SOME INADEQUACIES OF COMMON DESIGN PROCEDURES FOR DEEP FOUNDATIONS

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ABSTRACT

This paper examines some aspects of common deep foundation design that the author considers may be inadequate. The following aspects are considered:

- Ignoring foundation interactions;
- Ignoring the beneficial effect of the raft;
- Assuming a rigid cap or raft;
- Over-simplification of the geotechnical profile;
- Ignoring the beneficial effects of basement walls;
- Ignoring the effects of ground movements;
- Ignoring kinematic effects in seismic design.

Each inadequate aspect will be considered in turn, with examples given of the possible consequences. Some aspects lead to conservative designs, while others tend to be unconservative. Suggestions will be offered for addressing the perceived inadequacies, some of which are likely to involve the application of innovative techniques.

1 INTRODUCTION

In relation to design, "innovation" is a word that tends to be over-used and abused, as a descriptor of processes or ideas. A simple dictionary definition of "innovation" is as follows:

The process of translating an idea, invention or process into a good or service that creates value for which customers will pay.

One of the characteristics of innovation is that the innovative idea or process must satisfy a specific need, and thus such a need requires recognition and explanation. This paper will not address innovation overtly, but will focus on a number of perceived inadequacies in common design practice for deep foundations, which are as follows:

1. Ignoring foundation interactions;
2. Ignoring the beneficial effect of the raft;
3. Assuming a rigid cap or raft;
4. Over-simplification of the geotechnical profile;
5. Ignoring the beneficial effects of basement walls;
6. Ignoring the effects of ground movements;
7. Ignoring kinematic effects in seismic design.

Each inadequate aspect will be considered in turn, with examples given of the possible consequences. Some suggestions will be offered for addressing the perceived inadequacies.

2 IGNORING FOUNDATION INTERACTION EFFECTS

As an example of the effect of ignoring interaction within a piled raft system, the case is considered of a high-rise building in Doha, Qatar, an impression of which of shown in Figure 1.

The tower was designed to have a central high-rise tower 510 m tall, which was to be surrounded by a low-rise podium area. The foundation system was designed as a piled raft.

The foundation system is shown in Figure 2 and consisted of the following components:

1. 525 piles, with diameters of 1.0, 1.2 and 1.5 m.
2. Piles founded at four different levels: RL -26 m, -29 m, -54 m, and -60 m.
3. A raft thickness of 4.0 m for the majority of the foundation footprint.
4. Locally thickened areas of the raft beneath the lift over-run and core-wall areas of 6 m, 8.3 m and 12 m thickness.
5. A raft thickness of 0.8 m below the outer podium area.

Figure 1: Doha tower (artist's impression)

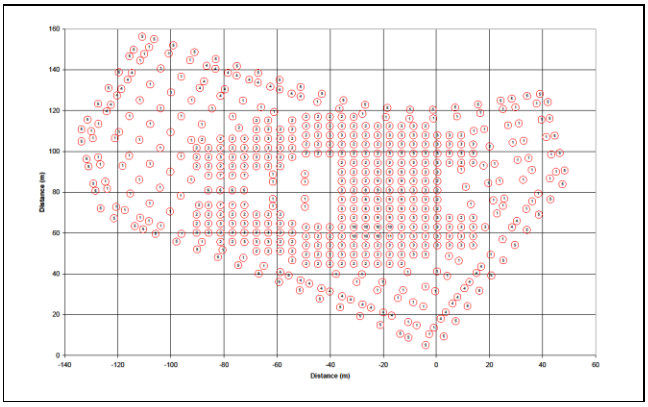

Figure 2: Foundation layout

The geotechnical model developed for the site was based on available in-situ and laboratory test data, and is shown in Table 1.

Table 1: Geotechnical model for Doha site

Stratum	*Top RL m*	*Thickness m*	*E'$_v$ MPa*	*E'$_h$ MPa*	*f_s kPa*	*f_b kPa*
Simsima L'stone	-18	3.5	2500	1750	600	-
Midra Shale	-21.5	3.0	700	490	525	-
Rus (1)	-24.5	75.5	500	350	425	5.9
Rus (2)	-100	Large	1000	-	-	-

The column loads were applied as uniformly distributed loads over the base area of the columns for the assessment of the pile loads and as one uniformly distributed load over the whole area of the tower footprint (0.95 MPa) for the assessment of the settlement and pile stiffness values. The serviceability assessment used the dead load plus the live load.

Analyses were undertaken to compute the settlement and pile load distribution within the foundation system, taking account of the flexibility of the raft foundation. The computer program GARP (Small and Poulos, 2007) was employed, using a finite element formulation to model the raft and idealizing the piles as non-linear interacting springs. A raft thickness of 4 m was used in the analyses, with the finite element mesh for the raft having a total of 945 elements and 3006 nodes.

The analyses carried out are listed in Table 2.

Table 2: Summary of analyses for Doha Tower

Run No.	*Details*
Q1	Normal analysis – all interactions included
Q2	Zero pile-pile interactions, but raft-raft, pile-raft and raft-pile interactions included
Q3	Zero pile-pile, pile-raft and raft-pile interactions; only raft-raft interaction accounted for

The computed maximum settlement for the three cases in Table 2 are shown in Figure 3. It can be seen that ignoring the pile-pile interactions reduces the maximum settlement from 81 mm to 41 mm, and ignoring the raft-pile and pile–raft interactions as well further reduces the maximum settlement to 20 mm. Thus, the settlement could be underestimated by a factor of 4 in this case if no consideration is given to the interactions among the piles and with the raft. Unfortunately such an approach is not uncommon among designers that are focused primarily on the structure itself.

The effect of ignoring interactions on the maximum rotation and the maximum raft bending moment in the x-direction are shown in Figures 4 and 5. The computed rotation becomes smaller if the interactions are ignored, with the maximum computed rotation decreasing by about 25% if all interactions are ignored. In contrast, the effect of ignoring interactions on the maximum bending moment is less marked.

Figure 6 compares the calculated maximum axial load in any of the piles. Ignoring all interactions (other than raft-raft) leads to a significant increase in the maximum pile load (almost 25%), and consequently, to more stringent requirements for reinforcement of the piles.

Clearly, it is vitally important not to ignore the interactions that exist within a pile raft foundation system. To do so gives an unconservative estimate of settlement and differential settlement, but a conservative estimate of axial pile loads.

