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Soil-foundation interaction analysis of a piled raft system supported by single helix screw piles

Analyse de l’interaction sol-structure d’un système de fondation sur pieux vissés à hélice simple

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ABSTRACT: Screw piles have been increasingly used as an alternative to traditional piling techniques to carry loads and control settlement of raft slabs. The available literature indicates that limited studies have been conducted to evaluate the performance of the piled raft foundations supported on screw piles. Therefore, it is essential to develop a robust design methodology for the reliable prediction of the performance of piled raft foundations supported on screw piles and manage the risks associated with their design. This paper presents the methodology adopted to analyse the soil-structure interaction (SSI) of a piled raft supporting a multi-storey building constructed over a highly variable ground profile. 3D numerical analysis was used to model the SSI between the soil and piled raft slab which included around 200 multi-turn single helix screw piles. The ground model was developed based on the results of a comprehensive site investigation program that included CPTs and DMTs. An innovative approach was used for the numerical simulation of the screw piles and results compared with empirical correlations to validate the adopted approach. The predicted performance of the piled raft was calibrated and optimised using the results of static pile load tests completed at bulk excavation level. Piles were initially installed adjacent to CPTs and DMTs locations and a site-specific correlation was developed between torque and soil type and strength/relative density. This correlation was then used as a general indication to validate the ground model. Where necessary the model ground conditions were amended to reflect those present on site. Monitoring of the settlement of the piled raft slab was completed and compared with the predicted results. A design flowchart is proposed, this summarises a design methodology that can be used to carry out SSI analysis.

RÉSUMÉ : Les pieux vissés sont de plus en plus utilisés comme une alternative aux techniques traditionnelles de pieux pour reprendre les chargements et contrôler les tassements des fondations superficielles. Les documents disponibles dans la littérature indiquent que les études menées pour évaluer la performance des dalles de fondation sur pieux vissés sont limitées. Par conséquent, il est essentiel de développer une méthodologie robuste de conception afin d’obtenir une prédiction fiable de la performance de ces dalles et de limiter les risques associés à leur conception. Cet article présente la méthodologie que nous avons adoptée pour analyser l'interaction sol-structure (ISS) d'une fondation sur pieux qui supporte un bâtiment de plusieurs étages construit sur un sol de niveau très variable. Une analyse numérique 3D a été réalisée pour modéliser l'interaction sol-structure (ISS) entre le sol et la dalle soutenue par les pieux. La structure comprend environ 200 pieux vissés multi-tours à hélice unique. Le modèle du sol a été développé sur la base des résultats d'une compagne de mesures comprenant des essais à pénétromètre statiques (CPT) et des essais dilatométriques (DMT). Une approche innovante a été utilisée pour la simulation numérique des pieux vissés et les résultats ont été comparés à des modèles empiriques afin de valider l'approche adoptée. Les performances calculées de la dalle sur pieux ont été calibrées et optimisées à l'aide des résultats d'essais de chargement statique de pieu. Les valeurs de couples enregistrées lors de l'installation des pieux ont été corrélées avec les types de sol et les résistances du sol aux emplacements des CPTs et cette corrélation a ensuite été utilisée pour vérifier le modèle de sol en subsurface adopté dans la modélisation numérique. Les tassements mesurés lors du programme du suivi ont été comparés aux résultats calculés. Un diagramme de conception est proposé résumant la méthodologie adoptée pour effectuer l'analyse SSI. Les défis et les leçons tirées de cette conception sont également discutés.

KEYWORDS: Screw pile, piled raft slab, foundation design, soil-foundation interaction, numerical analysis

1 INTRODUCTION

Screw piles are a type of piled foundation, or retaining wall anchor, that have been in use since the 1830’s. They are made of circular hollow steel sections with one or more helices welded to the shaft that provide a self-tapping mechanism during installation (Figure 1 (a)). The hollow stem may be filled with reinforced concrete following installation and is structurally connected to the building substructure. Shaft diameters typically range from 50mm up to 600mm and helix diameters generally range from 150mm up to 1200mm depending on capacity requirements. Screw piles are an option where the ground near the surface is, or has become too weak, to support a structure. Screw piles can also be used where the shape, size and location of the structure cannot be supported by alternative foundation methods (IPENZ 2015).

