The Application of 3D Finite Element Method in the Design of Large Piled Foundation System - Case Study: Melbourne Cement Facility

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ABSTRACT

This paper provides an overview of the foundation design and analysis process carried out for the Melbourne Cement Facility silo located in Port Melbourne, Victoria, Australia. The proposed silo is a cylindrical multi-compartment cement storage facility supported on a 2.6m thick concrete ring beam with an external diameter of 38.5m. The ring beam is supported by a piled foundation system comprising 155 CFA piles in an annular pile layout. The site is underlain by Quaternary Sediments of Yarra Delta which are further underlain by Werribee Formation. This paper describes a detailed soil-structure interaction analysis performed using the finite element program PLAXIS 3D, which was used to assess the foundation performance with particular attention to global and differential settlement of the pile group. The study evaluated the complex load sharing between the piles and the ring beam, and the differences in load mobilisation between piles within the group. The results of this study highlight the capability of 3D FEM analysis for obtaining an optimised foundation design solution and understanding and addressing various technical challenges associated with silo foundation systems of this type.

Keywords: piled raft, piled foundations, foundation design, finite elements, storage facilities, settlement analysis

1 INTRODUCTION

Piled raft foundations can be a practical and economical solution for very tall budlings and heavy industrial facilities where substantial vertical loads will be transferred to the substructure. The proposed cylindrical multi-compartment cement storage facility weighs up to 922 MN when all compartments are fully loaded. Considering the size and loads of such large storage facilities, the settlement of the foundation system supporting the structure is often a governing factor in the design. It has been shown that piles can be successfully used as settlement reducers (Burland et al. 1977; Mandolini et al. 2005) with a contribution from the overlying raft. Moreover, in many cases the primary objective of including piles in the design of pile raft foundations is to control settlements as raft alone can provide sufficient bearing resistance (Davis and Poulos 1972; Randolph 1994). However, as Viggiani et al. (2012) identified in the case of a so-called *small pile raft*, the un-piled raft could not solely carry the vertical loads and, in contrast to the case of a *large piled raft*, the key requirement for including piles is to satisfy an adequate factor of safety against bearing failure.

A considerable body of literature has been focused on developing methods for piled raft analysis. Early analytical works for the piled raft analysis were carried out by Davis and Poulos (1972) and Randolph (1983, 1994). Davis and Poulos (1972) presented a simplified analytical approach to evaluate the role of piles as settlement reducers in combined piled raft systems. The first series of simplified numerical analyses emerged as in two distinct approaches of *strip on springs* (Poulos 1991) and *plate on springs* (Poulos 1994; Viggiani 1998; Russo and Viggiani 1998). Ta and Small (1996) developed the Finite Layer Method for the analysis of pile raft foundations resting on layered soil stratum. More recent attention has focused on three-dimensional analysis of piled rafts (Smith and Wang 1998; Katzenbach et al. 1998; Katzenbach et al. 2005; Lee et al. 2010). Recent advances in Finite Element Modelling (FEM) have made full three-dimensional piled raft analysis a robust tool to capture the realistic soil-structure interaction and load sharing behaviour among the piles and the raft. In this study, the FEM program PLAXIS 3D was employed to evaluate the performance of the foundation system in regard to both stability and serviceability requirements. A detailed soil-structure interaction analysis was carried out to evaluate and understand the contribution of the piles and the ring beam in the design of the foundation system.

2 SITE LOCATION, SILO STRUCTURE AND LOADING CONDITIONS

The cement silo facility proposed for construction by Melbourne Cement Facility (MCF) is a cylindrical structure with a number of segmented storage compartments and a total height of about 71m. The structure is located in Port Melbourne, Victoria, Australia. The site location and the proposed location of the new silo is shown in Figure 1. The structure has an internal diameter of about 31.4m and comprises circular reinforced concrete walls with thicknesses of around 0.5m to 1.2m.

Figure 1. Site location and the proposed location of the new silo - source: Google Earth

The silo structure is supported on a 2.6m thick annular ring beam which is shown in Figure 2. Loads coming from the superstructure will be transferred from the cylindrical silo walls to the ring beam and the piles constructed underneath. The foundation system is designed to perform as a piled raft as the ring beam has a large dimension (and contact area) and can contribute to the load-bearing of the system. A quantitative assessment of the contribution of the piles and ring beam to the foundation performance will be discussed in subsequent sections. The overall dead load and live load of the silo structure is in the order of 900 MN in total. The load critical load cases that were adopted in foundation modelling and design are summarised in Table 1.

