Compression capacity of steel screw piles

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ABSTRACT

Over the last 15 years steel screw piles have emerged to the extent that they occupy between 10 to 20% of the Australian piling market, particularly in the area of low to medium rise buildings. With their rapid rise as a cost-effective and versatile foundation solution, much of the engineering profession has overlooked the limitations of this pile type. In particular, the critical dependence of the geotechnical strength on the structural bending strength of the helix plate is a unique and important feature of steel screw piles. The results of pile load tests are compared with theoretical predictions of ultimate geotechnical strength and empirical correlations with CPT cone resistance (q_c) and also the maximum installed torque. Some cautionary tales and recommendations for the designer of steel screw piles are given.

1 INTRODUCTION

Although suitable for many different applications, including underpinning (Yttrup 1998) and landslide stabilization (Yttrup & Miner 1998), the greatest impact of steel screw piles has been in the area of low to medium rise buildings. As a foundation to support vertical column loading, steel screw piles are typically used for working (or serviceability) loads of up to 1000kN.

There are several forms of steel screw piles in use ranging from continuous (solid) flight augers of up to 200mm diameter, similar to that used for geotechnical site investigation purposes, to those comprising a large diameter (up to 750mm diameter) helical plate welded to the base of a circular hollow section (CHS) steel shaft. This paper is focused on this more common type of steel screw pile, the typical dimensions of which are shown in Figure 1.

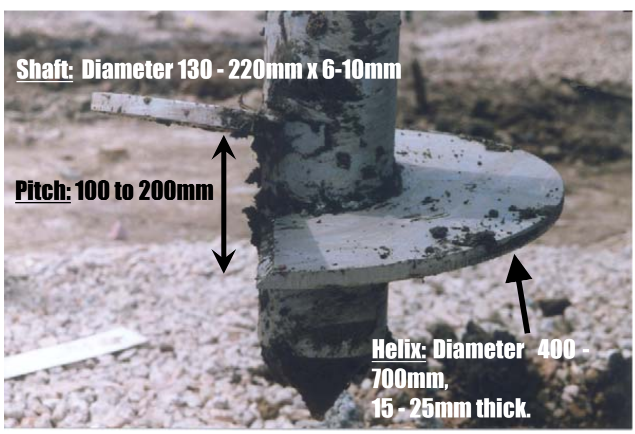

Figure 1: Typical dimensions of “heavy duty” steel screw piles

Aside from their many applications and benefits, steel screw piles are not without limitations, most of which stem from a misunderstanding of their behaviour during installation and under loading. In particular, the improper use of empirical torque correlations and an inadequate regard for the reliance on the structural capacity of the helix to achieve design geotechnical strengths has resulted in numerous load test failures and consequent disputes. Three case studies involving static load testing of steel screw piles in sand are presented and used to illustrate some important features of the behaviour of this pile type.

2 INSTALLATION & TORQUE

Steel screw piles are installed by the rotation of a planetary drive motor mounted on a hydraulic excavator. The pile is screwed into the ground with some downthrust also applied. When engaged, the helix plate is pulled into the ground by the helix reacting upwards, thereby unloading the founding stratum ahead of the helix. Thus, the stress-strain behaviour of the founding stratum beneath the base of the helix is similar to that of a bored or non-displacement pile.

Torque, which refers to the rotating or twisting force, is used by many steel screw pile contractors as a means of assessing the installed pile capacity, either for design or verification purposes. A correlation between installation torque and the uplift capacity of model helical screw anchors was established by Ghaly & Hanna (1991). Empirical correlations in the order of ten times the measured torque, in SI units (e.g. kN.m), at the founding level are commonly used to estimate the ultimate geotechnical strength (R_{ug}) in axial compression. (R_{ug} is typically defined as the load measured in a test load where the pile head displacement equals 10% of the pile base diameter). Such correlations are not valid unless founding within a completely homogenous soil profile, which is rare in the layered alluvial and residual soils that typically govern the coastal development centres around Australia.

Measured torque applies to the soils through which the helix has already penetrated and does not relate to the strength of the soil below the base of the helix. In this way, the measured torque provides no indication of underlying weaker (or stronger) materials and therefore is very different to the "set" used for establishing the founding level of driven piles. The author is aware of projects where a pile capacity shortfall occurred due to founding steel screw piles on thin sand layers underlain by clays of only soft to firm consistency.

