Foundation Investigation and Analysis for Tall Tower Developments

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ABSTRACT: Many tall buildings are supported on piled rafts and / or deep bored cast in situ piles. Good engineering design requires soil-structure interaction analysis and a clear understanding of the factors controlling the performance of the footing system. These rely on a sound understanding of the ground characteristics and individual and group pile performance, including adequate collection of data and testing, which can only be achieved through detailed and targeted ground investigation and in situ testing. This paper focuses on the ground investigation methods available and how the results are used to achieve a reliable estimate of footing system performance using soil-structure interaction analysis. It highlights the importance of accurate inputs into the analyses, especially in respect to the stiffness characteristics of the ground and the load displacement performance of individual piles. This is illustrated through a number of case studies of tall tower projects that the authors have been involved in.

KEYWORDS: Geotechnical investigation, Soil-structure interaction, Tall towers, Analysis, Design, Case study.

1. INTRODUCTION

Driven by the ongoing trends of urbanisation and population growth, there is an increasing global demand for tall towers. The pressure on land use and the preference for accessibility of new developments to public transport links has resulted in the construction of tall towers (defined herein as buildings of approximately 50 levels or greater) in areas of major cities which had not hitherto seen such construction. An example of this is the South Bank area of Melbourne, Australia, a reclaimed swamp area which has seen significant development in the past twenty years.

These trends have led to considerable challenges for the geotechnical and structural engineers undertaking the design of tall towers. However, developments in our ability to assess the engineering properties of the ground and to analyse and quantify the soil-structure interaction has allowed tall towers of greater heights to be constructed, and in areas in which the geological complexity had led others to believe that such construction was not feasible.

This paper provides a summary of the authors' recommendations for the assessment of the engineering behaviour and properties of the ground through geotechnical investigation, and a discussion of some of the important considerations in the analysis of tall towers. It also describes the importance of these, in addition to close collaboration between geotechnical and structural engineers, in the context of two prominent case studies.

2. GEOTECHNICAL INVESTIGATIONS

2.1 Background

The development of a scope of geotechnical services to support the investigation and analysis of footing systems and, if required, basement retention systems, requires a fundamental understanding of the likely geological conditions and properties of the subsurface materials at the site of the proposed development. It also requires consideration of the loads (both axial and lateral) that will be applied to the subsurface materials via the buildings foundation system, and in the case of basement retention, the retaining walls. It is critical that both the temporary and permanent conditions are considered with respect to subsurface soil and rock properties and loads.

2.2 Key geotechnical issues and considerations

2.2.1 Introduction

Based on the authors' considerable experience with the design and construction of towers (with and without basements) in a variety of geological settings, the following issues are considered to be important in developing the aims and scope of geotechnical investigation and analytical services.

A preliminary desktop assessment, development of a geological history, and conceptual ground and foundation models are essential first steps to any geotechnical services provided for tall tower developments. These aid the identification of key risks and the development of an appropriate scope for the geotechnical investigation works.

The design of foundations (either spread footings or piles) for any building but particularly for tall towers (which require competent founding strata) should be based on serviceability criteria (i.e. the Serviceability Limit State, SLS) as the Ultimate Limit State (ULS) is unlikely to be critical for foundations on such material. The scope of any geotechnical investigation must be adequate to reasonably define the subsurface stratigraphy and the relevant engineering properties (especially those related to the load deformation behaviour) of the soils and rocks within the zone of influence of the proposed building. For tall towers, it is generally insufficient to drill boreholes using auger or double tube coring techniques and to undertake occasional sampling and standard penetration testing. Such an investigation provides no reliable information on the load deformation behaviour of the ground.

Instead, the drilling, sampling and testing techniques that are adopted must be of sufficient quality and number to reasonably identify the key parameters affecting the performance of the foundations of the building. Such an investigation could comprise drilling of boreholes with the usual sampling and testing in the soil materials (for example, undisturbed tube samples, standard penetration testing), adopting triple tube coring immediately on encountering materials that are competent enough to recover samples from (i.e. in extremely weak rock) for identification and laboratory testing, and performing both borehole imaging (if appropriate and necessary, refer to Section 2.2.5) and high quality high pressure pressuremeter testing. Following completion of the borehole investigation, crosshole seismic testing and preliminary pile testing can be undertaken to provide further information and to potentially allow optimisation of the foundation design.

The number and depth of boreholes (and the in situ and laboratory testing undertaken) is dependent on the geological conditions at the site, the potential for variability in subsurface conditions across the site, the nature of the development including the anticipated footing system, and if applicable, the basement retention system.

The depth of boreholes should be based on an understanding of the geological conditions at the site and should be of sufficient depth to reasonably define the subsurface conditions within a depth of about twice the shortest plan dimension of the building below the base of the footing system. In most situations, where the geology is relatively well defined and in which the stiffness and strength of the ground increases with depth below the surface, the authors would expect any boreholes to extend a minimum of 5 m below the anticipated founding level of any shallow / spread footings and 2 m to 5 m (depending on the reliance on base resistance) below the toe level of any piles. In more complex ground conditions in which more competent materials (for example, basalt rock) overlie less competent materials, or where the ground conditions cannot be reasonably anticipated based on the geology, a number of significantly deeper boreholes may be required.

The number of boreholes should reflect the size of the site and the potential for variability in subsurface conditions across the site.

2.2.2 Subsurface conditions

In developing the scope of any geotechnical investigation, it is critical to understand the impact of the geological conditions on the proposed foundation and retention systems. Whilst subsurface conditions may vary significantly, most tall towers are supported at some depth on reasonably competent material, such as weak or weathered rock (or better). For the purposes of this paper, four separate scenarios or subsurface conditions have been considered, and are described as follows:

- Competent strata (i.e. weak rock or hard soils) of sufficient strength and stiffness at a shallow depth relative to any proposed basement excavation such that spread footings may be adopted.
- Relatively competent strata (i.e. extremely to very weak rock or very stiff, very dense or hard soil) at a shallow depth relative to any proposed basement excavation, but of insufficient strength or stiffness to allow spread footings to be adopted; hence a piled raft may provide a satisfactory footing alternative. In this situation, the authors anticipate that the applied axial loads may be shared between a traditional raft with settlement reducing piles beneath heavily loaded columns or cores, to limit settlements to acceptable levels.
- Significant depth to suitably competent strata relative to any proposed basement excavation such that piled foundations are required to support the design axial building loads. On the assumption that the soils overlying the competent strata do not comprise soft or loose to very loose sediments, a variety of pile construction methods can typically be adopted. Standard pile types regularly used under these circumstances include bored piles and continuous flight auger (CFA) piles socketed into a founding rock stratum or driven pre-cast concrete piles driven to refusal.
- Significant depth to suitably competent strata relative to any proposed excavation such that piled foundations are required to support the design axial building loads. In the situation where there is a significant depth of soft or loose to very loose sediments, the resistance of the foundation system (and soils over a depth of say 10 m to 15 m) to base shear or lateral loading particularly under transient loading conditions such as wind or earthquake can be critical. Under such conditions, large diameter piles (1200 mm to 1800 mm or larger) are usually required due to their increased resistance to lateral loading.

2.2.3 Assessment of engineering properties of founding material

On the basis that serviceability criteria are likely to be the critical consideration for assessment and design of foundations, the authors consider that high quality sampling and testing and high quality, high pressure, in situ pressuremeter testing are a critical component of any proposed geotechnical investigation.

In situ pressuremeter testing together with appropriate laboratory testing (such as unconfined compression testing) can be used to measure the deformation characteristics of the in situ rock mass at discrete test locations. The results of the pressure expansion curve taken at the initial portion of the curve can be used to assess an initial elastic modulus, whilst unload-reload data can also be obtained to investigate the deformation characteristics under unload-reload conditions.