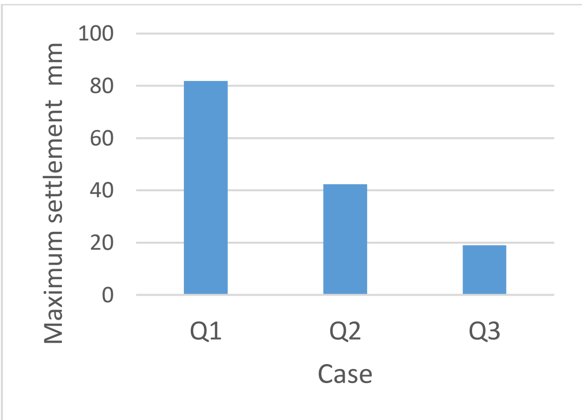

Figure 3: Effect of ignoring foundation component interactions on maximum settlement

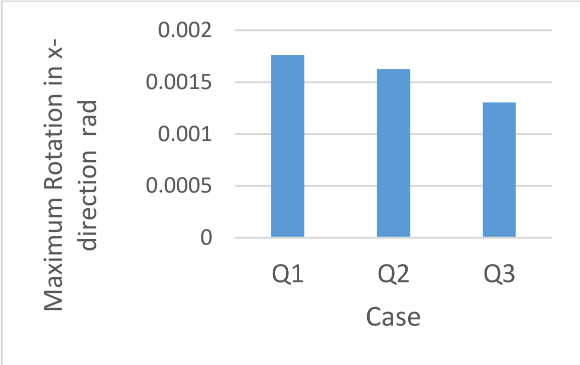

Figure 4: Effect of ignoring foundatiuon component interactions on maximum x-rotation

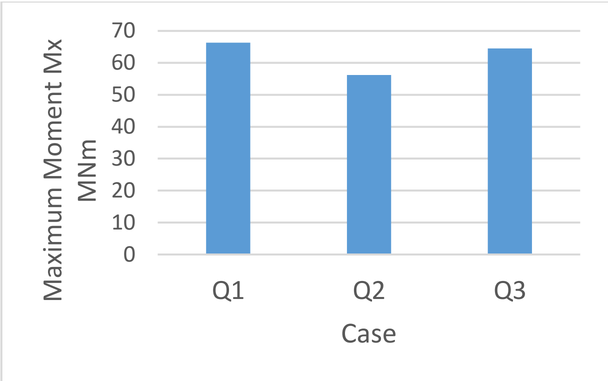

Figure 5: Effect of ignoring foundatiuon component interactions on maximum x-moment

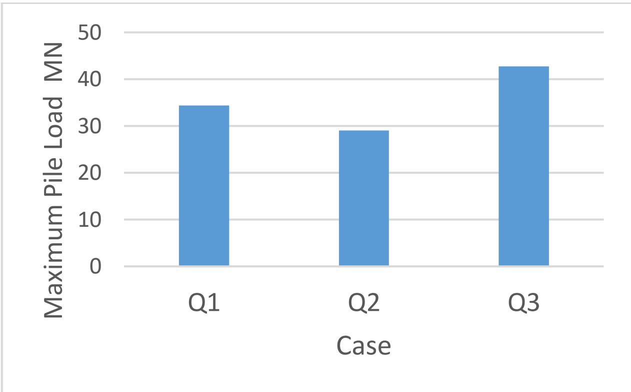

Figure 6: Effect of ignoring foundation component interactions on maximum axial pile load

2.1 IMPROVEMENTS IN PRACTICE

Improvements in design practice may be achieved in the following ways:

1. Emphasising that load test data on single piles should not be used directly to represent piles within a group, but that such data should be interpreted properly and adjusted to allow for group effects.
2. Using design methods and software that take proper consideration of the various foundation interactions.
3. Assisting the structural designer by providing values of pile stiffness values that take proper account of these interactions.

3 IGNORING THE PRESENCE OF THE RAFT

It is not uncommon for foundation designers to ignore the effect of raft-soil contact and to assume that the piles carry the entire structural load.

For the same case as considered above, ignoring the effect of the raft-soil contact can be simulated by setting the limiting pressure on the base of the raft to (almost) zero. All the load is then carried by the piles, which are now free-standing. Table 3 compares various aspects of the computed behaviour of the foundation system. The following characteristics are noted when the presence of the raft is ignored:

1. The computed maximum settlement is increased dramatically.
2. The maximum rotation is increased dramatically.
3. The maximum pile load is almost doubled.
4. The maximum bending moment in the raft is increased, but by a more modest amount than the other characteristics.

It seems clear that such a design, based on this over-conservative approach, would be inadequate and would almost certainly not satisfy the serviceability criteria, despite the presence of 525 piles in the system. However, by taking rational account of the presence of the raft, the settlements and rotations of the foundation are much more likely to be acceptable and to satisfy the serviceability design criteria.

Table 3: Effect of Ignoring the Raft Resistance

Value	*Allowing for raft*	*Ignoring the raft*
Max settlement mm	81.8	174.6
Min settlement mm	7.6	7.2
Max x-rotation rad	0.00176	0.01260
Max x-moment MNm	66.3	70.3
Min x-moment MNm	-46.2	-45.0
Max pile load MN	34.4	67.2
% load on raft	24.6	0

A. IMPROVEMENTS IN PRACTICE

Overcoming this perceived deficiency is relatively simple:

1. Recognition of the beneficial effects that the raft can provide;
2. Using a design approach that considers the effects of raft stiffness and capacity and that properly accounts for the various pile and raft interactions within the foundation system.

4 ASSUMING A RIGID PILE CAP OR RAFT

When designing or analyzing pile groups or piled rafts, it is common to make the simplifying assumption that the pile cap or raft is perfectly rigid. Because rafts in some modern high-rise buildings can be as thick as 5-6m, a rigid raft assumption may at first sight seem very reasonable. However, making this common assumption can lead to misleading outcomes, as it tends to over-estimate the loads in the outer piles within the system and under-estimate the loads in inner piles. As a consequence, the computed values of pile head stiffness may also be affected.

This leads on to the following important question: how thick does a raft have to be to be considered as rigid? To answer this question, recourse may be made to the work of Brown (1969), who considered the behaviour of a flexible circular raft on a finite elastic layer. Brown defined the relative flexibility of the raft via a factor K, given by:

$$K = E_r(1-\nu_s^2)(t/a)^3 / E_s \quad (1)$$

where E_r = Young's modulus of raft

ν_s = Poisson's ratio of soil

t = raft thickness

a = raft radius

E_s = Young's modulus of soil.

Brown's results indicated that a raft could be considered as perfectly flexible if $K \leq 0.01$, and virtually rigid if $K \geq 10$.

The criterion for rigidity can be facilitated by assuming that the factor K also applies to a rectangular raft having an area equal to that of the circular raft. If the average dimension of the raft is B, so that the area is B^2, then the requirement for rigidity can be approximated as follows:

$$(t/B)_{rigid} \approx \sqrt{\pi}.\ [K_{rigid}/(E_r/E_s).\ (1-\nu_s^2)]^{1/3} \quad (2)$$

where K_{rigid} = value of K for a rigid raft, i.e. 10.

A similar equation can be written for the relative thickness, $(t/B)_{flex}$, when a raft is perfectly flexible, by substituting, in Eq. 2, the value of K for a flexible raft (i.e. 0.01) instead of that for a rigid raft.