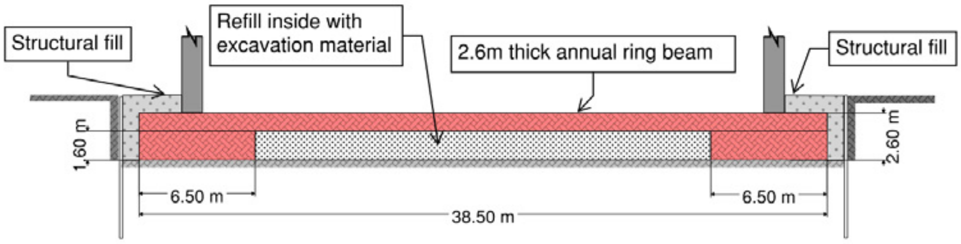

Figure 2. Silo ring beam geometry

Table 1: Summary of design load cases

Load Case	Description	Load Composition
SLS-1	Serviceability limit state for global settlement check – full silo loading	DL+LL
SLS-2	Serviceability limit state for differential settlement check – live loading on one half of silo only	DL across full silo LL on one half of silo
ULS-1	Ultimate limit state for maximum vertical loading based on AS 1170 load combination	1.2 DL + 1.5 LL
ULS-2	Ultimate limit state for earthquake loading based on dead & live vertical loads, shear force and overturning moment	DL + LL + EQ

Serviceability limit state load cases SLS-1 and SLS-2 were used for settlement assessment. It was assumed that all silo compartments are fully loaded for SLS-1. It is understood that some compartments of the silo can be either fully loaded or completely empty at times. Therefore, SLS-2 was adopted to account for the worst-case scenario in terms of differential settlements when half of the silo is empty (DL only) and the other half is fully loaded (DL+LL). Ultimate state limit load cases ULS-1 and ULS-2 were used to check the generated actions in piles against the structural capacity. ULS-1 was also used to check the overall geotechnical stability of the foundation system which will be discussed in more detail in the subsequent sections.

3 GROUND CONDITIONS

According to the Geological Survey of Victoria 1: 31, 680, the construction site is located in an area of recent Quaternary sedimentation which forms part of the well-known Yarra delta. Yarra delta sediments at the site are present beneath a fill layer, and comprise Port Melbourne Sand (PMS), Coode Island Silt (CIS), Fishermen's Bend Silt (FBS) and Moray Street Gravel (MSG). The Yarra Delta sediments are further underlain by Werribee Formation and Silurian age Siltstone of the Melbourne Formation. The groundwater level has been reported consistently at a depth range of 2.2m to 2.7m below ground surface, consistent with the adjacent river water levels. Ground models were developed based on the available site investigation data, which includes information from both borehole and CPTs. The ground conditions were found to be reasonably consistent with our general expectation of the Yarra Delta profile as described in many existing references, for example, Ervin (1992). A summary of the adopted ground model is presented in Table 2.

Table 2: Summary of geotechnical modelling parameters

Unit	**Depth BGL (m)**		**Unit Weight (kN/m^3)**	**Effective Cohesion c' (kPa)**	**Friction Angle ϕ' (deg)**	**Young Modulus E' (MPa)**
	From	**To**				
Fill	0	3.6	18	2	30	20
Port Melbourne Sand	3.6	6.5	19	2	30	20
Coode Island Silt	6.5	8	17	2	27	5
Fishermen's Bend Silt	8	29	19	7	27	50
Moray Street Gravel (interbedded clays and sands)	29	49.5	19	2 - 7	28 - 35	55 - 180
Werribee Formation (sand)	49.5	63.5	19	2 - 10	28 - 37	60 - 150
Siltstone, EW to HW	63.5	70	20	10 - 25	30 -32	60 - 200

4 FOUNDATION DESIGN

The large depth to siltstone rock meant that the piled foundations needed to be founded in the overlying soil profile at depths that could be readily achieved by an available piling plant. The size and significance of the silo structure, and the significant loading it imposes on the founding soils, meant that both geotechnical strength and serviceability requirements had to be carefully evaluated.