Where practical, in situ pressuremeter testing of the founding stratum should be undertaken in all boreholes with sufficient tests taken to allow characterisation of the deformation characteristics of the rock mass to be assessed over the zone of influence of the anticipated foundation scheme. A minimum of two pressuremeter tests per borehole where spread footings are proposed, and four tests per borehole (over a depth range of about 10 m) where bored piles socketed into competent founding material, are typically anticipated.

For situations where a piled raft footing system is the proposed footing alternative, pressuremeter testing should be undertaken at regular intervals (for example, every 3 m to 5 m) or in each competent stratigraphic layer over the full depth of every borehole.

In some ground conditions (for example, very deep granular or bouldery alluvium) and in some countries, quality sampling and quality pressuremeter testing may be unavailable or impractical to undertake. In such cases, high-quality crosshole seismic testing should be undertaken to assess the small strain stiffness profile with depth of the ground. The small strain stiffness cannot be used directly to assess stiffness at larger engineering strains, but can be used to estimate such values (as shown in the Nakheel Tower case study described in Section 4). Such testing usually requires installation of three boreholes set about 5 m apart. Only one of the boreholes is required to be a geotechnical investigation hole, the others may be drilled to full depth without coring or in situ testing. It is important that the verticality of each borehole is measured as the interpretation of the crosshole seismic testing relies on an accurate assessment of the distance between the source (in one borehole) and the receivers (in the other boreholes). Crosshole seismic testing is still of significant benefit when quality pressuremeter testing is available, especially for the assessment of piled raft performance.

Whilst the above discussion has concentrated on assessing the relevant properties of the competent strata layers which are fundamental to the likely settlement performance of a tower, when the near surface materials comprise soft or loose deposits, it is important that such materials are also investigated to better assess the lateral performance of the footing system under wind and earthquake load.

2.2.4 Resistance of the foundation system to lateral loading

Where buildings are of a significant height (say greater than 50 levels) and there is a significant thickness (say 10 m to 15 m) of soft or loose to very loose sediments, the resistance provided by the building foundations (piles) to lateral loading from wind or earthquake loading is likely to be a critical factor in the design of the building.

Lateral restraint to applied lateral loading from wind or earthquake can be provided by piles and pile caps. The response of a pile or group of piles to lateral loading is often governed by the allowable lateral deformation at the top of the pile rather than the ultimate lateral resistance provided by the soils. Where low strength or loose sediments are present over the upper portion of deep piles the allowable lateral movement of the foundation system can become critical. For these reasons, large diameter bored piles reinforced over their full depth can provide increased lateral resistance and hence help to reduce lateral deformations compared to partially reinforced smaller diameter CFA piles.

For developments that meet the criteria described above, three-dimensional (3D) numerical modelling of the soil-structure interaction (foundation response) under lateral loading is considered to be critical to the assessment of the foundation system.

Standard investigation techniques (for example, cone penetration tests (CPTs), boreholes with undisturbed tube samples recovered for laboratory strength and consolidation testing) are usually required to investigate the nature of the weaker sediments and provide parameters for analysis of the foundation system.

Where tall towers incorporate deep basements, the development of a geotechnical investigation also must consider the potential impact of sedimentary rock deposits, high in situ horizontal stresses and groundwater management. For the case where basement excavation is within weathered bedded and folded / faulted rock and extends say three or more levels below the top of the rock, additional investigation to assess the direction the dip angle of the sedimentary rock and the in situ stress in the rock are recommended.

2.2.5 Direction and dip angle of bedding in sedimentary rock

Recent experience on projects with deep excavations in weathered sedimentary (bedded) rock deposits has indicated potential for significant movement or instability of excavations due to movements on bedding planes that by previous conventional assessment methods would not have been classified as being at risk.

Therefore, for deep excavations in weathered sedimentary deposits, it is considered both prudent and necessary to undertake borehole imaging (acoustic televiewer) to investigate the direction or orientation of bedding and the dip angle of bedding within the rock mass. The authors are aware of some asset owners requiring this information in assessing protection works notices. The other advantage of undertaking this increased level of investigative works is that where the direction of the bedding is favourable to the excavation (i.e. into the excavated face), the basement retention system may benefit from the stability of the weathered rock which is essentially self-supporting, i.e. pressures on the retaining wall can be reduced in design (typically quantified by the use of numerical modelling).

2.2.6 In situ horizontal stresses

The experience of the authors with deep basements in a range of rock types for a number of tower projects indicates that the in situ horizontal stress present within the rock mass is often underestimated, especially in rock masses that have been compressed laterally and are folded and faulted. As a result, deformations of the basement retention system can be significantly underestimated. Whilst there are techniques available to measure in situ horizontal stresses, it is the horizontal strain that must be fully understood. This is beyond the scope of this paper, but is consider further in Lochaden *et al*. (2019).

2.2.7 Groundwater management

It is common for basements in competent founding materials to be designed as drained basements with groundwater intercepted by passive drainage behind retaining walls and a subfloor drainage system. These passive systems typically drain to a sump(s) within the basement with the accumulated water pumped off-site either to sewer via a trade waste agreement or to stormwater via the legal point of discharge. Unless inflows are likely to be excessive, the potential rate of inflow is often not investigated in detail.

The recent experience of the authors in Australia is that there is an increasing reluctance from water authorities to accept groundwater from drained basements into either the sewer or stormwater, and hence it may not be feasible to construct future developments as drained basements.

It is likely that applications for the ongoing off-site disposal of groundwater will require information on both the water quality (i.e. environmental considerations) and an estimate of the potential rate of groundwater inflow. Groundwater drawdown is also an important consideration in the adoption of a drained basement due to the potential for the consolidation and associated settlement of low strength cohesive materials. However, as basements in such materials are typically constructed as a sealed basement, this is not discussed further herein.

Given budget constraints around private building developments, large scale pump tests are often not economically practical, and hence estimates of the mass permeability of the ground are often made based on in situ permeability tests in standpipes (i.e. slug tests). The results of the in situ permeability tests can then be used in numerical modelling programs, to assess potential rates of inflow, with the results from such programs validated by simple hand calculations. Given the sensitivity of the hydraulic conductivity parameter, sensitivity analyses are highly recommended.

Potential groundwater inflows in fractured rock are difficult to estimate with confidence as the rate of inflow is often controlled by the extent and nature of discontinuities within the rock mass. Hence, assessment can often only be made by estimating an overall permeability for the rock mass. An order of magnitude variation in the permeability of the rock mass will result in an order of magnitude variation in the estimated rate of groundwater inflow. To this end, in situ permeability testing in a series of 100 mm diameter boreholes then must be extrapolated to a basement with a wall and floor area of several thousand square metres. Where practical, the review of data from existing basements (in similar geological / sub surface conditions) with respect to rates of off-site disposal of groundwater can be a valuable means to assist the validation of in situ permeability testing and back-calculation of a mass permeability for materials around the basement.

3. ANALYSIS

3.1 Introduction

The analysis of individual shallow footings is reasonably routine for the majority of geotechnical engineers. The settlement of such footings under the loads applied by tall towers may be reasonably approximated by simple closed-form hand calculations, but of course is dependent on the adoption of an appropriate stiffness. The design of a shallow footing system for tall towers, however, involves the interaction between multiple shallow footings, and more sophisticated analysis methodologies are typically adopted in such cases, such as two-dimensional (2D) and 3D numerical analysis. Similarly, more sophisticated analysis methodologies are typically adopted for the assessment of pile group effects.

The assessment of feasibility and the preliminary design of piled / piled raft foundations is typically undertaken using simple closed-form hand calculations (see Poulos, 2001) and 2D finite element analyses which may adopt either plane-strain or axisymmetric conditions. Such methods are useful to assess the approximate required number, diameter, length and location of piles. 3D numerical analysis is now routinely adopted for detailed design, in part due to the availability and relatively low cost of computational power.