Figure 7 plots the relationship between the relative raft thickness, t/B, for both rigid and flexible rafts, for typical values of E_r (30000MPa) and ν_s (0.3). Rafts with a t/B value on or above the line for a rigid raft would be classed as rigid, those falling on or below the line for a flexible raft would be flexible, while those falling between the lines for rigid and flexible rafts would be classed as partially flexible.

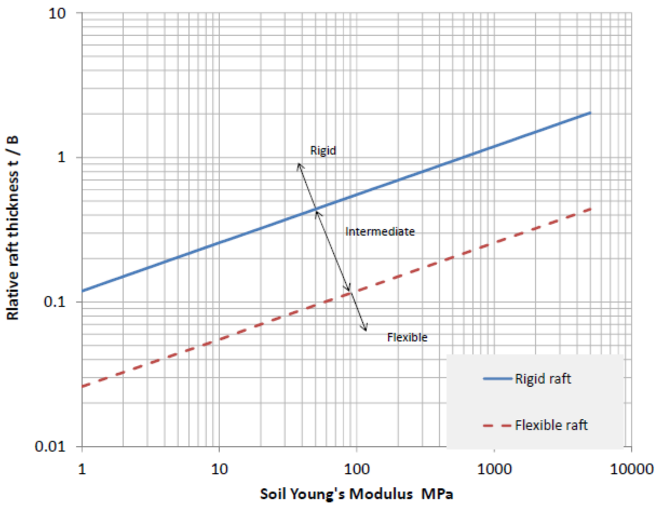

Figure 7: Thickness requirements for rigid and flexible rafts

The following points can be noted:

- The value of $(t/B)_{rigid}$ for a rigid raft increases as the soil modulus increases.

- Even for very soft soils, for example Es = 10 MPa, $(t/B)_{rigid}$ is about 0.25. Thus, for an average dimension of 50m, the raft would need to be about 12.5m thick to be truly rigid.
- For a very stiff soil layer, for example, Es = 500 MPa, $(t/B)_{rigid}$ is almost 1.0. Thus, for an average dimension of 50m, the raft would need to be about 50m thick!
- For a more common raft thickness of 3m, a raft with an average dimension of 50m would have t/B = 0.06, and this would be almost perfectly flexible even for a soft soil, and certainly perfectly flexible for the very stiff soil.

It therefore seems clear that pile caps and piled rafts supporting high-rise structures are likely to tend towards the perfectly flexible category.

As an example of the effects of assuming a rigid pile cap, the case of the 151 storey Incheon Tower, shown in Figure 8, has been considered. The foundation layout is shown in Figure 9.

Figure 8: Incheon Tower (artist's impression)

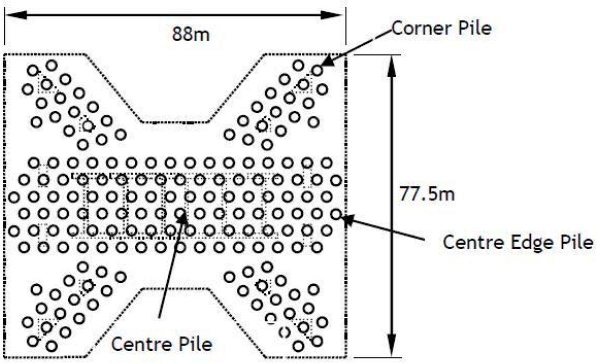

Figure 9: Pile Layout Plan Incheon Tower (*Abdelrazaq et al., 2011*)

The detailed design analyses were carried out using the program CLAP (Combined Load Analysis of Piles, Coffey, 2011) and a later version of the program GARP (Small and Poulos, 2007), CLAP is a development of the DEFPIG program (Poulos, 1990), and was used for the ultimate limit state load cases (ULS). GARP was used for analysing the serviceability (SLS) loadings. As part of the design process, estimates were required of the maximum axial loads in each pile within the foundation system, and initially, the program CLAP was used. CLAP implicitly assumes that the raft supporting the piles is rigid, and as a consequence, the computed axial loads on some piles were found to be very large.

To investigate the effect of the rigid raft assumption, the foundation performance was re-assessed using GARP, taking the flexibility of the raft into account. The serviceability load case (i.e dead and live loads) was considered and the loads were applied at column and core locations.

Table 4 presents a summary of foundation settlement, axial loads and stiffness on the corner, centre edge and centre piles of the foundation (see Figure 9). The maximum predicted settlement occurred within the heavily loaded core area, while the computed pile stiffness values were greatest for the outer piles. As the analysis considered non-linear pile behaviour, the outer piles had a higher initial stiffness, but this stiffness degraded more rapidly under increasing loading than the central piles.

Considering a rigid raft for the foundation, the total and differential settlements were predicted to be smaller, with higher pile head loads for corner and centre-edge piles, thus resulting in higher vertical pile stiffness values, especially on the outer piles, when compared with those for a flexible raft.

When the flexibility of the raft was incorporated, the pile load distribution was found to be more uniform, with slightly higher pile loads being predicted at the centre of the foundation where the heavily loaded core is located. The loads on piles for a rigid raft case were approximately two times the loads for a flexible raft, except for the centre piles.

Table 4: Summary of foundation performance

		Rigid Raft	Flexible Raft
Pile Load (MN)	Centre Pile	24	49
	Centre Edge Pile	65	33
	Corner Pile	85	43
Pile Stiffness (MN/m)	Centre Pile	511	726
	Centre Edge Pile	1418	932
	Corner Pile	1604	1292
Raft Settlement (mm)	Maximum	52	67
	Minimum	26	28

It is interesting to refer to Figure 7 to assess the relative flexibility of the 5.5m thick raft. The average dimension of the foundation was about 70m, so that the ratio t/B was about 0.08. The average Young's modulus within a depth equal to this dimension was about 275 MPa, and for this modulus, the value of t/B for a rigid raft would be about 0.75, i.e. the raft would need to be about 52.5m thick. In fact, even for a flexible raft, the value of t/B from Figure 13 would be about 0.17, so that the raft, with a t/B of less than half this value, could be classed as perfectly flexible. Based on the assessment, it is concluded that it is important to model the flexibility of the raft to avoid having to design for unrealistically large loads in the outer piles within the group, and also to obtain more realistic distributions of settlement within the foundation footprint.

A. IMPROVEMENTS IN PRACTICE

The potential inaccuracies of assuming a rigid raft can be overcome via the following procedures:

- Checking whether the ratio of raft thickness to raft size justifies a rigid raft assumption;
- If not, using an analysis that takes account of raft flexibility. This may be via a piled raft analysis, a structural analysis in which relevant values of the pile stiffness are used, or via a three-dimensional finite element, or finite difference, analysis, in which all components of the foundation system are appropriately modelled.

5 OVER-SIMPLIFICATION OF THE GEOTECHNICAL PROFILE

Over-simplification of the geotechnical profile can occur for several reasons, including:

1. Inadequate ground investigation to an appropriate depth which will be influenced by the foundation;
2. The use of simplified analysis and design tools that do not allow readily for variable ground conditions below the founding level of the foundation system;
3. Inadequate attention given to the potential variability of ground stiffness with depth, even in a relatively homogeneous geo-stratum.