According to the adopted serviceability design criterion, calculated foundation settlements under the serviceability load combinations (SLS-1 and SLS-2) should not exceed the allowable settlements. Allowable settlements for silo were specified as below:

- Total settlement is not to exceed 300mm.
- Differential settlement under Load Case SLS-2 is not to exceed 60mm from one side of the silo to the other.

Design for geotechnical strength was based on the limit state design principles documented in the Australian Piling Code (AS 2159-2009). This requires piles to be designed in such a way that the design geotechnical strength (which is the ultimate geotechnical strength multiplied by an appropriate geotechnical strength reduction factor) will be not less than the design action effect. However, it has been well documented that applying such geotechnical criteria to each individual pile within a pile group can result in considerable conservatism in the design (e.g., Poulos, 2017). This is acknowledged in AS 2159-2009 in clause 3.2.2, which allows individual piles within the group to be overloaded for the ULS load case provided that the ultimate geotechnical design strength of the pile group satisfies the requirements of the code. To this end, the ULS-1 load divided by an appropriate Geotechnical Strength Reduction Factor (φ_g), was modelled with the acceptance criteria being that the pile group must maintain overall stability. This is indirectly a check on the overall factor of safety for the foundation system design and stability under this load case demonstrates that the requirements of AS2159-2009 are met with respect to strength.

The foundation system has the ring beam supported on 155 No. 900mm diameter reinforced concrete CFA piles with pile toes founded in the Moray Street Gravel (MSG) unit. The layout for the CFA piles includes five rows of piles from inside to outside of the ring beam. The pile spacing varies between rows, but averages about 2.1m.

Available site investigation data suggested that the MSG is comprised of interbedded sand and clay layers. To account for the lithological variation of interbedded granular and cohesive sublayers within the MSG unit and the possibility of encountering clay lenses at the toe of some of the piles, a series of sensitivity analyses were performed to evaluate the foundation performance with an appropriate range of ultimate base resistance values.

Appropriate pile design parameters for shaft friction and base resistance were determined in consultation with Wagstaff Piling, based on previous experience and available load test results from previous tests in similar ground profiles, ensuring that the selected values were reasonably expected to be achievable at the site for CFA piles installed with the available equipment. Table 3 presents the unit stresses that were adopted in our modelling.

Table 3: Summary of pile resistance parameters

Unit	**Depth (m)**		**Ultimate Shaft Friction (kPa)**	**Ultimate Base Stress (kPa)**
	From	**To**		
Fill	0	3.5	0	-
PMS	3.5	6.5	20	-
CIS	6.5	8	20	-
FBS	8	17	50 to 80	-
MSG	34	40	100 to 150	3000-5000*

** Sensitivity checks were carried out for the range of base stresses presented which cover the expected range of outcomes in practice.*

Using the estimated unit stresses, the ultimate geotechnical strengths of piles considered in the modelling process ranged between 9.2MN and 11.3MN, depending on the various founding assumptions.

5 NUMERICAL ANALYSIS

Recent advances in computational speed have facilitated the utilisation of relatively complex and detailed 3D analyses of piled raft foundations. Commercially available software packages, such as PLAXIS3D, can model complex ground conditions, pile arrangements and soil-structure interaction. In this study, modelling of the silo foundation system was carried out using the PLAXIS 3D software.

Simulation of piles in a sufficiently accurate and realistic manner is a key aspect of a piled foundation analysis. There are two distinct methods in PLAXIS for modelling piles in a 3D analysis. One involves modelling piles as volume elements with a user-defined interface. Although considered to be the more robust approach, the application of volume elements can be extremely demanding in terms of processing time for a large foundations system with a considerable number of piles.

The alternative option, which is adopted in this study, is the application of embedded beam elements in which the piles interact with the surrounding soil by means of special embedded interface elements. Lee et al. (2010) showed that the application of embedded beam elements can be considered a rigorous numerical approach as it results in very similar predictions compared to volume elements for pile rafts. Although pile shaft friction can be calculated automatically by the software (by relating the shaft profile to the strength properties of the soil), in this exercise the shaft resistance is manually defined to ensure the modelled ultimate geotechnical resistance of piles is consistent with the values given in Table 3.