The raft slab is a common shallow foundation system that is used to support structures. In many instances, while a raft slab has sufficient capacity to resist the applied loads, induced settlements, both total and differential are excessive, making a raft slab unsuitable for the project. The traditional piled raft is a recognised footing system with piles installed to limit the total and differential settlements of the slab within acceptable limits.

In Sydney a piled raft would typically be considered in areas where paleochannels exists. Such areas are characterised by deep alluvial deposits and generally a high groundwater table. In these conditions Continuous Flight Auger (CFA) piles would be a suitable piling technique and are widely used. However, for CFA piling rigs to operate a working platform is required and, where piles are required to be installed in a basement excavation, the costs associated with establishing the rig into the excavation, constructing a working platform and disposing of the generated spoil are appreciable. In contrast screw piles can be installed by relatively small excavators that are more readily craned into basement excavations and may not require a working platform. In addition, installation is typically quick with no spoil generated.

Common advantages of screw piles are that they are easy to install, require minimal equipment, can be quickly installed and loaded, are suitable for areas with limited access, are removable and reusable, require minimal dewatering, offer high tensile and acceptable compressive capacities, can work on slopes, produce minimal noise and vibration during installation and are cost effective (Livneh and Naggar 2008 and Sakr 2011). Due to these advantages, they have increasingly been used as an alternative to conventional piling techniques on suitable sites to support compressive and tensile loads and control the settlement of raft foundations.

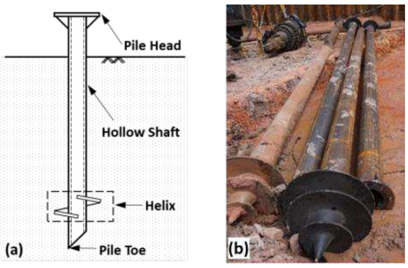

Figure 1. (a) Components of a screw pile; (b) Multi-turn single helix screw pile

Available literature shows that limited studies have been conducted to evaluate the settlement of the raft slab systems supported by screw piles (Mohajerani et al. 2016). Therefore, it is essential for geotechnical engineers to develop a robust design methodology for the reliable prediction of the performance of piled raft foundations supported on screw piles and to manage associated design risks.

This paper presents the methodology adopted for the design of an approximately 1950m^2 piled raft foundation supported by more than 200 multi-turn single helix screw piles (Figure 1 (b)). The piled raft and configuration of screw piles were designed to reduce total and differential settlements to acceptable magnitudes and resist uplift forces on completion of construction and cessation of dewatering.

2 PROJECT DESCRIPTION

The proposed development includes construction of a new five-storey residential building over two-levels of basement parking. Excavation to achieve Bulk Excavation Level (BEL) was anticipated to require cuts to a maximum depth of about 8.7m to achieve the lowest proposed BEL of RL -2.3m, which allows for the construction of a 0.5m thick raft slab.

3 GROUND INVESTIGATION AND GROUND MODEL

The ground investigation program was carried out to accurately characterise the subsurface conditions. This comprised four Cone Penetrometer Tests (CPT1 to CPT4), two Seismic Dilatometer Tests (DMT1 and DMT2) and two boreholes (BH1A and BH2A).

The DMT's were undertaken for accurate settlement characterisation of the soil units under the raft slab, which was a critical factor in SSI analysis. DMT results are noticeably reactive to factors that are scarcely felt (especially in sands) by other tests, such as stress state/history, aging, cementation and structure. Such factors are scarcely reflected e.g. by qc (cone penetration resistance from CPT) and by NSPT, and in general, also due to the arching phenomenon, by cylindrical conical probes. (G. Totani et al. 2001).

The boreholes were primarily used to confirm the depth to bedrock and were terminated after minor penetration (less than 0.5m) into the bedrock. The test location plan is presented in Figure 2.