Two examples will be given below to illustrate the consequences of ground profile over-simplification.

5.1 ASSUMED UNIFORM CONDITIONS BELOW FOUNDATION LEVEL

It is not uncommon for foundation analysts to assume that the ground conditions below the pile tips remain constant and extend to relatively large depths. The consequences of this assumption may be that pile-pile interactions are over-estimated. Such an over-estimation was experienced by the author in relation to the foundation design for the Emirates twin towers in Dubai (Poulos and Davids, 2005). These towers are shown in Figure 10.

Figure 10: The Emirates Towers soon after completion of construction

Predictions of the behavior of a single test pile were found to be in reasonable agreement with the measured behaviour of test piles. On this basis, the parameters developed for a single pile were used to predict the settlement of the piled rafts supporting the two separate towers.

Conventional pile capacity analyses were used to assess the ultimate geotechnical capacity of the piles and raft. In additional to the conventional analyses, more complete analyses of the foundation system were undertaken with an early version of the computer program GARP (Poulos, 1994). This program utilized a simplified boundary element analysis to compute the behaviour of a rectangular piled raft when subjected to applied vertical loading, moment loading, and free-field vertical soil movements. The raft was represented by an elastic plate, the soil was modelled as a layered elastic continuum, and the piles were represented by hyperbolic springs which can interact with each other and with the raft. Beneath the raft, limiting values of contact pressure in compression and tension were specified, so that some allowance could be made for non-linear raft behaviour. In addition to GARP, the program DEFPIG (Poulos and Davis, 1980) was used for the pile stiffness values and pile-pile interaction factors, and for computing the lateral response of the piles.

For the analysis of settlements under the design loads, the same values of Young's modulus were used as for the single piles. Measurements were available only for a limited period during the construction process and these are compared with the predicted time-settlement relationships in Figure 11 for typical points within the Hotel Tower. To the author's disappointment, it was found that, for both towers, the actual measured settlements were significantly smaller than those predicted, being only about 25% of the predicted values after 10-12 months.

The disappointing lack of agreement between measured and predicted settlement of the towers prompted a "post-mortem" investigation of possible reasons for the poor predictions. At least four such reasons were examined:

1. Some settlements may have occurred prior to the commencement of measurements;
2. The assumed time-load pattern may have differed from that assumed;
3. The rate of consolidation may have been much slower than predicted;
4. The interaction effects among the piles within the piled raft foundation may have been over-estimated.

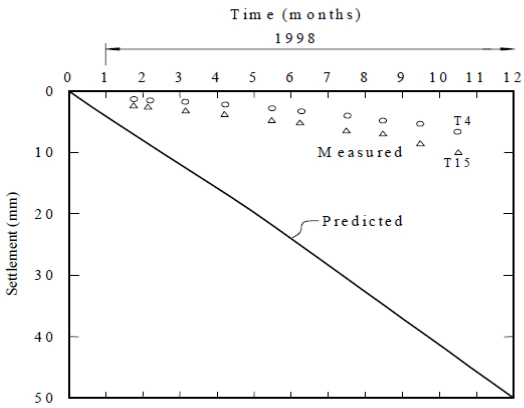

Figure 11: Predicted and measured time-settlement behaviour of Hotel tower

Of these, based on the information available during construction, the first three did not seem likely, and the last was considered to be the most likely cause. Calculations were therefore carried out to assess the sensitivity of the predicted settlements to the assumptions made in deriving interaction factors for the piled raft analysis with GARP. In deriving the interaction factors originally used, it had been assumed that the soil or rock between the piles had the same stiffness as that around the pile, and that the rock below the pile tips had a constant stiffness for a considerable depth. In reality, the ground between the piles is likely to be stiffer than near the piles, because of the lower levels of strain, and the rock stiffness below the pile tips is likely to increase significantly with depth, both because of the increasing level of overburden stress and the decreasing level of strain. The program DEFPIG was therefore used to compute the interaction factors for a series of alternative (but credible) assumptions regarding the distribution of stiffness both radially and with depth. The ratio of the soil modulus between the piles to that near the piles was increased to 5, while the modulus of the material below the pile tips was increased from the original 70 MPa to 600 MPa (the value assessed for the rock at depth). The various cases are summarized in Table 7.

Figure 12 shows the computed relationships between interaction factor and spacing for a variety of parameter assumptions. It can be seen that the original interaction curve used for the predictions lies considerably above those for what are considered (in retrospect) more realistic assumptions. Since the foundations analyzed contained many piles, the potential for over-prediction of settlements is considerable, since small inaccuracies in the interaction factors can translate to large errors in the predicted group settlement. In addition, Al-Douri and Poulos (1994) indicate that the interaction between piles in calcareous deposits may be much lower than those for a laterally and vertically homogeneous soil. Unfortunately, this experience was not incorporated in the Class A pile group settlement predictions for the towers.

Curve No.	Modulus of Layer below MPa	Modulus of Soil between Piles to Near-Pile Values
1	80	1.0
2	80	5.0
3	200	5.0
4	600	5.0
5	600	1.0

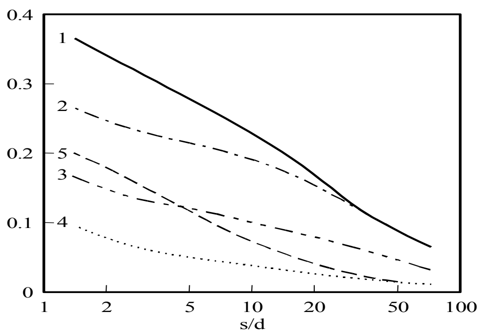

Figure12: Sensitivity of computed interaction factors to analysis assumptions

Table 5: Summary of Revised Calculations for Hotel Tower

Case	*Modulus below 53 m MPa*	*Ratio of max. to near-pile modulus*
Case 1: (Original calculations)	80	1
Case 2	80	5
Case 3	200	5
Case 4	600	5
Case 5	600	1

Revised settlement calculations, on the basis of these interaction factors, gave the results shown in Figure 13. The interaction factors used clearly have a great influence on the predicted foundation settlements, although they have almost no effect on the load sharing between the raft and the piles. For Case 4, the maximum settlement is reduced to 29% of the value originally predicted, while the minimum settlement was only 25% of the original value. If this case were used for the calculation of the settlements during construction, the settlement at Point T15 after 10.5 months would be about 12 mm, which is in reasonable agreement with the measured value of about 10 mm.