A linear elastic perfectly plastic model with Mohr-Coulomb failure criteria was adopted for all soil layers. Effective stress analyses were carried out using drained material properties to evaluate the long-term settlement behaviour of the foundation system. Long term consolidation over the design life of the structure was also considered, by estimating the secondary compression of soils over the 50-year design life of the structure. The adopted geotechnical modelling parameters are summarised in Table 2. To ensure loading was accurately distributed to the foundation system, loads were converted to equivalent pressures on top of the ring beam. Figure 3 shows the model geometry and pile arrangements.

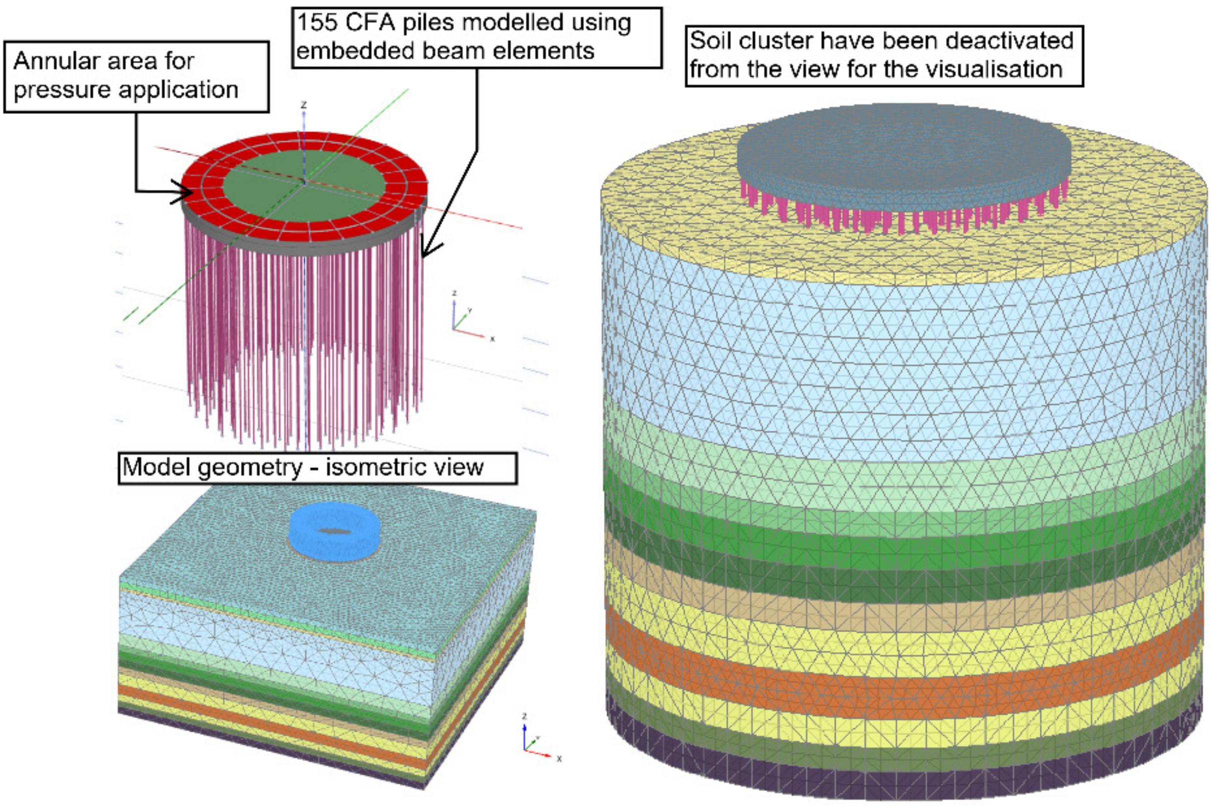

Figure 3. PLAXIS 3D model geometry for the silo foundation system

6 RESULTS AND DISCUSSION

Table 4 summarises the estimated total and differential settlements for the foundation system under SLS load cases, for each of the adopted founding levels and end bearing assumptions. It can be inferred from the data in Table 4 that the settlement performance of the foundation system is not greatly sensitive to the range of assumptions that were used. We consider this is likely to be because a significant proportion of the SLS loads are being resisted by shaft friction over the length of the piles, and shed to the surrounding ground. As a consequence, the load is distributed to a large soil mass and small differences in founding level and end bearing do not have a significant impact on settlements. Figure 4 presents the predicted absolute and differential settlement contours for load case SLS-1 and SLS-2 for founding condition case 4 specified in Table 4.