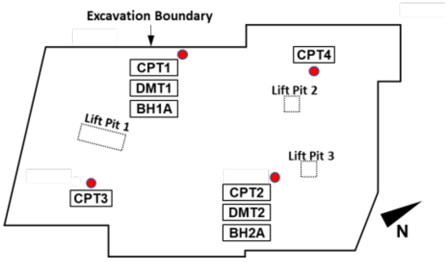

Figure 2. Test location plan of ground investigation program

The CPTs, DMTs and boreholes indicated a subsurface profile generally comprising shallow fill over deep alluvial soils with sandstone bedrock at depths ranging from between about 20.2m to 27m. Alluvial soils, comprising interbedded sands and clays were encountered at all four test locations and extended from the base of the fill to the depth of CPT refusal. The nature of the interbedded soils was relatively inconsistent across the site, with no apparent continuous layers. The sands ranged from loose to very dense relative density but with occasional very loose bands, and the clays from stiff to hard strength. Within several meters below the proposed basement level, the soils typically comprised clay of stiff to very stiff strength and sands of medium dense relative density, but with elastic modulus values of as low as 7MPa being interpreted from the DMT results, which was significantly lower than those interpreted from the CPT test results (i.e. 15MPa). This demonstrates the sensitivity of the DMT test results. The CPT plots are presented in Figure 3.

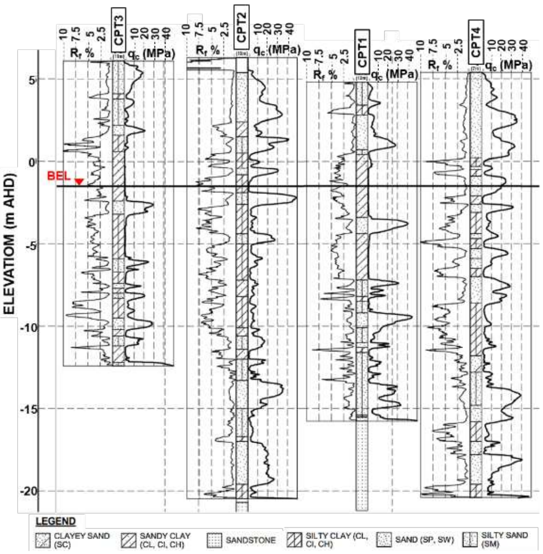

Figure 3. CPT results

Following removal of the CPT rods and probe, standing water was measured in CPT2, CPT3 and CPT4 at depths between 2.7m and 3.2m. CPT1 collapsed on completion of testing to a depth of 0.6m, which is common in a granular soil profile.

Due to the variable subsurface profile and limited strength of some of the soil strata, a piled raft foundation supported on screw piles was proposed as a cost-effective solution to limit both total and differential settlements of the raft slab to within the nominated design criteria.

4 SETTLEMENT DESIGN CRITERIA

The design of the piled raft slab has been completed on the basis of a settlement or serviceability design criteria. The design criteria were nominated by the structural engineer and required that the following not be exceeded:

- Total vertical settlement of raft slab not exceed 50mm.
- Differential vertical settlement not exceed 1/500 of span length.
- Differential vertical settlement between adjacent columns not exceed 10mm.

5 FINITE ELEMENT ANALYSIS

5.1 *Introduction*

3D finite element analysis was carried out using PLAXIS 3D, which is a Bentley Systems software package for 3D geotechnical analysis. Staged modelling was completed to simulate the installation of the shoring system, dewatering, bulk excavation, installation of screw piles, construction of the raft slab and building, which is simulated by the loading of the raft slab for the load combinations provided by the structural engineer. The predicted structural actions and displacements of the proposed raft slab and axial forces in the screw piles were reported.

5.2 *Geometry and Construction Methodology*

The lower basement level is split with the finished floor level (FFL) of the western third of the basement at RL- 1.05m before stepping down to the eastern portion of the slab which has a FFL of RL-1.8m. It is anticipated that excavation to allow the construction of the raft slab will result in BEL’s varying from to RL-1.75m and RL-2.4m. Consequently, excavation for the basement will require cuts to depths of up to approximately 8.7m below existing surface levels.