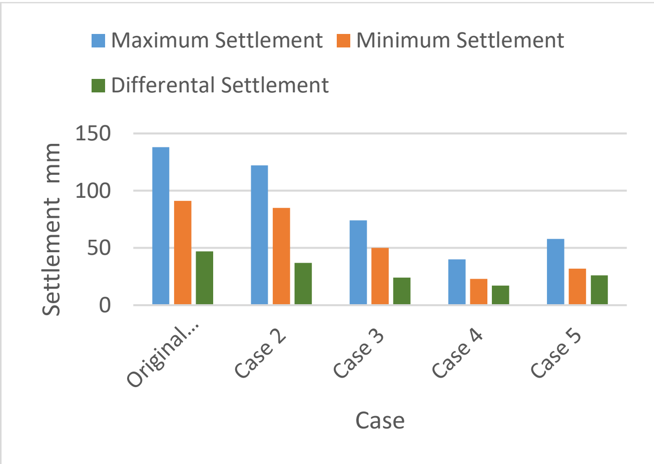

Figure 13: Effect of ground profile assumption on computed settlements

This project demonstrated the vital importance of proper characterization of the ground, not only along the piles, but also beneath the piles. Especially for foundation systems of considerable width (as is typical of tall buildings), the assumptions made about ground stiffness at depth can have a profound effect on the computed settlements.

5.1.1 Improvements in Practice

The inadequacies of over-simplifying the ground profile and assuming constant properties with depth can be countered by:

1. Carrying out a proper ground investigation to the depth that will be influenced by the foundation system;
2. If use is made of a method of analysis which involves interaction factors, they should be computed for the actual ground profile and not for idealised case where the properties remain constant with depth.
3. The interaction factors used should allow for a limit to the distance within which pile-pile interaction occurs, and account should also be taken of the fact that the ground stiffness increases with decreasing strain level.

4.2 EFFECT OF IGNORING COMPRESSIBLE UNDERLYING LAYERS

Golder and Osler (1968) have described an interesting case of a series of furnace foundations on piles, which were founded at a relatively high level, well above a deep layer of compressible Leda Clay. Figure 14 shows the stratigraphy of the site and some of the key engineering properties revealed by the investigations. The configuration of the pile group is also shown in this figure. A number of the original boreholes extended to depths up to 236 feet (72m) without encountering bedrock.

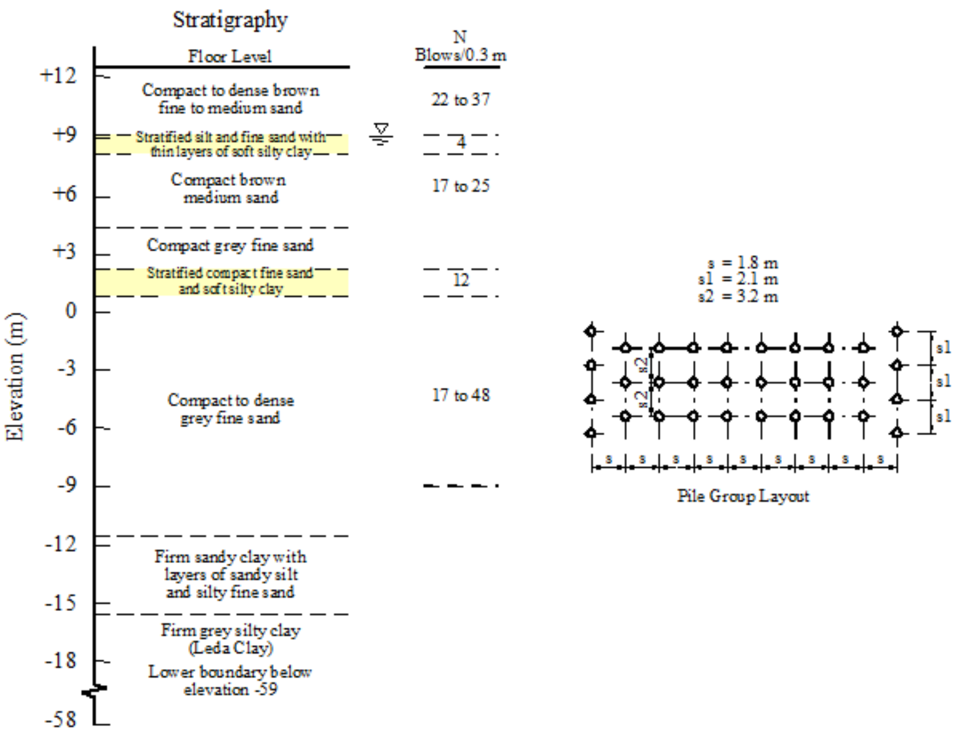

Figure 14: Stratigraphy and pile group layout for furnace foundation (Golder and Osler, 1968)

A load test was carried out on a pile similar to that used for the furnace foundations. At a typical working load of 75 US tons (668 kN), the measured settlement was about 0.04 inches (1.0 mm). Applying normal pile group settlement theory to this result, it might have been expected that the settlement of a 32-pile furnace group would have been of the order of 3 to 6 mm.

15 years of settlement records were available for a bank of five furnaces, and these measurements enable some interesting conclusions to be drawn regarding the sources of settlement of the foundations. Figure 15 reproduces the measured settlements over the bank of five furnaces, and reveals the following interesting characteristics:

- The maximum settlement nearly 15 years after construction was about 73 mm and was continuing to increase;
- The measured settlements were an order of magnitude greater than those which may have been expected simply on the basis of the pile load test;
- The settlement of the end furnaces (Furnace 1 and Furnace 5) was clearly affected by the other furnaces, and showed a significant tilt.

It was estimated by Golder and Osler that, taking into account the settlement of the compressible layers below the pile tips, the anticipated final settlement of the end furnace (No.1) could be of the order of 87 mm, consisting of 10 mm of pile group settlement, 13 mm consolidation of the silty clay layer below the pile tips, and 64 mm from the deep Leda clay. Clearly, the major source of settlement in this case was the underlying compressible clay layer. This had little or so effect on the settlement of the test pile, but had a critical influence on the settlement of the extensive pile group.

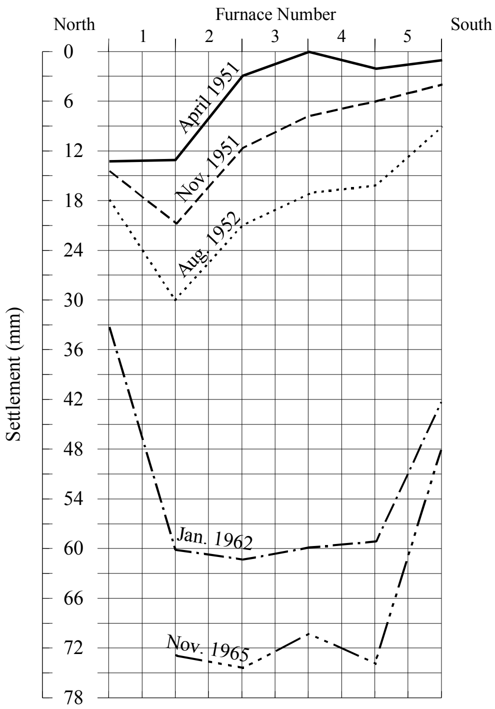

Figure 15: Settlement observations along north-south line through furnaces 1 to 5 (Golder and Osler, 1968).