3.2 Assessment of pile behaviour

In order to have confidence in the design, it is critical to assess the suitability of the design methodology adopted to model the pile load displacement behaviour. This is typically undertaken by the comparison of the calculated to the known pile load displacement behaviour. The known pile load displacement behaviour will ideally consist of data from site-specific fully instrumented test piles. However, it is the experience of the authors that such data are not routinely available. Comparison of the calculated pile load displacement behaviour to pile data from adjacent sites and sites with similar ground conditions may therefore be required.

The means by which commercially available software consider the interface between the pile and the soil / rock is an important consideration in such a back-analysis. These means have been described by Haberfield & Lochaden (2018) with respect to the PLAXIS suite of software (the use of which has grown significantly in the past ten to fifteen years, to the extent that it is now routinely globally used to model the load deformation performance of piles), and are summarised briefly below.

A pile may be considered as a volume element or as an embedded beam (the latter applicable only for 2D plane-strain and 3D models). When a volume pile is adopted, an interface with a reduced strength and stiffness in comparison to that of the adjacent soil / rock mass is adopted to consider the shear zone between the structural element and the adjacent soil / rock mass. The behaviour of the interface is typically described by linear elastic perfectly plastic behaviour. When an embedded beam is adopted, the interaction of the pile shaft and base with the adjacent rock is described by linear elastic behaviour with a finite strength and linear elastic perfectly plastic behaviour, respectively. The maximum allowable shaft and base resistance which the embedded beam can mobilise is provided as an input by the user. Alternatively, the shaft resistance can be related to the strength properties of the rock.

Haberfield & Lochaden (2018) conclude that modelling of the pile as a volume element allows the designer significantly more flexibility in the back-analysis of the results of a pile load test, as the stiffness of the interface may be altered without altering that of the adjacent soil / rock mass, and the strength properties of the interface may be modified. However, the adoption of the volume element is more complicated (with respect to geometry and meshing) than the adoption of the embedded beam.

Axisymmetric 2D analysis in which the pile is modelled as a volume element is a useful tool to ensure that the calculated pile load displacement behaviour is similar to the known behaviour. The interface properties required to reasonably match the calculated to the known pile load displacement in the 2D analysis may then be confirmed in a 3D analysis of a single pile. Alternatively, if the pile is modelled as an embedded beam, or if the computational time is not deemed to be excessive, 3D analysis of a single pile under loading may be undertaken without undertaking axisymmetric 2D analysis.

3.3 Selection of ground model

The purpose of this section is to provide a brief discussion of some of the more important considerations in the selection of a model which adequately describes the behaviour of the soil / rock (termed “ground” herein, hence ground model) for the analysis of piled / piled raft foundations. More general guidance on this aspect may be found elsewhere (for example Lees, 2017).

All ground models provide only an approximation of the true behaviour of the ground. An increasing degree of complexity is not necessarily either desirable or required, due in part to the potentially extensive and sophisticated nature of testing which may be necessary to assess the inputs to such models. There are many case studies reported in the literature in which very simple constitutive models have been adopted to great effect (for example, Lochaden *et al*., 2019). An appropriate ground model should be capable of modelling the critical aspects of the behaviour of the ground for the specific case under consideration.

An appropriate assessment of the stiffness of the ground is of critical importance to the design of the basement and footing system for a tall tower, and as described in Section 2, is a primary consideration in the scoping of an appropriate geotechnical investigation. One of the behavioural aspects which the authors consider to be of primary importance in the design of tall towers is the tendency for the stiffness of some materials to be higher in unloading and reloading compared to virgin loading. This tendency may be assessed in the laboratory (for example, oedometer and triaxial testing) and in the field using pressuremeter testing. This effect may be considered in an analysis by either manually changing the stiffness value for the material in question based on consideration of the stress history, or by adopting a constitutive model which automatically considers the stress dependency of the stiffness.

Many tall towers have deep basements constructed in rock. In Melbourne, for example, many such basements have been constructed in the Melbourne Formation (MF), the weak weathered siltstone which underlies parts of the city. Haberfield (2017) discusses the engineering implications of the bedding planes present in the MF primarily with respect to basement design and construction. These bedding planes can be persistent for significant lengths (greater than 100 m) and with measured strengths as low as 12°. Such rock is typically considered as either a continuum (i.e. the bedding planes are not explicitly modelled, and relatively lower strength and stiffness values are adopted for the rock), or as a discontinuum (i.e. the bedding planes are explicitly modelled, and relatively higher strength and stiffness values are adopted for the rock). Whilst both of these approaches are valid for the design of basement retention systems, the authors consider that the discontinuum approach is preferable for an analysis in which the footing system of the tall tower is being assessed. The adoption of a continuum approach results in calculated displacements which are significantly greater than those measured, due primarily to the reduced strength and stiffness values which are adopted in such an analysis.

3.4 Limit state design

The adoption of limit state design is now required by many design codes globally. However, many of these design codes provide little guidance on how limit state design should be appropriately considered in soil-structure interaction analyses, noting that it is the understanding of the authors that this will be rectified in revisions which are currently underway to both the Australian standard for retaining wall design (AS4678, 2002) and Eurocode 7. The analysis of the basement / footing system for a tall tower should be based on reasonable and appropriate geotechnical strength and stiffness inputs. The authors therefore consider that a material factoring approach, that is the adoption of shear strength parameters (and stiffness, in some design codes) which have been reduced by some factor, is not appropriate for such analyses. Instead, prudently conservative best-estimate values should be adopted for design. The structural ULS may then be considered by applying an appropriate factor to the calculated structural actions. The geotechnical ULS should be considered by reducing the strength of the materials in a stepwise fashion until a valid failure mechanism is deemed to have occurred. Further information is provided in Lees (2013).

4. CASE STUDY – NAKHEEL TOWER

4.1 Introduction

The Nahkeel Tower in Dubai was designed to extend to a height in excess of 1 km. With about 2,000,000 tonnes dead load (DL), the structure would have been one of the heaviest ever built. The project was placed on hold in early 2009 and construction is yet to recommence. However, ground engineering works which have been undertaken include the ground investigation and development of site conceptual model, construction and testing of instrumented trial barrettes, assessment of the ground response under the tower loading, the design of a system of barrettes to control ground response, tower settlement and tilt, and construction of approximately half of the foundations.

This section of the paper briefly discusses the ground investigation undertaken for the project, how the constitutive model for the ground behaviour was developed, and the methods used to assess soil-structure interaction.

The foundation system concept adopted for the tower was a piled raft. The raft design had a variable thickness of up to 8 m under the most heavily loaded structural elements, and founded at a depth of about 20 m below ground level at the base of a 120 m diameter excavation supported by a circular (in plan) embedded diaphragm wall. Approximately 400 barrettes were proposed, for installation to depths of between approximately 60 m and 80 m below ground surface.

4.2 Geotechnical investigation

4.2.1 Desktop study and scope of geotechnical investigation

A desktop study of available information indicated that the geology at the site was typical of that of Dubai and generally comprised recent aeolian deposits overlying shallow marine deposits, inferred to be of Quaternary age and comprising predominantly a weak, unweathered carbonate rock called calcisiltite. These conditions are similar to those at the Burj Khalifa tower, which is currently the tallest building in the word, as described in Poulos (2016).

The scope of the geotechnical investigation for the proposed tower was based on the assumption that a piled raft foundation system would likely be adopted. The depth of the piles was expected to be less than 100 m. Preliminary analyses of a piled raft indicated that the large diameter of the building and high loads would potentially stress the ground to depths in excess of 200 m. Consequently, it was important that the geotechnical investigation focused not only on the stiffness of materials below the raft and within the depth of the piles, but also on the ground below the piles, and boreholes to 200 m depth were proposed.

Past geotechnical investigations in similar ground conditions in Dubai usually comprised double tube coring in the calcisiltite which resulted in the recovery of broken core samples and suggested potentially fractured ground. However, this was inconsistent with the young geological age of the material, which suggested that the calcisiltite should be relatively unfractured and that it was the double tube coring that had resulted in the fracturing. As the fracturing of the material affects the stiffness of the weak rock mass, a better method of drilling was required and triple tube drilling was adopted. The subsequent core recovery using triple tube coring confirmed that the ground was relatively homogenous and free of discontinuities.