The relationship between the maximum measured installation torque at the founding level (T) and R_{ug} determined from static load testing is plotted for 32 tests is shown in Figure 2.

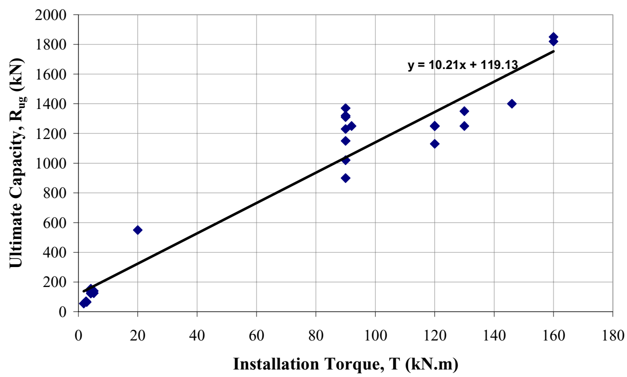

Figure 2: Relationship between maximum measured torque and R_{ug} from load tests

Also, torque is measured by a hydraulic pressure gauge, which is usually crudely calibrated in the gearbox manufacturer's workshop and typically in one or two gears only. Most gearbox manufacturers would be horrified to learn that torque is being used as a basis to pile design and/or construction verification.

It is therefore not appropriate for contractors to target a minimum measured torque as the sole means of design or construction verification of R_{ug}. In soil profiles indicated to be relatively uniform by boreholes or cone penetration testing (CPT), 10 times T (in SI units) may however provide a useful qualitative guide to R_{ug} and required founding levels.

3 CLASSIFICATION OF STEEL SCREW PILES

In AS 2159 (1995), steel screw piles are classified as "preformed displacement" piles. Due to the method of installation there is no significant densification of the soil surrounding the piles. Static load test results have shown that the ultimate base bearing pressure (f_b) is closer to 0.3 times the CPT cone resistance (q_c) than $0.6q_c$ – $0.8q_c$, typical for full displacement (e.g. driven) pile types. It is therefore not appropriate to use the same design parameters that apply to driven piles and in this way the classification of steel screw piles is considered misleading.

4 CASE STUDIES

The results of sixteen static (compression) load tests carried out on steel screw piles founded in sand are presented in the following case studies. The tests were conducted on three different sites, on piles installed by three separate piling contractors. For all tests, the application and measurement of load and deflection was carried out in accordance with the Incremental Sustained Load (ISL) procedure described in AS 2159 (1995).

4.1 Swansea Apartments

Six tests were conducted for a four storey residential development at Swansea, near Newcastle in New South Wales (NSW). The piles tested were founded in dense to very dense sand at 8.5 - 9.0 m depth. CPT indicated q_c values in the range 20 – 30 MPa at the founding level. The test piles comprised 700 mm diameter, 20 mm thick helix plates at the base of a 219 mm diameter circular hollow section (CHS) shaft.

The load test results for all piles tested are shown in the form of a graph showing applied load versus pile head deflection, in Figure 3.

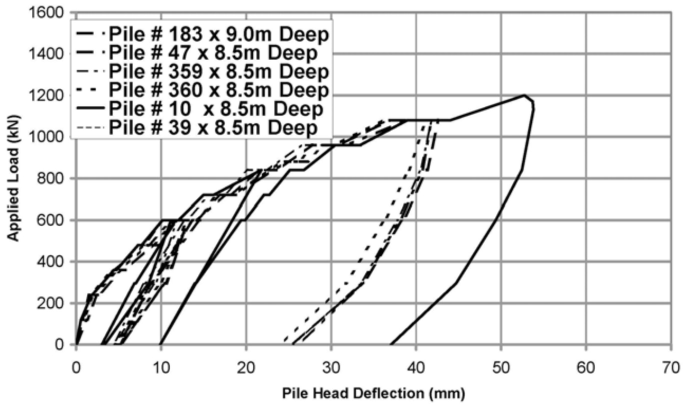

Figure 3: Results of load tests at Swansea

The results of all six load tests were fairly similar, particularly up to the serviceability load of 600 kN (75%S*). Pile 10, which was tested to 150%S*, did not meet the maximum or residual deflection acceptance criteria given in Table 8.2 of AS 2159 (i.e. <50 mm & <30 mm respectively).