5.2.1 Improvements in Practice

The inadequate design approach revealed in this case history can be improved via the following processes:

1. Carrying out a detailed ground investigation to a depth that the entire foundation system will influence. This case clearly demonstrates the importance of taking account of the compressibility of underlying compressible layers below the pile tips;
2. Considering the interaction among adjacent foundations.
3. Avoid relying solely on the results of a single pile load test to predict pile group behaviour, without a proper appreciation of the ground conditions.

6 IGNORING THE BENEFICIAL EFFECTS OF BASEMENT WALLS

Many structures, especially tall buildings, incorporate a basement into the substructure to provide parking and storage facilities. In the design of foundation systems, the effect of the basement is often ignored when assessing the foundation capacity and stiffness, even though the basement may extend to a considerable depth below the surface.

Chow and Poulos (2019) have explored the effects of a basement on the capacity and stiffness of a piled or piled raft system, using the three-dimensional finite element program PLAXIS 3D. A numerical example was presented to illustrate the effects of a basement wall on the capacity and stiffness of the foundation system The wall was assumed to be rectangular in shape, of plan dimensions B_r x L_r, and embedded to a depth of D_r below the ground surface, as shown in Figure 16. The direction of lateral loading was parallel to the dimension L_r.

A square raft of 16 m x 16 m in dimension was assumed to be supported by a 4 x 4 pile group with a centre-to-centre spacing of 4 m, embedded in a deep uniform stiff clay profile. The piles had a diameter of 1 m and a length of 24 m. The piled raft was assumed to be rigidly connected to the basement walls.

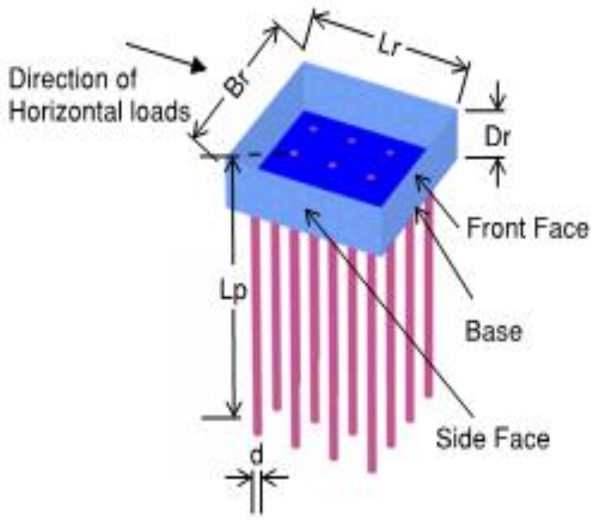

Figure 16: Geometry of basement wall and foundation system

The foundation system was subjected to rapidly applied loading, such that the soil would have an undrained behavior with an undrained shear strength of 80 kPa. A constant shear resistance of 50 kPa was assumed along basement walls and the beneath the underside of the raft. The ultimate skin friction (f_s) on the piles was assumed to be 56 kPa based on the α-method, $f_s = \alpha s_u$. The ultimate bearing capacity (f_b) and lateral yield pressure were assumed to be $9s_u$ = 720 kPa. The parameters used for the analysis are summarized in Table 6. Figure 17 illustrates the foundation layout in plan and section.

The finite element mesh used for the analysis employed a total of 19836 elements and 29568 nodes. The soil was modeled as a homogenous continuum obeying the Mohr Coulomb criterion. The piles were modeled by embedded beam elements with interface elements, while the raft and basement walls were modeled by plate elements. In order to simulate the slip along the raft base and basement walls, a thin layer of 0.1 m thick with the strength as specified (base shear of raft and shear resistance along basement wall) was introduced underneath the raft and adjacent to the basement walls.

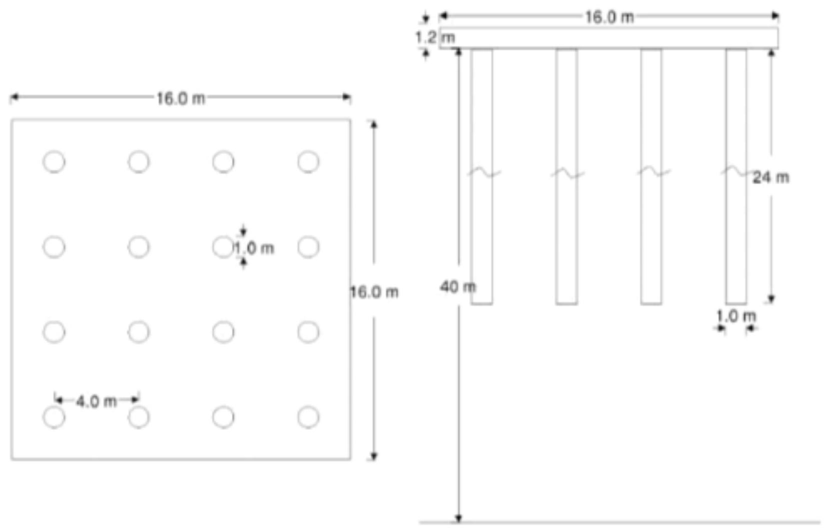

Figure 17: Details of foundation system analysed

The depth of the basement wall depth varied from 0 to 10 m and the foundation system was subjected to three load cases:

- uniform vertical loading,
- uniform horizontal loading, and
- bending moment applied at the centre of the foundation.

Table 6: Parameters for problem considered

Parameter		*Value*
Young's Modulus of Clay (MPa)	Ev (vertical)	50
	Eh (horizontal)	35
Undrained Shear Strength of Clay, cu (kPa)		80
Ultimate Skin Friction (kPa)		56
Ultimate End Bearing (kPa)		720
Young's Modulus of Pile (MPa)		30,000
Young's Modulus of Raft (MPa)		30,000
Thickness of Raft (m)		1.2
Base Shear along raft (kPa)		50
Young's Modulus of Basement Wall (kPa)		30,000
Thickness of basement wall (m)		0.5
Shear Resistance along Basement Wall		50

Figure 18 presents the computed percentage increase in vertical and lateral capacity as compared with those of the piled raft with no wall. The results indicate that with wall embedment, both the vertical and lateral capacity of the foundation system increase. The horizontal capacity increases dramatically, while the vertical capacity increases relatively modestly. It should be emphasised that, with a wider foundation system, the relative increase in capacity will be less than that shown in Figure 18 for a relatively limited width of pile group.

The author has also attempted to undertake simplified analyses by representing the basement walls as a series of equivalent piles having the same axial stiffness and bending stiffness as the wall. The piles had a diameter equal to the thickness of the wall and a length equal to that of the wall. The parameters of the equivalent piles were such that they provided an equal axial and lateral capacity as that of the walls.