As understanding the load deformation characteristics of the ground was essential to the performance of the proposed piled raft, it was considered necessary to undertake in situ pressuremeter testing, crosshole seismic testing, quality laboratory testing of recovered samples, and full-scale pile load testing.

4.2.2 Scope of investigation

A total of nine boreholes of between 120 m and 200 m depth were drilled. All boreholes were advanced using PQ, HQ or NQ triple tube drilling techniques, with the borehole diameter varying depending on the type of in situ testing scheduled.

Pressuremeter testing was undertaken in three boreholes at depth intervals of about 5 m. At least one unload / reload loop was incorporated into each test, and creep tests were undertaken as part of about 30% of tests, in which the pressure was held constant for up to 2 hours whilst displacement was measured.

Crosshole seismic testing was undertaken at two locations to depths of 200 m below ground level. Arrays of 3 boreholes with 3 m between each borehole were used for the crosshole seismic testing.

Laboratory testing undertaken on samples recovered from borehole core included moisture content, bulk density, particle density, point load testing, high pressure oedometer, constant normal stiffness direct shear testing, resonant column and cyclic triaxial testing and an array of chemical testing on soil and groundwater. Stiffness was measured in the laboratory using primarily Unconfined Compressive Strength (UCS) tests with end platen displacement measurement.

4.2.3 Stratigraphy

The boreholes indicated that the general subsurface stratigraphy comprised of:

- An upper 6 m thick layer of loose saturated sand. This unit is subsequently referred to as Unit A. A surficial layer of precipitated gypsum and other salts forms a thin crust at the surface of the site.
- Recent aeolian deposits comprised of carbonate rich sand with thin, high strength indurated layers. This forms a capping layer over the site. The sand extends from ground surface to a depth of about 20 m. This unit is subsequently referred to as Unit B.
- Shallow marine deposits, inferred to be of Quaternary age and comprised of predominantly calcisiltite unconformably underlie Unit B. This material is a low strength rock with carbonate content typically greater than 70%. It extends to a depth of about 70 m below ground surface.
- A second shallow marine sedimentary sequence underlies Unit C and extends to the maximum depth investigated of about 200 m. This unit is comprised predominantly of calcareous siltstone with some calcisiltite. Although the carbonate content is variable, it is typically lower than that of Unit C. This Unit is characterised by high gypsum content. Gypsum is present as massive layers of up to 2.5 m thick, as well as nodules and veins. Borehole correlation between the massive gypsum layers suggest the bedding within this material has a shallow dip of about 8°.

Units C and D described above are generally massive. Some tight, closed joints are present within these units which are thought to have formed as a result of stress changes during burial. There were no tectonic induced discontinuities observed. The general site stratigraphy is presented in Figure 1.

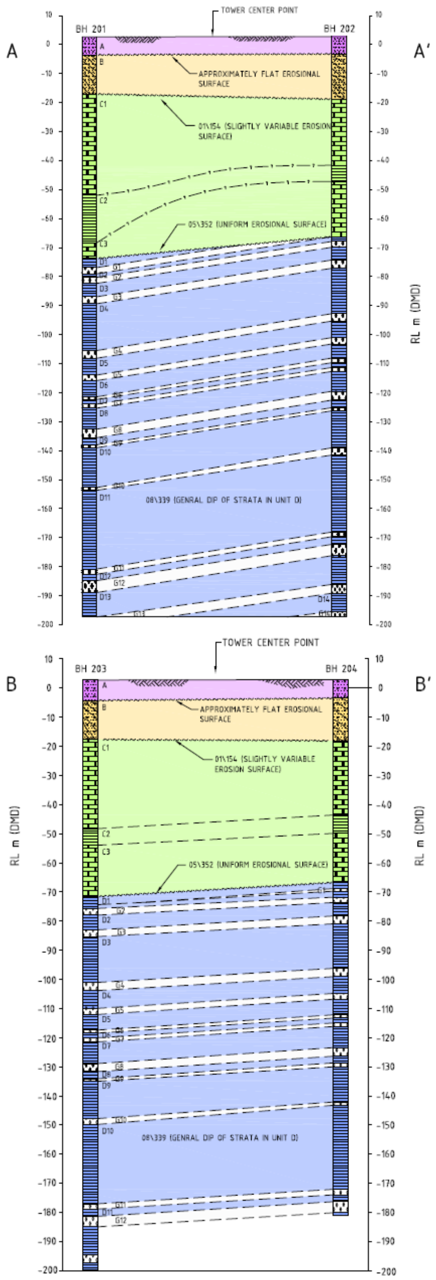

Figure 1 Orthogonal cross sections through the site showing general stratigraphy

4.2.4 Sampling and testing

During the geotechnical investigation, samples of soft calcareous rock (calcisiltite) brought to the surface from depths in excess of 100 m had a consistency of firm to stiff clay. This was unexpected given the overburden stress at that depth. The development of horizontal cracks (delamination) was visually observed within minutes of it being extracted from the core barrel. This behaviour was inferred to have occurred as a result of stress relief, and the breaking the cemented bonds and sample degradation was postulated to be due to the expansion of dissolved gasses within the pores of the samples and the low permeability of the material.

The effect of the cementation of the samples became apparent from one-dimensional (1D) consolidation tests. The void ratio (e) was observed to change only slightly with increasing vertical effective stress (σ'_v) until the strength of the bonds between the silt sized particles is exceeded. Once this occurred, the rate of consolidation dramatically increased. Such testing allowed the yield point of the cemented bonds to be assessed for samples with various in situ void ratios, which in turn allowed a bond strength envelope to be plotted in e versus σ'_v space. Figure 2 presents the results of an oedometer test undertaken on a sample recovered from a depth of 182 m.

The observed and measured behaviour of the samples indicated the importance of limiting the stresses applied to the material from the piled raft and indicated that more reliance should be placed on the in situ testing in preference to laboratory testing to define the engineering properties, particularly the stiffness of the ground beneath the proposed tower.

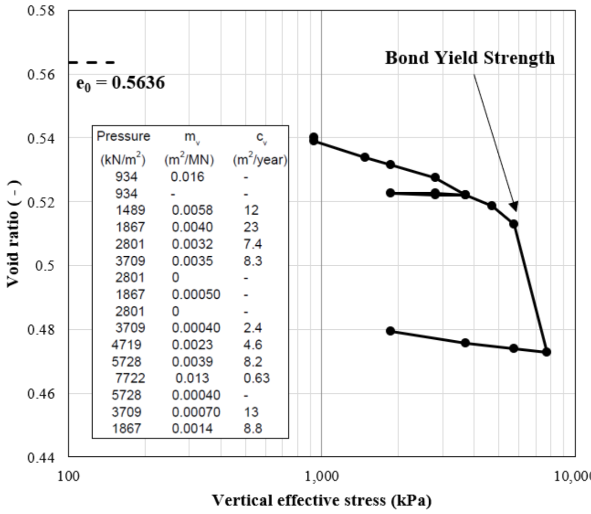

Figure 2 Results of oedometer test undertaken on sample recovered from 182.5 m with approximate Bond Yield Strength indicated

4.2.5 Stiffness measurements

Small strain stiffness was obtained from the results of the crosshole seismic testing. Cyclic triaxial testing and resonant column testing was undertaken in the laboratory. Typically, the small strain stiffness measured in the laboratory was about 5 times less than that measured in situ at the location from which the sample was taken, which is consistent with the stress relief and micro-cracking of the laboratory test samples.

Figure 3 presents the initial Young's modulus measured in the pressuremeter testing undertaken in 3 boreholes (BH203, BH204 and BH208). Also shown on the same plot are the results of Young's modulus measured on samples tested in the laboratory in UCS tests with end platen displacement measurement. The general shape of the profiles with depth correlates well between the different boreholes suggesting relatively uniform ground conditions underlying the site. This is consistent with borehole core observations.