4.2 The Entrance Apartments

Six load tests were undertaken for a three to four storey residential development at The Entrance, on the Central Coast of NSW. Piling was conducted from the original surface and all piles were founded in medium dense to dense sand at 5.5 - 6.5 m depth, with q_c ranging between 10 MPa and 25 MPa.

For this load-testing programme, particular emphasis was placed on assessing the effect of varying helix sizes, i.e. diameter and thickness. The test piles comprised a single helix of between 600 mm and 750 mm diameter at the base of a CHS shaft ranging in size from 168 mm and 219 mm in diameter. Also, the thickness of the helix plates varied between 20 mm and 40 mm.

The effect of varying the helix thickness and diameter can be observed in Figure 4, which shows the applied load versus pile head deflection response for four adjacent piles founded at the same or similar depths. Two piles had a helix thickness of 20 mm and the other two, a thickness of 40 mm. The average response for the piles with 20 mm and 40 mm thick helices are also shown.

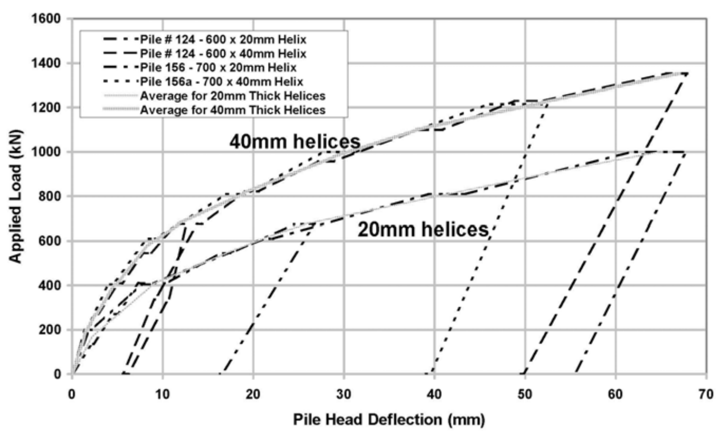

Figure 4: Comparison of load test results for helices of 20 mm and 40 mm thickness

The results indicate the significantly greater capacity that can be achieved by using the 40 mm thick helix (e.g. 50% extra load at the serviceability load levels). Also of particular interest was the almost identical load-deflection response for the 600 mm and 700 mm diameter helices, for both the 20 mm and 40 mm thick helix plates. Thus, no increase in pile capacity was apparent for larger diameter helix, for helices of the same thickness.

It is noted that the load test results indicate that the piles were close to the AS 2159 (1995) deflection criteria for the nominated serviceability loads (400 – 675 kN), of less than 15 mm and 7 mm for the serviceability and "zero" load respectively. Large pile displacements were recorded however for the maximum test load cycle (to 150%S*), such that the acceptance criteria was exceeded, particularly for the piles with only 20 mm thick helices.

4.3 Newcastle West Apartments

Two static load tests were performed for a new residential building of five storeys located to the west of Newcastle CBD. The building featured a partial basement at about 2 m below street level and piling was conducted from the bulk excavation level (BEL). The test piles were founded at approximately 6m below BEL within very dense sand. An average q_c of 30 MPa was indicated at this level. Both test piles consisted of 700 mm diameter, 25 mm thick helices at the base of a 193 mm diameter CHS shaft.

Testing for the piles was undertaken using a novel configuration of steel loading beams and reaction piles, which allowed the simultaneous measurement of both pile head and pile toe (or base helix) deflections in response to the applied test loading.

The results of one test are shown in the form of a graph of applied load versus pile head (and toe) deflection, in Figure 5. The difference between the pile head and pile toe displacement curves is explained by the elastic compression or shortening of the pile shaft, which was approximately 5 mm at the serviceability load of 750 kN and 10 mm at the maximum test load of 1500 kN (150%S*). Again, pile head displacements were marginally in excess of the acceptance criteria in AS 2159 (1995) at the serviceability load, but greatly exceeded criteria at the maximum test load.