For axial loading, a later version of the program GARP was employed (Small and Poulos, 2007), while for lateral and moment loading, the program CLAP was used. In the latter cases, the pile cap was assumed to be rigid.

Figure 19 shows, for typical working load levels, the percentage reduction in responses, i.e maximum settlement under vertical load, lateral deflection under lateral load, and rotation under moment loading. It can be seen that in all three cases, there is a reduction in response with increasing wall depth, with the lateral deflection and rotation experiencing the largest reductions. For the lateral loading, the rate of reduction of lateral deflection decreases once the wall depth exceeds about 6m. This tends to reflect the fact that the wall has an effective length which, when exceeded, results in little or no additional reduction in deflection

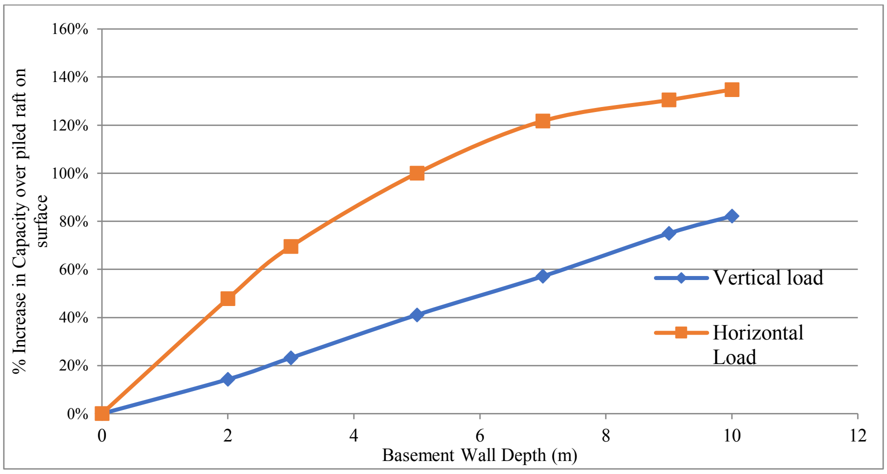

Figure 18: Percentage increase in capacity due to basement wall embedment (Chow and Poulos, 2019)

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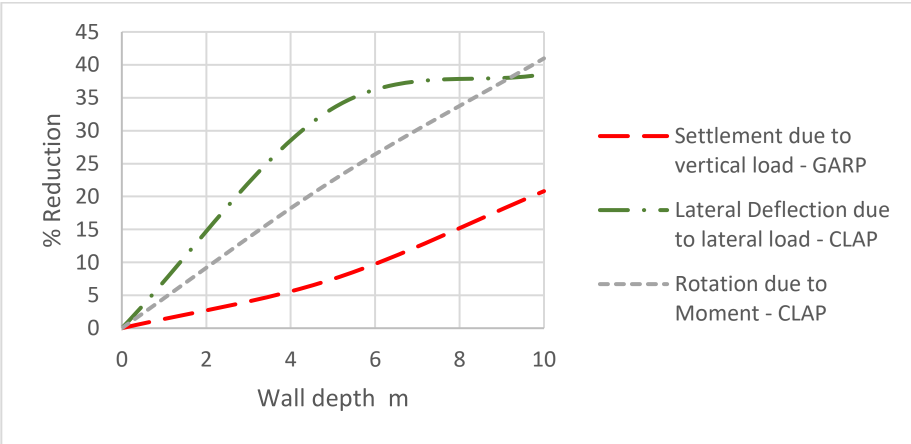

Figure 19: Reduction in responses due to basement wall

For axial loading, the reduction in maximum settlement is accompanied by an increase in the differential settlement, since the walls act to “hold up” the outer edges of the raft. However, for the case analyzed, the increase in differential settlement is relatively modest, being only about 12% for a 5 m deep wall, and about 20% for a 10 m deep wall.

At the same time, there is also a modest re-distribution of axial loads, as shown in Figure 20. The loads in the corner and mid-side piles tend to reduce, while the load on the centre piles increases. The piles nearest the corner of the raft are most affected, with a decrease in axial load of about 15% being experienced for the 10 m deep wall.

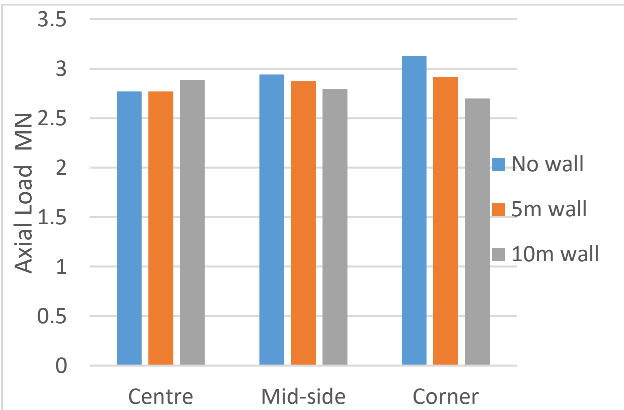

Figure 20: Effect of basement walls on axial load distribution

The presence of the walls has a “shielding” effect on the foundation piles and thus tends to reduce the bending moments developed within these piles. Table 7 shows the computed percentage reduction in pile head bending moments due to the wall, for both horizontal and moment loading. This reduction is most significant for walls up to about 5-6 m deep, and the effect diminishes thereafter. The reductions in pile head moment suggest that the requirements for structural design of the piles are reduced because of the presence of the walls.

Table 7: Percentage reduction in average pile head bending moment

Wall depth m	% reduction in head moment due to horizontal loading	% reduction in head moment due to moment loading
0	0	0
5	44	30
10	45	37

The implications of the above analysis results are as follows:

For both vertical and horizontal loads, the displacements decrease as the depth of basement walls increases.

The basement walls provide additional vertical and horizontal resistance to the foundation system, and thus can provide a larger margin of safety against failure (especially horizontal) than a piled raft without embedment.

The basement walls provide additional rotational stiffness to the foundation system thus contributing to the reduction of its angular rotation.

The induced bending moments within the foundation piles tend to be reduced significantly (in this example), with most of the benefit arising from relatively shallow walls. There is also a modest re-distribution of axial load within the foundation piles.

It should be recognized that the larger the area of the foundation footprint, the smaller will be the effect of the basement walls surrounding the foundation system. Nevertheless, it seems highly desirable to incorporate the basement walls into the foundation design to avoid undue conservatism in relation to the forces and bending moments in the piles, and on the other hand, to avoid under-estimating differential settlements in the vicinity of the walls.

6.1 IMPROVEMENTS IN PRACTICE

As a simple preliminary approach, the effects of basement walls may be allowed for as follows:

- By adding a proportion of the vertical and lateral resistances of the walls to the resistances of the foundation system, taking due account of the possible interactions between the walls and the deep foundations;
- By adding a proportion of the estimated vertical and lateral stiffnesses of the walls to those of the foundation system. Calibration of this proportion is ongoing, but as an initial estimate, a factor of 0.4 may be reasonable;
- By modelling the walls as a series of piles of equivalent vertical and lateral stiffness. The author is currently investigating the validity of this approach, and example of the outcomes are given above.