The shoring system comprised anchored sheet piles installed to a toe level of RL-7.5m. Excavation to BEL was carried out in a staged manner with the progressive installation of one or two rows of anchors as the excavation deepened. Once BEL was reached the screw piles were installed to RL-10m, the raft slab and tanked basement constructed, the structure built and dewatering ceased.

The design and behaviour of the shoring system was excluded from our scope of work. Consequently, staged excavation and progressive anchoring of the shoring system was not modelled with excavation and the installation of anchors completed concurrently and simulated in one stage. In this regard it has been assumed that the shoring walls undergo no lateral deflection following excavation (i.e. the wall is fixed in the x and y direction but free to move in the z direction).

5.3 *Geotechnical Model*

A 3D geotechnical model, as shown in the Figure 4, was developed based on the site investigation results. The geotechnical model divides the subsurface profile into a number of sand and clay units, each with specific geotechnical properties, as shown in Figure 5. Geotechnical parameters were selected for each geological unit based on the results of the in-situ testing (borehole logs, CPT and DMT traces) and empirical correlations well established in geotechnical engineering.

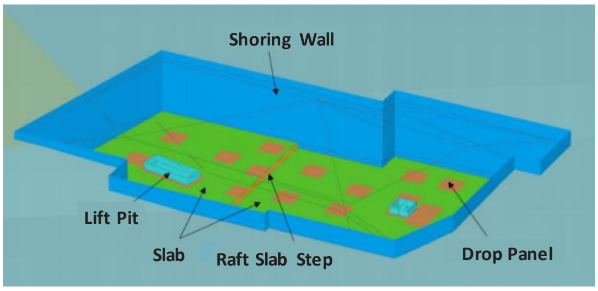

Figure 4. Schematic view of PLAXIS 3D model of excavation

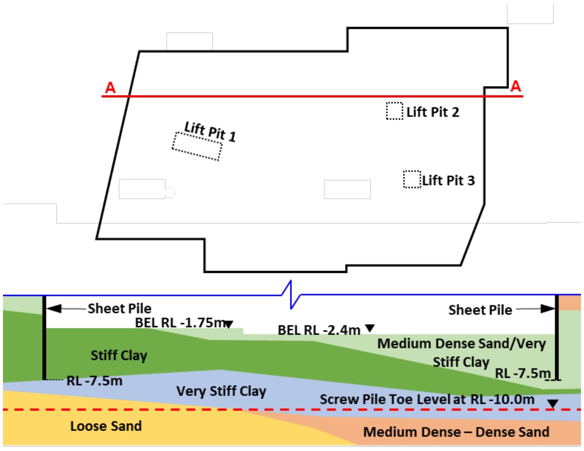

Figure 5. Schematic section (A-A) of adopted geotechnical model in numerical analysis

The groundwater level across the site was modelled at RL3.5m. During dewatering the groundwater level was lowered below the bulk excavation level to RL-4.0m and, following construction, dewatering was ceased and groundwater levels around the basement were allowed to return to RL3.5m.

5.4 *Applied Loads*

The numerical analysis was carried out using the load combinations provided by the structural engineer. For serviceability this included column loads applied to the raft slab and line loads along applied to the top of the shoring wall.

5.5 *Geotechnical Parameters*

The Hardening Soil (HS) model was used to model the behaviour of the soils. The aim of using HS model was to simulate the unloading behaviour of the soil units and estimate the ground movements more realistically. HS model describes the soil stiffness more accurately by using three different input stiffnesses: secant stiffness in drained triaxial test (E_{50}^{ref}), tangent stiffness for primary oedometer loading (E_{oed}^{ref}) and unloading/reloading stiffness (E_{ur}^{ref}). In the HS model, the total strains are calculated using a stress-dependent stiffness. The plastic strains are calculated by introducing a multi-surface yield criterion (Schanz, T. et. al 1999).

The adopted soil parameters in the numerical analysis are presented in Table 1. The depth to sandstone bedrock was adopted as the bottom of the model.