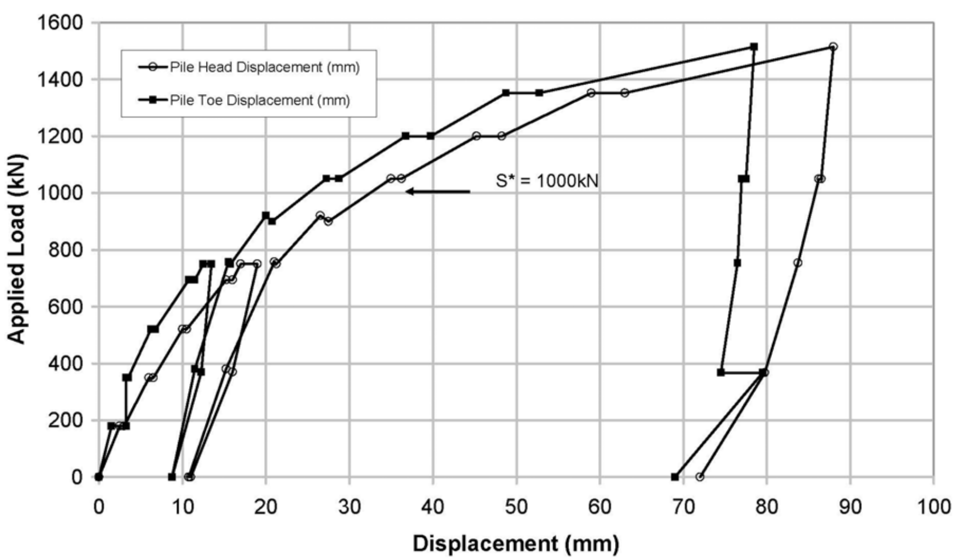

Figure 5: Typical load test result for Newcastle West

4.4 Discussion of Results

A summary of the results for the 14 load tests conducted is presented in Table 1. The measured R_{ug} is compared with predictions of R_{ug} from both theoretical and empirical methods, for the steel screw piles tested. The theoretical R_{ug} was calculated for a deeply buried circular footing using the bearing capacity factors and expression given by Terzaghi (1943). The empirical R_{ug} was estimated on the basis of correlations for f_b of bored piles in sand, as described previously (i.e. $0.3q_c$). For both methods of predicting R_{ug}, the base helix was assumed to be completely rigid and shaft friction was ignored, as is appropriate for this pile type.

Table 1: Summary of Measured and Predicted Ultimate Geotechnical Strengths

Site	Test Pile Dimensions			Measured Ultimate Strength (kN)	Theoretical Ultimate Strength (kN)	Empirical Ultimate Strength (kN)[3]
	Depth [2] (m)	Shaft (mm)	Helix (mm)			
Swansea[1]	9.0	219 x 8 CHS	700 X 20	1250	2350	2660
	8.5	219 x 8 CHS	700 X 20	1250	2280	3230
	8.5	219 x 8 CHS	700 X 20	1250	2130	2660
	8.5	219 x 8 CHS	700 X 20	1250	2130	2660
	8.5	219 x 8 CHS	700 X 20	1250	2280	3230
	8.5	219 x 8 CHS	700 X 20	1250	2280	3230
The Entrance	5.5	219 x 8 CHS	700 X 20	1020	1280	2890
	6.4	219 x 8 CHS	750 x 40	1320	1160	2120
	5.4	168 x 8 CHS	600 x 20	1150	940	2200
	5.9	168 x 8 CHS	600 x 20	1230	740	1530
	5.4	168 x 8 CHS	600 x 20	900	940	2200
	5.6	168 x 8 CHS	600 x 40	1310	900	1950
Newcastle West	5.9	193 x 10 CHS	700 X 25	1400	1620	2890
	5.9	193 x 10 CHS	700 X 25	1350	1620	2890

Notes:

1. Some extrapolation was necessary to estimate measured ultimate strength for Swansea load tests that were terminated at 40-45 mm (pile head) deflection.
2. Depth denotes the length of the pile from the bulk excavation level down to the founding level.
3. Empirical ultimate (geotechnical) strength based on; $f_b = 0.3q_c$.