The discrepancy between the stiffness measurements made in the field and laboratory for the calcareous materials, Unit C and D was attributed to the effects of stress relief. Greater reliance was therefore placed on the in situ pressuremeter testing for the development of a geotechnical model for analysis.

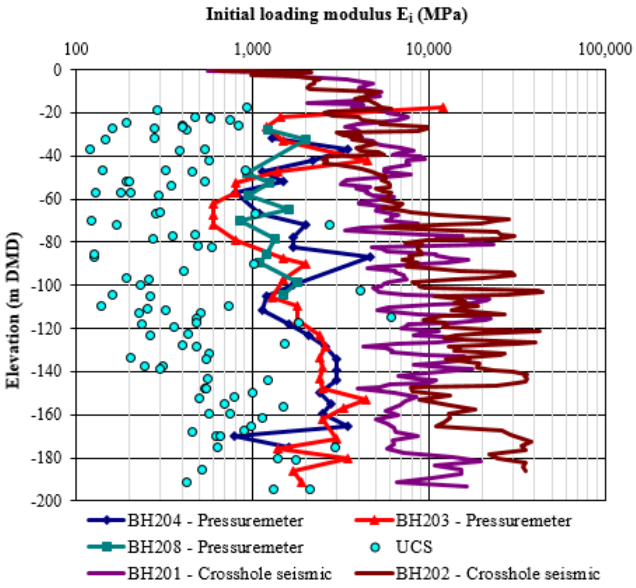

Figure 3 Initial Young's Modulus

4.2.6 Test barrettes and static test results

Due to the significant depth of the piles (perhaps up to 100 m), barrettes were proposed in lieu of piles. Three test barrettes with cross-sectional dimensions of 1.2 m x 2.8 m were installed to depths of 65 m (TB02 and TB03) and 95 m (TB01). The test barrettes were installed using an hydrofraise with polymer support. The hydrofraise cutting action results in a relatively smooth excavated surface and hence a concrete rock interface which is essentially devoid of roughness. High slump concrete was placed by tremie. Concrete design characteristic 28 day strength was 60 MPa. Strengths significantly in excess of 60 MPa were achieved during construction.

Load testing of the barrettes comprised two levels of Osterberg cells in each test barrette. Each level of cells was capable of providing a design bi-directional load of 54 MN. However, during testing loads were increased to the capacity of the equipment resulting in bi-directional loads of up to 83 MN. The Osterberg cells were positioned to measure performance of the lower 20 m or so of the barrettes.

The test barrettes were instrumented with displacement tell-tales and strain gauges. In addition, instrumentation was also located in the rock below the toe of the barrettes to directly measure the displacement of the rock at this location.

The barrette load tests were used to investigate load deformation behaviour of the shaft and base of the barrette under static, cyclic and long-term conditions and also as large-scale loading tests to confirm modulus estimates. The measured load versus displacement performance of the two shorter test barrettes (TB02 and TB03) for loading at the lower and upper levels of Osterberg cells are shown in Figures 4 and 5, respectively. Also shown are predictions of the performance. The predictions were obtained on the basis of the adopted design properties for the ground and on the as-constructed barrette geometry. The predictions of performance were completed prior to testing of the barrettes.

Base drilling was also undertaken within the test barrettes through ducts cast into the barrettes. The objective of this drilling was to assess the presence of debris on the base and quality of the contact between the concrete and underlying rock. The drilling indicated that the contact was not clean and that debris was present.

The test barrettes provided confirmation of the constitutive model developed on the basis of the earlier in situ and laboratory testing. Key elements of the model that influenced the foundation design included:

- If the ground stress exceeds the bond yield strength, a collapse type behaviour resulting in consolidation and creep could ensue.
- There is likely to be poor contact between the barrette and ground, possibly due to debris on the base of the barrettes.

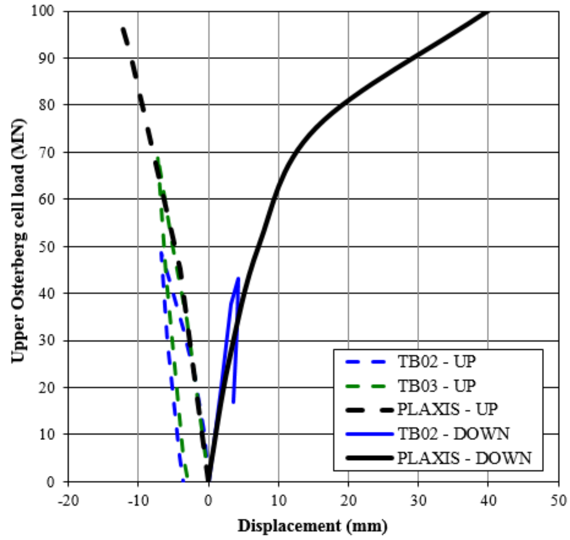

Figure 4 Measured versus predicted performance for loading at upper Osterberg cells

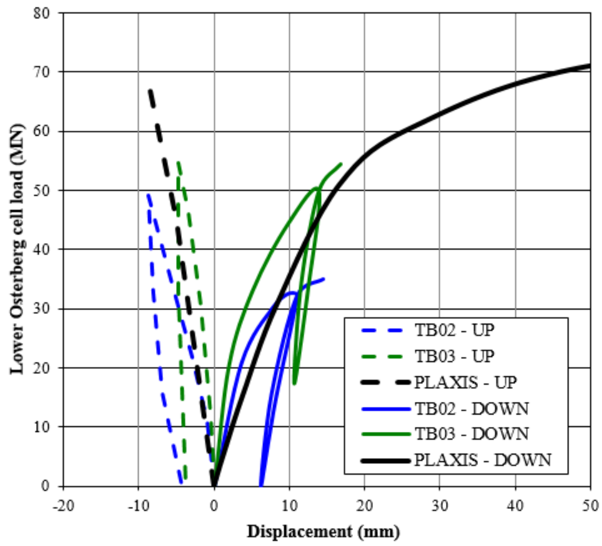

Figure 5 Measured versus predicted performance for loading at lower Osterberg cells

4.3 Soil-structure interaction analysis

4.3.1 Basis of foundation design

The proposed piled raft was to have a diameter of about 105 m and a thickness ranging between 4 m and 8 m. The raft slab was to be founded at about RL -17.5 m in the top of the Unit C material, and to be supported by barrettes. The preliminary schematic comprised 184 barrettes of 2.8 m by 1.2 m (plan dimension) and 224 barrettes of about 2.8 m by 1.5 m, a total of 408 barrettes. The final design, as modified by later analyses, adopted 392 barrettes. The number of barrettes was dictated by the ultimate structural load that could be carried by each barrette, and not by geotechnical factors.

The footing layout is shown in Figure 6. Analyses were carried out for various combinations of dead load, live load (LL), wind load (WL) and earthquake load. Loads and load combinations for these analyses were provided by the structural engineers.

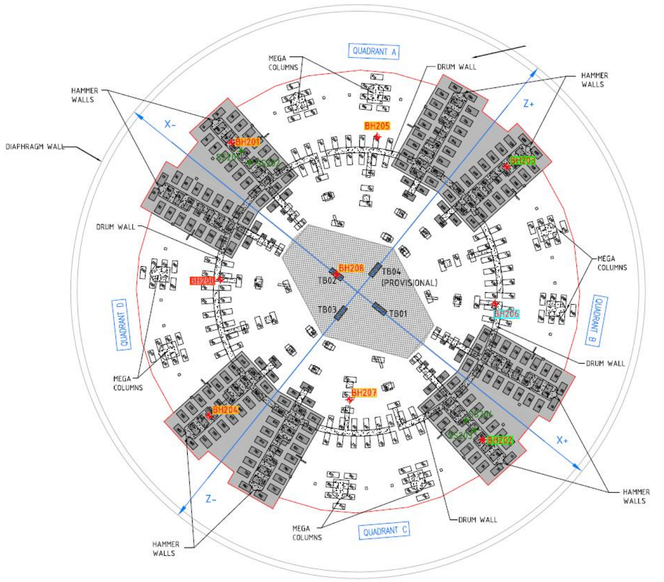

Figure 6 Plan view of footing layout

4.3.2 2D Analysis of foundation

As the first stage of design development, the design performance of the proposed footing system was analysed to:

- Calculate settlements of the tower under design dead, live and wind loads.
- Provide equivalent spring stiffness values for the raft and barrettes that could be used in the structural analysis of the footing system.