Table 4: Summary of calculated settlement from PLAXIS analysis

Founding Condition Case	Maximum calculated vertical settlement (mm) SLS-1	Maximum calculated vertical differential settlement (mm) SLS-2
1.Toe RL -38.5m, f_{bu}= 5MPa	93	50 (33 to 83)
2.Toe RL -36.5m, f_{bu}= 5MPa	100	56 (35 to 91)
3.Toe RL -38.5m, f_{bu}= 3MPa	94	51 (33 to 84)
4.Toe RL -36.5m, f_{bu}= 3MPa	101	58 (34 to 93)

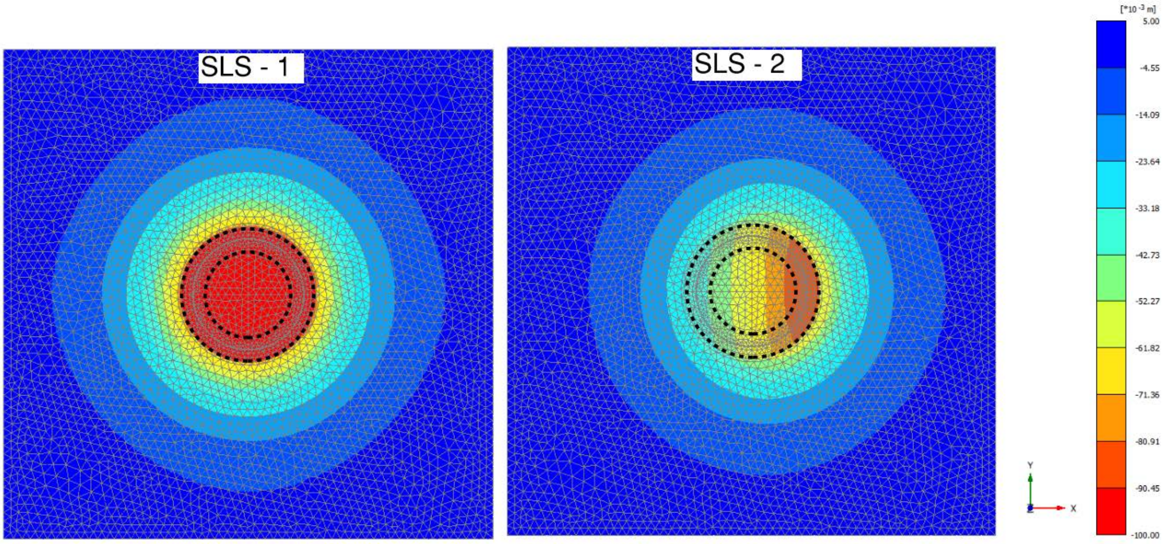

Figure 4. Absolute and differential settlement contours for load cases SLS-1 and SLS-2

The load sharing behaviour between the piles and the ring beam is summarised in Table 5, as the ratio of the sum of the pile head loads over the total load being applied on the foundation system. This is sometimes referred to as the combined piled raft foundation coefficient (Katzenbach et al. 2005). Table 5 indicates that the superstructure loads are mostly carried by the piles, with a bearing contribution of about 80%, and the remaining 20% of the loads are transferred to the subsoil directly through the ring beam contact area.

Table 5: Pile and ring beam load sharing

Case	Q_p/Q_t
SLS - 1	0.80
ULS - 1	0.78

Figure 5 illustrates the calculated pile head axial loads under ULS-1 for founding condition case 2. This Figure shows that the loads are unevenly distributed among piles, ranging from around 4.5 MN to 10 MN. It is noted that some of the variations in individual pile loads is thought to be attributable to the FEM modelling process; specifically localised meshing effects. For this reason, the variability in actual pile loads is expected to be less in practice than the model outputs would suggest.

Nonetheless, a large variation in individual pile loads has also been demonstrated in other studies (e.g., Poulos, 2017). A highly non-uniform distribution of loads among piles highlights the importance of considering group stability when assessing the ultimate limit state geotechnical capacity of the foundation system to avoid unnecessary over-conservatism in the design by designing all piles to the highest individual pile load.