Table 1. Adopted soil parameters in the numerical analysis

Parameter	Material [1]					
(HS)	1	2	3	4	5	6
γ (kN/m^3)	17	17	18	18	18	18
γ_{sat} (kN/m^3)	18	18	19	19	19	19
c’ (kPa)	0	0	0	2	5	2.5
φ (°)	28	30	33	27	27	30
E_{50}^{ref} (MPa)	10	20	100	7	50	70
E_{oed}^{ref} (MPa)	10	20	100	7	50	70
E_{ur}^{ref} (MPa)	30	60	300	21	150	210
m[2]	0.5	0.5	0.5	1.0	1.0	0.8

[1] 1: Fill and Very Loose Sand, 2: Loose Sand, 3: Medium Dense to Dense Sand, 4: Stiff (St) Clay, 5: Very St Clay & 6: Medium Dense to Dense Sands/Very St Clay.

[2] Stiffness stress dependency parameter.

Where soil is in contact with structural elements, a reduction factor (R_{inter}) of 0.67 has been adopted. This is applied to the soil strength parameters to model the reduction in shear strength between the two dissimilar materials.

5.6 *Shoring System and Raft Slab*

The shoring system and raft slab (including the drop panels) were modelled as plate elements. The parameters adopted for the raft slab and drop panels are presented in Table 2.

Table 2. Adopted structural parameters in the numerical analysis

Structural Element	γ (kN/m^3)	E (kPa)	t/D (mm)	ν
Raft Slab	24	2.8×10^7	600	0.2
Drop Panels	24	2.8×10^7	800	0.2
Shoring System	24	2.8×10^7	600	0.2
Sheet Pile	79	2×10^8	65	0.15

5.7 *Screw Pile*

The screw piles consist of a Circular Hollow Section (CHS) shaft connected to a single circular helix plate at the end. The screw pile steel shaft and helix were modelled as a node to node anchor and square (with the equivalent area of a 0.45m diameter circular plate) plate elements, respectively. This approach was adopted due to the difficulty in numerically modelling the screw pile geometry (i.e. hollow shaft and helix plate) and significantly reduced the number of numerical elements and accelerated the development and calculation of the numerical model.

The adopted approach was validated for serviceability load condition by comparing the performance of screw piles with results of Static Load Test (SLT). In this regard the ground conditions encountered at CPT4 were adopted and the maximum serviceability load was calculated from the results of the SLT4 static load test using the method proposed by Davisson (1972). The adopted screw pile parameters in the numerical analysis are presented in Table 3.

Table 3. Adopted parameters for screw piles in the numerical analysis

Screw Pile Element	γ (kN/m^3)	E (kPa)	t (mm)	D (mm)	D_{eq}[1] (mm)	ν
Shaft	79	2×10^8	6.4	168.3	-	0.15
Helix	79	2×10^8	18	450	400	0.15

[1] D_{eq} is equivalent width for a 450mm diameter screw pile.

5.8 *Modelling Stages*

The numerical model was run through a number of stages in an attempt to simulate the existing conditions and the construction staging. These are summarised in Table 4.

Table 4. Adopted modelling stages in the numerical analysis

Stage No.	Description
1	Initial phase to generate the initial stress state
2	Install sheet pile retaining wall
3	Dewater to RL -1.75m, excavate to RL -1.55m
4	Dewater to RL -4.0m and excavate to RL -2.4m
5	Install 0.45m diameter screw piles to RL -10.0m
6	Construct raft slab and drop panels
7	Install lift pit retaining walls
8	Excavate lift pits and construct the base of the lift pits
9	Apply load cases

The model displacements were reset to zero at the end of Stage 8 to allow displacements of the piled raft slab to be compared to the settlement design criteria. By resetting displacements at this stage, deformation associated with the construction of the piled raft slab are modelled but those that will not affect the serviceability of the slab (i.e. rebound of the subgrade following excavation, settlement of the slab under its own self-weight whilst the concrete is still wet etc.) have been ignored. A snapshot of the 3D numerical model is presented in Figure 6.

