11mm prior to corrosion of the screw piles. With the assumed 4mm corrosion in 100 years, the calculated maximum deflection is 12mm with the water level assumed at the ground surface.

The interpretation of the screw pile creep test results shows that the creep movement over the 100 years design life is up to about 3.5mm (see Figure 4). For Test No. 6, it was observed that there is negligible additional pile movement due to the 5 cyclic serviceability loads applied.

In summary, the maximum deflection of the ramp supported on screw piles is estimated to be approximately 16mm under the serviceability conditions which satisfies the design criterion of 25mm for the ramp structure. The actual measured movements of the ramps after de-commissioning of dewatering when the ramp construction was completed were well within 10mm.

5 CONCLUSIONS

Steel screw piles are used to resist the uplift groundwater pressure acting against the ramp structure of the Tugun Bypass Tunnel. A semi-empirical method was adopted for the calculation of tensile capacity of the screw pile. Full scale in-situ pull-out tests of screw piles are carried out to observe the pile performance as well as derive the design and installation parameters. Three-dimensional numerical modelling using PLAXIS 3D Foundation was undertaken to investigate the soil-structure-groundwater interaction in response to the buoyancy effects. The long term screw pile serviceability performance was predicted using both the numerical modelling and the in-situ pull-out test results. The actual response of the ramp structure compares closely with the predictions.

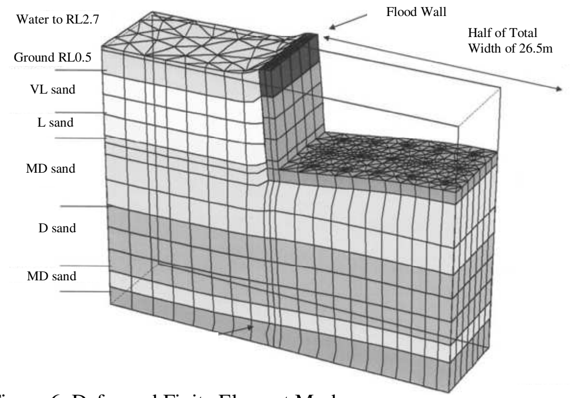

Figure 6. Deformed Finite Element Mesh

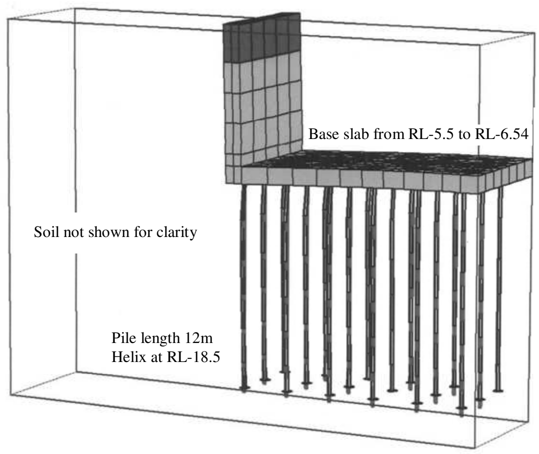

Figure 7. Screw Pile Arrangement in 3D Model

REFERENCES

Clemence, S.P. 1982. "Uplift and Bearing Capacity of Helix Anchors in Soil", Research Contract Report TT-1 12-1, Vol. 1, Niagara Mohawk Power Corporation, Syracuse, NY.

Das, B.M. 1990. Earth Anchors. Elsevier, Amsterdam, 241p.

Hsi, J., Lambert, S. and Thomas, M. 2008. "Tugun Bypass Tunnel Using Top-Down Cut-and-Cover Method", 13th Australian Tunnelling Conference, Melbourne, pp. 29-38.

Meyerhof, G.G. and Adams, J.I. 1968. "The Ultimate Uplift Capacity of Foundations", Canadian Geotechnical Journal, Vol. V, No. 4, pp. 225-244.

Mitsch, M P. and Clemence, S.P. 1985. "The Uplift Capacity of Helix Anchors in Sand" Uplift Behavior of Anchor Foundations in Soil, Proceedings of ASCE, N.Y. pp. 26-47.