The results from both PLAXIS 2D and REPUTE analyses were combined to provide representative spring stiffness values for the barrettes and the raft for use in structural models of the Tower footing system. Based on these results, the structural engineers for the project were able to refine the barrette / raft layout and dimensions and the column loads. The footing system was then re-analysed using the above process to arrive at new spring stiffness values for the new loads. The process was iterated until convergence in loads and deflections was obtained. Typical calculated settlement profiles across the raft for static (i.e. dead and live loads) and wind loading cases (i.e. dead, live and wind loads) are shown in Figure 7.

PLAXIS 2D was also used to assess the effect of base debris on the settlement response of the pile to loading and the stresses at the toe of the barrettes. It was determined that the base debris does not have a significant impact on the calculated settlement, but results in locally higher stresses in the ground close to the toe of the barrettes, due to load shedding from the base to the shaft of the barrette. The calculated stresses at the toe of the barrettes were such that the bond yield strength in the material is likely to be exceeded where base debris is present, with associated time-dependent compression of these materials. This would result in a risk of greater settlement or tower tilt. The impact of base debris, should it occur, was reduced by using the much stronger gypsum layers to spread the load from the barrettes onto the underlying Unit D material. If the barrettes were founded through the uppermost gypsum layers at about RL -75 m, the shaft resistance developed in the gypsum would offset the loss of base resistance due to base debris and hence reduce the local areas of high vertical stress in the Unit D material.

Spring stiffness value for barrettes were estimated and typically varied under the combination of dead and live loads from approximately 0.3 MN/mm to 0.8 MN/mm with an average of

0.5 MN/mm. Spring stiffness values for the raft were calculated to vary between 1 kPa/mm (centre of raft) and 10 kPa/mm (edge of raft) depending on the location beneath the raft.

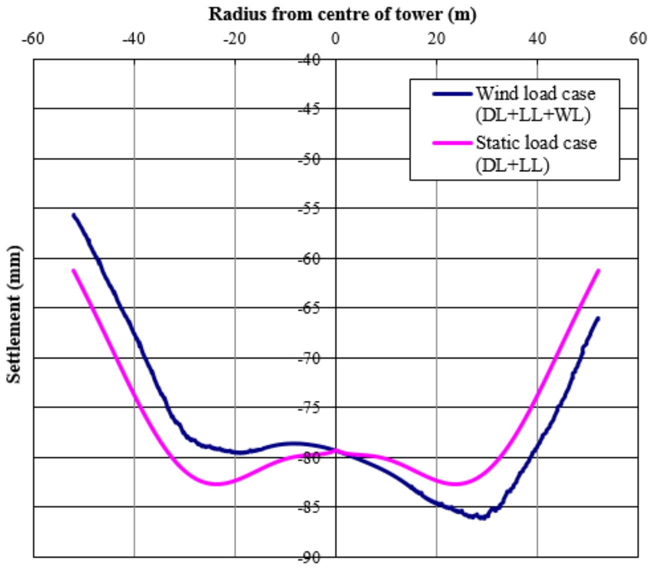

Figure 7 Settlement profile for unfactored ground stiffness

4.3.3 3D Analysis of foundation

The results of the 2D axisymmetric analyses provided the basis for a viable footing system for the tower. Although versatile and relatively quick to undertake, these analyses only provide an indication of the 3D response of the footing system. The 2D analyses enabled evaluation of the benefit or otherwise of changing barrette layouts and lengths, and development of the final footing system. 3D analyses using PLAXIS 3D were undertaken to allow a better assessment of the performance of the footing system under non-symmetrical load cases such as wind and earthquake loading. The objectives of the 3D analyses were:

- To calculate the settlement profile of the tower raft under gravity and wind working load cases.
- To confirm geotechnical stability of the footing system under ultimate load conditions.
- To calculate the stiffness of the barrettes and the raft for gravity and wind working load cases for use in structural analysis of the foundation system by WSP.
- To calculate barrette actions (shear force and bending moment) within barrettes for ultimate load cases (including base shear).
- To estimate the impact of debris at the base of the barrettes on the settlement performance of the footing system.
- To estimate the vertical stress increase below the toe of the barrettes under working load and ultimate load conditions for estimation of potential long-term settlement (creep).

Three working load combinations and two ultimate load combinations were analysed. Analyses were undertaken for each working load combination assuming cases of full base resistance and no base resistance. The analyses assuming full base resistance were considered to provide a reasonable estimate of short-term performance while the analyses with no base resistance provided a conservative estimate of long-term performance (for the properties and conditions assumed). For each analysis, the following values were evaluated:

- Vertical settlement at the head of the barrettes.
- Vertical load at the head of the barrettes.
- Axial stiffness of each barrette.
- Geotechnical factor of safety for each barrette.

A screenshot of the PLAXIS 3D model is shown in Figure 8. As described earlier, the subsurface stratigraphy at the site comprises relatively uniform beds of sedimentary material. The bedding within the different units and the contacts between them are generally sub-horizontal, or with a slight dip. The dip of the beds was modelled in PLAXIS 3D as seen in Figure 8.

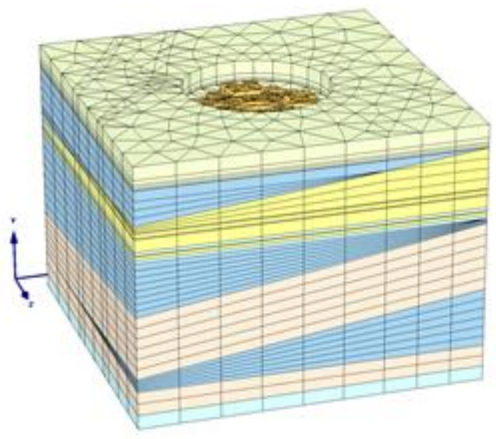

Figure 8 Screenshot of PLAXIS 3D model for Nakheel Tower

The outputs of the analyses were presented in spreadsheets which gave the load and settlement estimates for each of the 392 barrettes. These results were used by the structural engineers as input to their analyses, which resulted initially in revised barrette loads. Further foundation analyses were performed until the calculated barrette head loads and settlements converged with the structural inputs.

The calculated maximum and minimum settlements under working load conditions are summarised in Table 1.

Table 1 Ranges of calculated settlements of major structural elements

Load Case	Full base resistance (short-term)	No base resistance (long-term)
DL + LL	62 mm – 72 mm	74 mm – 87 mm
DL + 0.8WL (windward minimum)	34 mm – 46 mm	42 mm – 57 mm
DL + 0.8WL (leeward maximum)	70 mm – 80 mm	87 mm – 99 mm
DL + 0.75LL + 0.6 WL (windward minimum)	46 mm – 60mm	58 mm – 70 mm
DL + 0.75LL + 0.6 WL (leeward maximum)	72 mm – 82 mm	86 mm – 100 mm

For the dead plus live load case, the calculated settlements assuming full base resistance were about 10 mm to 15 mm less than those obtained from the analyses assuming no base resistance. It was considered the analyses assuming full base resistance provided a reasonable estimate of settlement performance of the tower footing system in the short-term.

The analyses indicated that under full design gravity loading of the tower, the bond yield stress immediately below the barrettes was likely to be exceeded and some creep would occur. This would lead to load transfer from the base of the barrettes to the shaft. Alternatively, on the assumption that some debris was present at the base of the barrettes, in the short-term the debris would be incompressible and hence the full base resistance may be relevant. However, over time the fluid within the debris would drain and hence load would be transferred from the base to the shaft. It is probable that both mechanisms may occur concurrently.