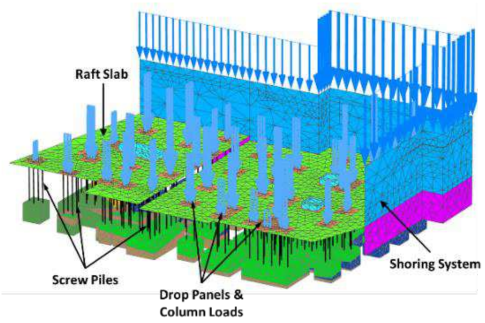

Figure 6. A Snapshot of the 3D numerical model

6 CALIBRATION AND MONITORING PLAN

6.1 *Static Load Testing*

The numerical analysis adopted a 3D ground profile by extrapolating between the subsurface condition encountered at the four CPT's and two seismic DMT's. Based on this testing it is clear that there is significant variability, not just in the soils encountered between the test locations but also in the uniformity and continuity of the soils across the site.

To provide greater confidence in both the subsurface conditions modelled and the material parameters adopted, four static load tests (SLTs) were completed in close proximity to the CPT locations on sacrificial screw piles. These were completed prior to the installation of the footing piles and allowed both calibration of the adopted geotechnical model and confirmation that the capacity and load/displacement response of the screw piles was consistent with the results of our numerical analysis. This data was also used to validate the adopted approach for numerical simulation of single helix screw piles. The static load testing set-up and the results of one of these tests is presented in Figures 7(a) and 7(b), respectively.

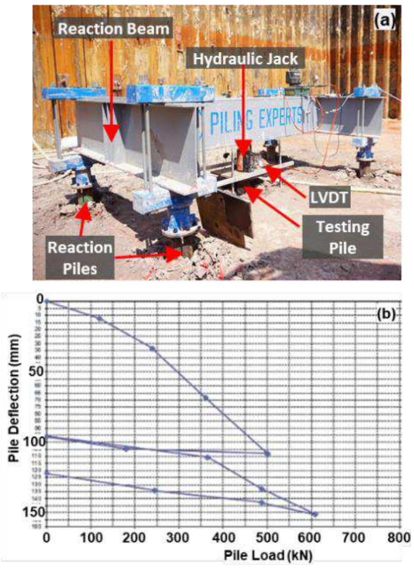

Figure 7. (a) Static load testing set-up & (b) Results of SLT4 in vicinity of CPT4 location

6.2 *Torque Readings*

Torque readings were taken during the installation of the screw piles and were also used to verify and calibrate the adopted geotechnical model in the numerical analysis. Piles were initially installed at the CPT locations and the torque readings from these piles were used to develop a site-specific correlation between the torque readings and material types and relative density/strength. With this correlation, greater confidence of the ground conditions between the test locations could be achieved and, where necessary, the model amended to reflect actual site conditions. The piling contractor was requested by the geotechnical designer to provide a complete record of pile torque readings on installation of the following pile groups.

I. Torque readings for all screw piles installed for the static load tests.
II. Torque readings for the first twenty screw piles. The locations of these screw piles were defined by the geotechnical engineer in a way to cover the entire site.
III. Torque readings for the remaining screw piles upon their installation.

It should be emphasised that the torque readings and their correlation with CPTs and DMTs results were used as a general indication to validate the adopted ground profile in the numerical analysis and in isolation to verify pile capacity or soil properties. While torque readings are generally not a good guide to pile capacity, calibrated torque readings do provide an indication of both the soil type and stiffness through which the piles have been installed. It is recognised that this is an approximate approach and that there is some ambiguity between clayey and sandy soils of differing strength and relative density. However, it allows an assessment of the materials through which the piles are installed to be made and the suitability of the ground model checked. While we recognize that this relationship between soil type and stiffness does not provide a definitive characterisation and must be used with caution, it does provide additional information that allows us to develop greater confidence in validity of our model.

6.3 *Survey Monitoring Plan*

Survey monitoring was carried out during the construction phase to compare the actual behaviour of the piled raft system with those predicted. This monitoring was completed at the following stages:

- Following pouring of the raft slab.
- Following pouring of the floor slabs (i.e. basement 1 Floor Slab, Ground Floor Slab etc.) and on completion of the building.