The measured R_{ug} is significantly less than that predicted using empirical correlations for all of the piles tested. In stronger (high q_c) ground conditions, such as for the Newcastle West site, the empirical R_{ug} was between 100% and 160% greater than the measured R_{ug}. The theoretical estimates of R_{ug} were also mostly substantially greater than that measured, particularly where deeper screw piles were used and stronger ground conditions were indicated. For The Entrance site however, where embedment depths were less, only medium dense soils were indicated and the groundwater table was near the surface, the measured R_{ug} was sometimes greater than that predicted from theory, for the smaller helix sizes.

The piles tested typically failed the deflection acceptance criteria given in Table 8.2 of AS 2159 (1995) for the 'ultimate' load cycle (to 150%S*). A number of the test piles were exhumed and examined in an effort to understand the behaviour of the screw piles under loading. In most cases the helices were clearly dished and bent upwards indicating plastic deformation had occurred during loading. Thus, structural failure of the piles had occurred, which would explain why the measured R_{ug} was significantly less than that predicted from empirical methods and often from traditional bearing capacity theory.

It is considered that the incompatibility between geotechnical and structural reduction factors is a primary cause of the helix deformation during test loading. Steel screw piles are typically designed on the basis of a Geotechnical Strength Reduction Factor (Φ_g) of about 0.5, whereas the Structural Strength Reduction Factor (Φ_s) is typically 0.9. Therefore there is insufficient structural capacity in reserve for test loading to 150%S*, as required by AS 2159 (1995) and that is why plastic deformation of the helix occurs. Structural failure would not be expected to occur at normal serviceability loads. This observation is consistent with the load test results, which were typically close to Code deflection criteria at the serviceability load. It is also noted that the piles with a helix thickness of 40mm mobilised an R_{ug} in excess of that predicted from theory. Therefore, steel screw piles to be tested to above serviceability loads should incorporate an additional thickness of steel to ensure that the helix does not deform during testing.

Yttrup & Abramsson (2003) noted that the mobilized base pressure was significantly less than predicted from commonly used empirical analysis methods for pile design and that the deficit becomes more pronounced for stronger ground conditions or thin helices. Further, it was noted that for strong ground or thin helices, the helix plate will fail in bending by plastic deformation before the ultimate geotechnical strength can be realised. On the basis of finite element analyses and a series of laboratory plate bending tests, the authors proposed a four-part expression to predict R_{ug} for steel screw piles. The expression includes terms for the bending strength of helix and the bearing capacity that will be mobilised beyond the 'plastic hinge' that defines the limit of the unyielding annulus of the helix plate.

Thus, in summary, the base resistance and therefore the ultimate geotechnical capacity (R_{ug}) of steel screw piles are governed by the simultaneous mobilisation of structural and geotechnical resistance.

5 CONCLUSIONS

The relationship between torque (T) and the ultimate geotechnical strength (R_{ug}) of an installed steel screw pile is tenuous, particularly where the soil profile is layered. It should not be used in isolation as a design tool or as a means of construction verification. In relatively uniform soil profiles an approximate correlation of R_{ug} = 10T (in SI units) may be used as a guide to establishing founding levels. Comprehensive site investigation, preferably using CPT methods, should form the basis of establishing founding levels for steel screw pile contracts.

The ultimate geotechnical capacity for steel screw piles is typically well below that predicted by empirical correlations for non-displacement piles and usually less than predicted from traditional bearing capacity theory. A separate classification of “small displacement piles” is warranted for this pile type in AS 2159 (1995) to reflect the insignificant level of densification associated with installation.

One of the most important and unique aspects of the behaviour of steel screw piles is that the ultimate base resistance that can be mobilized is limited by the structural strength of the helix plate. Designers should make due regard for dependence of this pile type on the structural or flexural strength of the helix for mobilisation of the design geotechnical strength. In particular, piles to be tested will generally require an additional thickness of the helix plate to sustain the maximum test loading cycle prescribed in AS 2159 (1995), to 150%S*.

The thickness of the helix is the most critical dimension governing the mobilised geotechnical strength of steel screw piles. Load testing has demonstrated that there is little value in increasing the helix diameter beyond 600 mm, unless accompanied by a commensurate increase in the thickness of the helix. The serviceability load of steel screw piles founded in sand should generally be limited to about 600 kN for helices of 600 mm diameter and 20 mm thickness, increasing to a maximum of 750 kN for helices of 700 mm diameter and 25 – 40 mm thickness.

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