The consequence is that over time, at least some load would be transferred from the base of the barrettes to the shaft of the barrettes. The extreme end condition of this is that the base of the barrettes may carry little or no load. This condition was modelled by the analyses assuming no base resistance. It was therefore considered that a reasonable upper estimate of the long-term settlement of the tower footing system under the design case parameters was provided by analyses which assumed no base resistance.

Where full base resistance was assumed, barrette axial loads under the dead load plus live load combination varied from 16 MN to 57 MN, and from 12 MN to 79 MN for the wind loading cases. These loads translated to a geotechnical factor of safety typically greater than 2.5. Barrette stiffness values ranged between about 0.2 MN/mm and 1 MN/mm.

The maximum axial load in the 1.2 m x 2.8 m and 1.5 m x 2.8 m barrettes under the working load cases analysed were 64 MN and 79 MN, respectively. These are less than the barrette structural working load capacities of 64.5 MN and 80.6 MN provided by WSP.

Where no base resistance was assumed, barrette axial loads under the dead load plus live load combination ranged between 13 MN and 47 MN and for the wind load combinations between 7 MN and 56 MN. For the most onerous wind load case analysed, the geotechnical factor of safety was typically greater than 2.5. Barrette stiffness values ranged from 0.16 MN/mm to 0.6 MN/mm. Where the raft was 4 m thick or greater, the calculated raft stiffness was about 12 MN/mm.

5. CASE STUDY – AUSTRALIA 108

5.1 Introduction

Australia's tallest building (excluding spires) at approximately 320 m in height is currently under construction in the South Bank area of Melbourne, at a site which is located approximately 300 m south-east of the Yarra River. Construction has commenced and is expected to be completed in early 2020 (Figure 9). Due to the challenging ground conditions at the site (including up to 20 m of uncontrolled fill and soft clay), the tower is supported on large diameter piles socketed in low to medium strength siltstone at about 40 m depth. A summary of the challenges of basement construction in Melbourne is presented in Lochaden & Haberfield (2018). A summary of the approach undertaken to design the footing system for the tower is described herein.

Figure 9 Australia 108 under construction in April 2019 (left), and artist rendering of Australia 108 post-construction (right)

Due to the poor near surface ground conditions, a primary interest for this tower was the behaviour of the combined tower and footing system under wind loading. The significant height and slenderness of the structure results in high design wind loads. The footing system is required to provide sufficient lateral stiffness under such loading, especially with respect to the dynamic response of the tower. However, the presence of the near surface soft clay deposits, namely the Coode Island Silt (CIS), a Quaternary age deposit of the Yarra River Delta, led to significant challenges in the design of the footing system.

The dynamic response of the building under wind loading was found by the project structural engineers to be relatively sensitive to the design of the foundation system, and so close collaboration was required between the geotechnical and structural engineers throughout the design process. The structural engineers required accurate input regarding the foundation stiffness for use in the structural analysis of the tower. The model used for structural analysis of the tower considered the footings supporting the columns as springs with axial, lateral and rotational stiffness. The design stiffness values for the footings was computed using PLAXIS 3D.

5.2 Geotechnical investigation

The geotechnical investigation at the site was undertaken in two stages, in which the results obtained from the first stage of investigation and the authors' experience of tower design and construction in Melbourne, allowed refinement of the requirements for the second stage of investigation. Based on a total of seven boreholes which extended to over 50 m in depth, the subsurface stratigraphy at the site comprises:

- Fill: about 2 m thick, variable (a mixture of soft to firm silty clay and loose to medium dense silty sand and sand) fill materials; overlying
- CIS: about 18 m thick, soft silty or sandy clay, becoming soft to firm and firm with depth; overlying
- Fishermens Bend Silt (FBS): about 4 m thick, firm to stiff silty clay; overlying
- Moray Street Gravel (MSG): about 8 m thick, medium dense and dense silty sand and stiff sandy silt; overlying
- Werribbee Formation (WF): about 5 m thick, stiff to very stiff sandy clay; overlying
- MF: highly weathered, low strength siltstone becoming less weathered and of medium to high strength with increasing depth.

The ground water level at the site is at about RL 0 m (or about 2 m below the existing ground surface).

The ground investigation included high pressure pressuremeter testing and unconfined compression testing on the siltstone. Previous investigations in the nearby area provided relevant data on the properties of the CIS and the underlying alluvial soils.

For the assessment of the performance of the combined tower and footing system under the dynamic load case of wind loading (i.e. loads due to wind gusts which are applied over a short time duration), short-term properties were assessed for each of the subsurface materials. Typical properties adopted for the wind load analyses are provided in Table 2.

Table 2 Soil / rock properties for wind load case

Soil	Cohesion (kPa)	Friction angle (deg)	Young's Modulus (MPa)
Fill	0	28	10
CIS	15+1.5z	0	4.5+0.45z
FBS	5	28	30
MSG	8	30	80
WF	10	30	30
MF	500	43	1000

5.3 Stability system to resist wind loading

The primary foundation elements which resist wind loading are referred to herein as the stability system, and comprise three levels which are described below with reference to the pile layout set out in Figure 10.

The lowest level of the core is the lift over-run pit which is supported on 16 no. 1800 mm diameter bored piles founding in the siltstone with floating 600 mm diameter secant piles between the adjacent 1800 mm diameter piles. The 1800 mm and 600 mm diameter piles act to form a secant pile wall and the retention system for the lift over-run pit. The 1.5 m thick base slab of the pit is structurally connected to the secant pile wall, whilst a 1.9 m wide capping beam structurally connects the secant pile wall to the core wall.

Piles with diameters of 600 mm, 900 mm and 2100 mm diameter (as well as the pile cap) founding at depth in the siltstone provide support to the intermediate level of the core. These piles and pile caps are connected to the core capping beam by a 300 mm thick slab and walls.

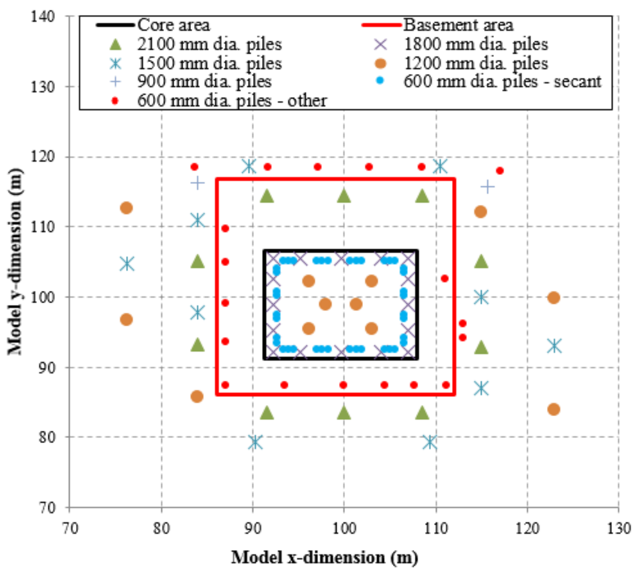

Figure 10 Pile layout of the foundation system

Bored piles ranging in diameter from 600 mm to 2100 mm and which found in the siltstone provide support to the upper most level of the stability system which is at ground surface level. These piles are connected to the intermediate and lowest levels of the stability system by 250 mm concrete slabs and walls.

The footing arrangements (i.e. locations and geometry of piles and pile caps) were modified throughout the soil-structure interaction analysis process to obtain an optimal solution with respect to calculated structural actions and displacements of the system.