7 RESULTS

7.1 *Raft Slab Movement*

The maximum total and differential settlements of the raft slab for both dewatering and uplift conditions for critical load combinations were extracted from the numerical analysis and reported to the structural engineers. The vertical displacement contour plot for the piled raft slab for a critical load combination is presented in Figure 8.

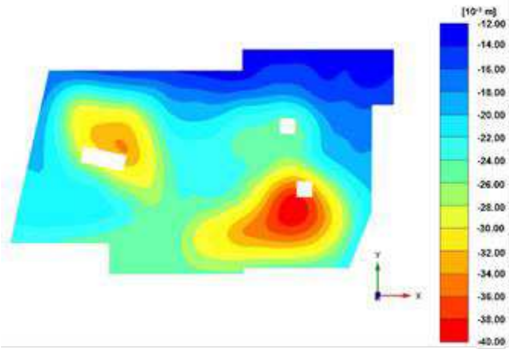

Figure 8. Vertical displacement across the raft slab for a serviceability load combination

7.2 *Raft Slab Structural Actions*

To aid in the design of the raft slab, the performance of the slab under the various load cases was modelled and the induced shear forces and bending moments predicted. The heat maps of the predicted shear forces and bending moments across the raft slab for the critical load case were extracted from the 3D model and forwarded to structural emigres to complete the structural design of the raft slab. The heat maps of the predicted shear forces and bending moments experienced by the raft slab for the critical load combination are presented in Figure 9.

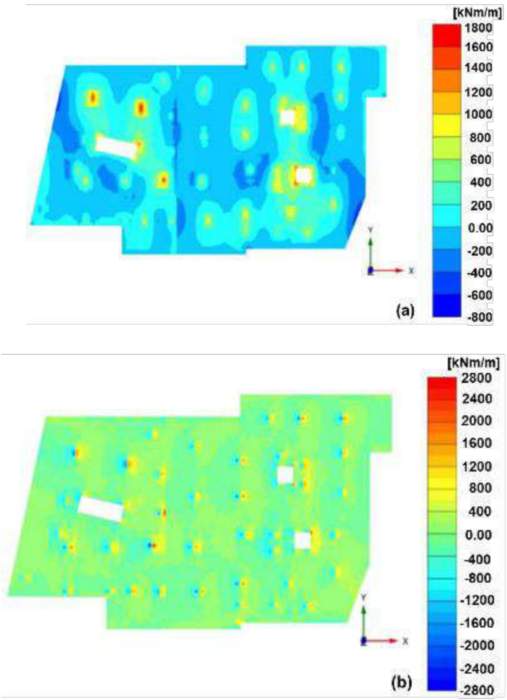

Figure 9. Predicted structural actions across the raft slab for the critical load combination; (a) bending moments, and (b) shear forces

7.3 *Screw Pile Forces*

Several iterations were required before the screw pile arrangement satisfied the settlement design criteria. While the use of a node to node anchor meant that only axial forces are calculated, due to the very slight curvature of the slab under loading bending moments generated in the shaft of the piles are small. Consequently, for this project bending moments were not significant for this project. This may not be the case where piles are installed at least in part through weak soils that provide only limited lateral restraint to the pile shaft. The settlements induced in the slab under the various load cases, the arrangement of the piles and the axial forces in the piles for all the load combinations were reviewed by the structural consultant to ensure that the arrangement satisfied both the design criteria and provided a buildable solution.

8 COMPARISION OF NUMERICAL ANALYSIS RESULTS WITH MONITORING RECORDS

Survey monitoring of the surface of the raft slab was commenced following its construction and continued during the construction of the remainder of the structure. The monitoring results have been reported to the geotechnical design team on a regular basis to be compared with the numerical predictions. Figure 10 presents the monitoring results following completion of construction of the structure. These are overlaid on the predicted vertical displacement heat map for the load combination of 1G+1Q after termination of the dewatering process.