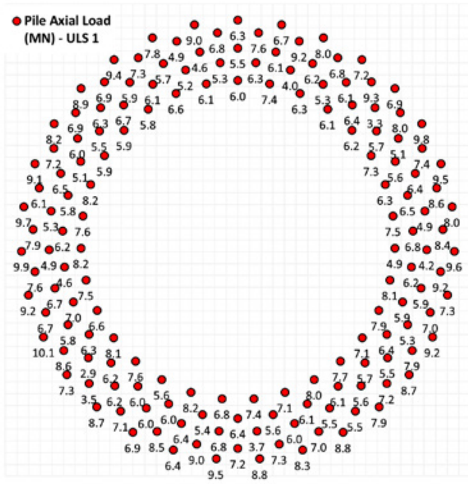

Figure 5. Calculated pile head loads, ULS-1

As mentioned in the preceding sections, an additional load case, based on the ULS-1 loading divided by the adopted geotechnical reduction factor (φ_g), was analysed for all founding condition cases and a satisfactory outcome was achieved. This demonstrated compliance with AS2159-2009 with respect to the ultimate geotechnical capacity of the pile group, and an adequate overall factor of safety for the foundation system design.

To account for the localised variability in individual pile loads (inferred to be due to meshing effects) in the design, our approach was to average the individual pile loads across a selection of adjacent piles for each row of piles. Table 6 presents the average maximum axial pile head loads across inside, middle and outer rows of piles under SLS-1 and ULS-1 load cases. It is noteworthy that the outside row piles attract significantly higher loads compared to the middle row and inner row piles.

Table 6: Summary of calculated pile head loads

Case	Maximum axial pile head load (kN)
SLS - 1	Outside Row = 9413 Middle Row = 3159 Inside Row = 5765
ULS - 1	Outside Row = 10050 Middle Row = 5747 Inside Row = 6591

To better understand the load sharing mechanism between piles in different rows, it is necessary to consider the differential movements between these piles and the surrounding soil. Figure 6 shows a cross-section of the vertical settlement contour plot through the midsection of the silo from the PLAXIS analysis.

This figure needs to be interpreted in conjunction with the tabulated values in Table 6. As shown in Figure 6, the soil mass enclosed inside the ring of piles settles much more compared to the soil mass outside the footprint of the silo. Due to the relative rigidity of the ring beam, the absolute settlements of all piles are very similar. Therefore, the outer row piles settle more relative to the surrounding soil mass, which results in mobilisation of higher shaft and end bearing resistances in these piles. In contrast, the soil mass inside the silo footprint undergoes a much larger settlement and the displacement of the inner row piles relative to the soil is correspondingly lower; hence, less resistance is mobilised in the inner row piles. The middle row piles mobilise the least resistance, as the soil mass around these piles is being loaded by adjacent piles in both the inner and outer rows; hence, middle row piles experience the least relative pile/soil displacement.

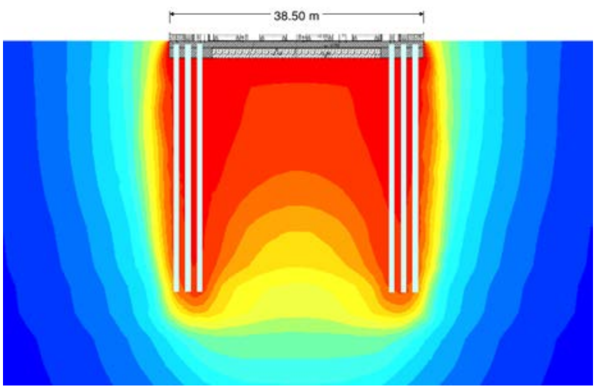

Figure 6. Vertical settlement contour plot

7 CONCLUSION

This paper presented the analysis undertaken as part of the design of the foundation system for a large cement silo storage facility. A satisfactory outcome was achieved in terms of both geotechnical strength and serviceability performance. The study showed that 3D FEM analysis can be employed to understand the group performance of the foundation system and the distribution of load to foundation elements. The results were used successfully to develop an optimised design solution for the piling scheme. The nature of the load transfer between the piles and the ring beam was able to be assessed, as well as the distribution of loads to individual piles and inner, middle and outer pile rows. The analysis indicated that the distribution of vertical loads was highly non-uniform among piles in different rows, which can be explained by the relative displacements experienced by piles in each row relative to the surrounding soil. The study illustrated the importance of considering the foundation system as a group, in order to avoid over-conservatism in the pile design.

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