5.4 Analysis model

PLAXIS 3D was adopted as the primary tool for the geotechnical analysis of the footing stability system. A screenshot of a typical PLAXIS 3D model adopted is presented in Figure 11. The overall model size is 200 m x 200 m x 85 m (length x width x depth) and has greater than 60,000 soil elements and 90,000 nodes. A finer mesh was used around the core area and gradually graded to larger elements (about 5 m dimension) outside the core. This mesh was deemed to be optimal as increases in the mesh refinement were found to result in negligible changes in calculated deformations and structural actions, but to result in substantial increases in computational time. Footings within a radius of about 30 m from the centre of the building but which were not part of the stability system described above were also included in the analysis, such that the structural actions in these elements resulting from the displacement of the stability system under wind loading could be assessed.

The PLAXIS 3D structural elements which make up the foundation system are presented in Figure 12, noting that the ground floor slab is not shown to allow the other structural elements to be clearly visible. The piles were modelled as embedded beams, whilst pile caps and the raft footing of the lift over-run pit were modelled as volumes to better capture the lateral resistance provided by these elements. The core walls and ground floor slab were modelled as plate elements.

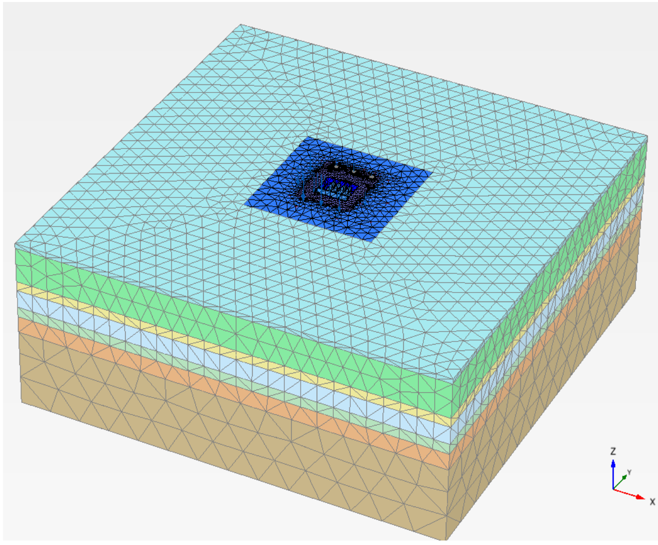

Figure 11 Screenshot of PLAXIS 3D model for Australia 108

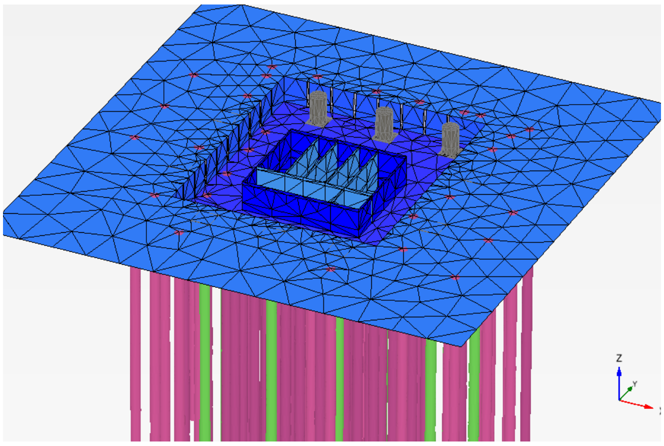

Figure 12 PLAXIS 3D model of stability system of Australia 108

5.5 Interaction between geotechnical and structural engineers

The primary aim of the PLAXIS 3D analyses was to optimise the footing system with respect to the length, diameter, and number of piles, whilst ensuring that the proposed system was practical to construct and performed satisfactorily with respect to calculated structural actions and displacements of the system. Over ten load SLS and ULS load cases which considered wind and earthquake conditions were analysed.

Following an initial meeting between structural and geotechnical engineers and provision of some preliminary geotechnical advice with respect to the likely foundation system, the structural engineers provided a preliminary footing layout for the stability system. The geotechnical engineers provided some preliminary spring stiffness values for each foundation element and the structural engineers conducted preliminary analysis for the perceived worst-case loading condition. From the results of this analysis, a first estimate of structural reactions at each level of the stability system were provided to the geotechnical engineers. These reactions were then applied in the PLAXIS 3D model. The output from the PLAXIS 3D analysis was interpreted to provide profiles of displacements of the stability system, structural actions within the piles and interconnecting structural units (walls, floor and raft), and updated spring stiffnesses for each foundation element. Based on these results, pile, wall, raft and slab sizes were revised and the structural analysis undertaken on the revised stability system. New reactions were then provided to the geotechnical engineers and the above process repeated. This iterative process was undertaken a number of times until a reasonable match between the vertical and horizontal displacements computed from the structural and geotechnical analyses was obtained.

Once a satisfactory footing solution was obtained for the worst wind load case, other wind and earthquake cases were undertaken. The earthquake loading cases were found to be less critical than the wind load cases.

The iterative process was complicated significantly by the torsional forces acting on the stability system and by the three different levels of the stability system.

An example of a typical match between the calculated horizontal displacements from both geotechnical and structural analysis are presented in Figure 13.

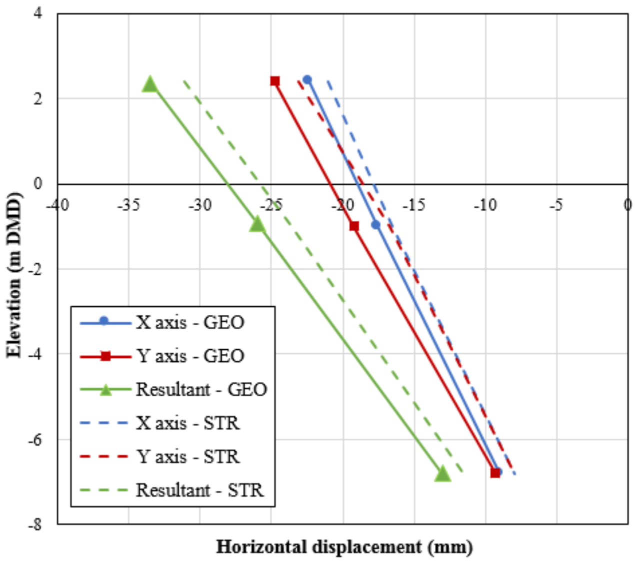

Figure 13 Comparison of calculated lateral displacements from geotechnical (GEO) and structural (STR) analyses

6. SUMMARY

This paper discusses the authors' recommendations for geotechnical investigation for tall towers. The scope of the investigation must target the specific ground conditions at the site, which must initially be assessed from a desktop study of available information. This desktop study allows the likely subsurface conditions to be identified and preliminary foundation concepts to be developed. The ground investigation is then used to confirm the geological model and refine the geotechnical model and properties of the site. The ground investigation must do more than simply quantify the subsurface stratigraphy and the engineering properties of the primary founding materials; rather it must specifically target the load deformation performance of such materials. Therefore, every site is different, and the scope of the ground investigation must be tailored to suit.

Obtaining suitable samples from significant depth for laboratory testing which are sufficiently undisturbed to provide reliable estimates of load deformation performance is difficult. Greater reliance should be placed on quality in situ testing, which should be focused on the assessment of stiffness and load deformation behaviour. Such testing includes high quality high pressure pressuremeter testing, crosshole seismic testing and pile load testing. However, this does not negate the requirement for appropriate high quality laboratory testing.

Once a suitable geotechnical model has been developed, the detailed analysis of the foundation system can be undertaken using soil-structure interaction software packages. The geotechnical engineer must carefully consider a number of issues, including the limitations of the design methodology (including ground and constitutive models) and the means by which the design methodology may be validated (for example, against measured data). The material factoring approach of limit state design is considered to be inappropriate for such analyses, and prudently conservative best-estimate values should be adopted.

The close interaction between structural and geotechnical engineers and careful soil-structure interaction analysis is critical to the successful design of tall tower developments. This approach was instrumental in developing and constructing a foundation system for Australia's tallest building, for which the stability of the building under wind loading in particular was a challenge due to the significant thickness of soft clay at the site. This approach was also described for Nakheel Tower, which if constructed would have been greater than 1 km in height.

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