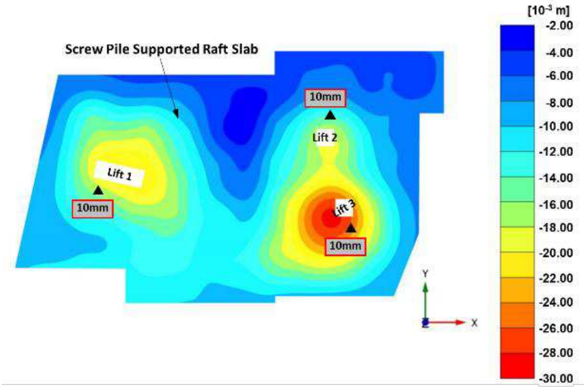

Figure 10. Survey monitoring results overlaid on the predicted vertical displacement heat map - 1G+1Q uplift condition

It can be noted that there is a good agreement between the predicted displacements and the monitoring results close to the lift pits 1 and 2. The settlement predictions around lift pit 3 is higher than the recorded settlements, which appears odd considering the loads applied to the slab at this point. This could be due to stiffer subsurface conditions being present at this location than have been captured in our numerical model.

9 DESIGN FLOWCHART

A design flowchart is proposed in Figure 11, which summarises the adopted approach to conduct the SSI analysis for piled raft foundations supported on screw piles.

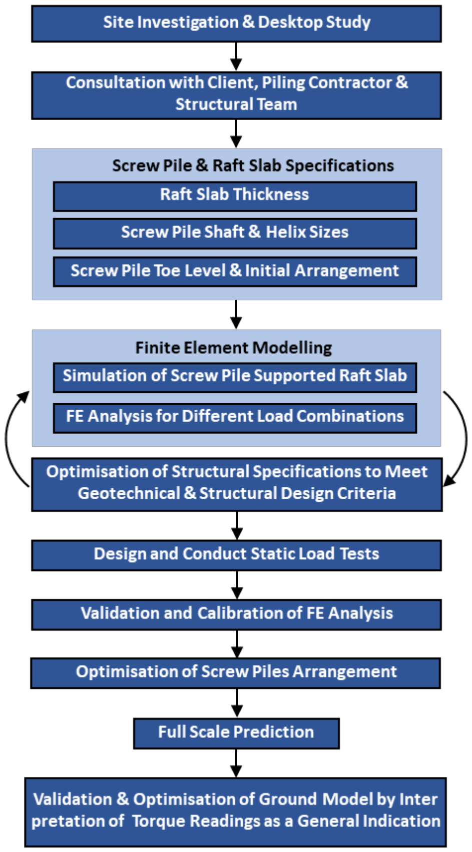

Figure 11. Design flowchart for SSI analysis for piled raft foundations supported on screw piles

10 CONCLUSIONS

This paper presents the adopted methodology for the design of a piled raft foundation supported by more than 200 single helix screw piles. It also describes how the screw piles have been modelled using node to node anchors and square plate elements.

The importance of the design methodology was centred around the principal of continuous feedback to verify the analysis, not only over the design period but also the construction stage of the project. In this regard, while a comprehensive ground investigation program was undertaken to initially develop the geotechnical model, the assumptions inherent in this model were continually checked during the construction period and the model amended where necessary. This feedback was derived initially from the static pile loads tests and then from the site-specific correlation derived between the screw pile torque readings and the soil units. Survey monitoring of the slab during construction also provided feedback on the performance of the slab during the loading stages. A design flowchart is also proposed, which summarises the design methodology when undertaking SSI analysis for design of piled raft foundations supported on screw piles.

Whilst the use of node-to-node anchors meant that only axial forces were measured, the very small curvature of the deflected slab meant that bending moments induced in the shaft of the piles were not significant.

The comparison between the results of the numerical analysis and survey monitoring indicated a reasonably good correlation, although the monitoring did not show the same variability as predicted in the model. This suggests that the SSI analysis has fairly well predicted the behaviour of the foundation although there is some questing regarding the apparent uniformity of the survey results and the more variable predicted deflections.

11 ACKNOWLEDGEMENTS

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