For detailed design purposes, a three-dimensional finite element, or finite difference, analysis can of course be undertaken, representing the basement walls as embedded structural elements with appropriate stiffness and interface strengths.

7 IGNORING EXTERNAL GROUND MOVEMENTS

In contemporary urban environments, it is not unusual for excavations of tunnelling works to be carried out in proximity to planned or existing deep foundations. If such works are known or anticipated, it is possible to make allowances in the design of the deep foundations. If they are carried out after construction of the deep foundations, then it is necessary to assess the impact of such works on the integrity and movement of the existing foundations.

Consideration is given here to the effects of tunnelling adjacent to a deep foundation, as shown in Figure 21. The ground movements due to the tunnel can be estimated via the equations developed by Loganathan and Poulos (1998) and the effect of these movements can be analysed as per the approach described by Chen et al (1999).

For the case shown in Figure 21, a volume loss of 1% is assumed for the 6m diameter tunnel, and the centreline of the tunnel is assumed to be 12m from the centreline of the pile, which is 36m long and 1.2m in diameter. The pile head is assumed to be fixed into a pile cap so that rotation is restrained. A horizontal load of 1 MN and a vertical load of 5 MN are applied at the pile head.

Pile-soil interaction analyses have been carried out to compute the induced horizontal displacement, bending moment, vertical settlement and axial load in the pile. These are shown in Figures 22 to 25 in turn.

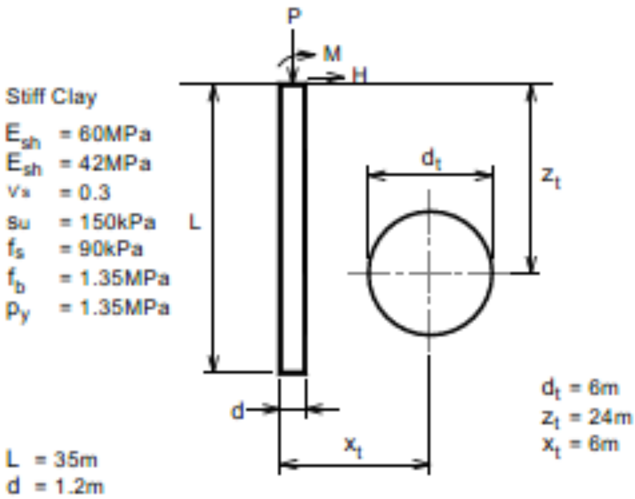

Figure 21: Example of a pile near a new tunnel

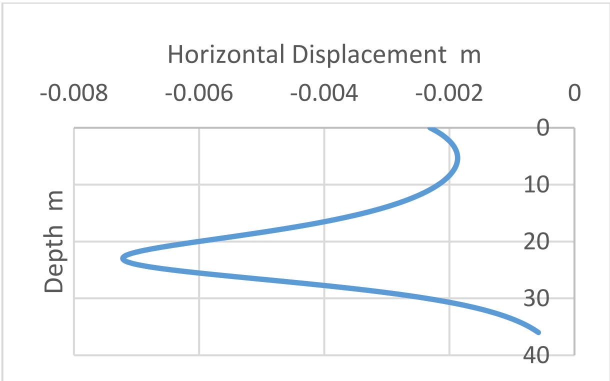

Figure 22: Horizontal displacements in pile due to applied load and tunnel

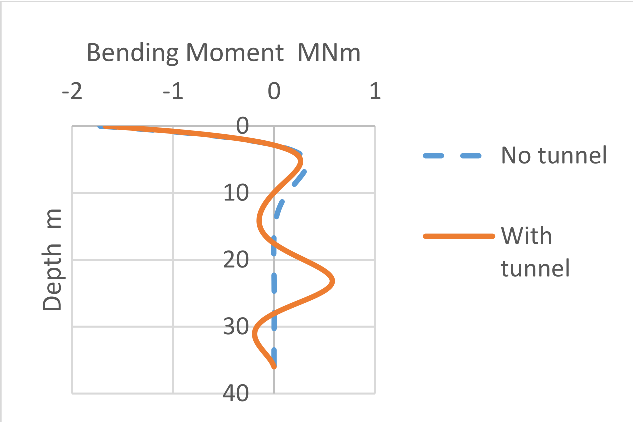

Figure 23: Bending moments in pile due to applied load and tunnel

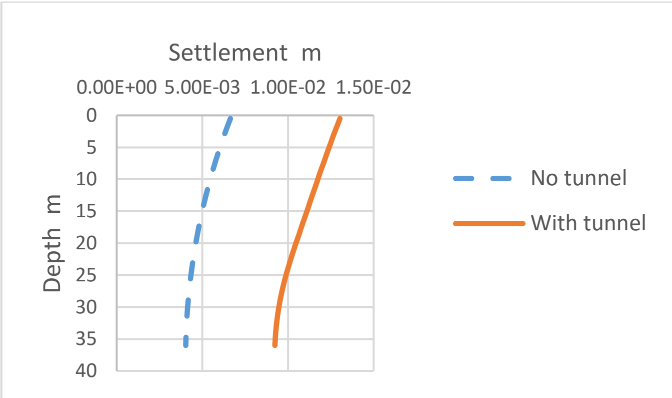

Figure 24: Vertical settlement of pile due to applied load and tunnel

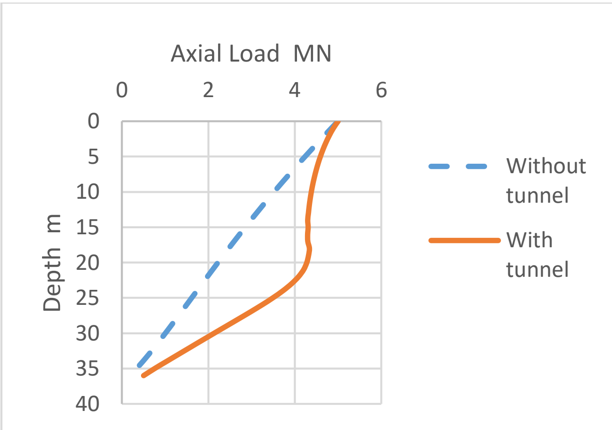

Figure 25: Axial forces in pile due to applied load and tunnel

From these figures, the following observations can be made:

1. The maximum horizontal displacement occurs near the level of the tunnel invert, and is significantly greater than the pile head displacement;
2. The maximum moment at the pile head is little affected by the tunnel, but there is a significant additional moment developed near the level of the tunnel invert. In many cases, this would be at a lower level than that at which the reinforcement is often terminated;

3. The tunnelling causes the pile head settlement to almost double in this case;
4. There is a “downdrag” component of axial force near the level of the tunnel invert, although in this case, the maximum axial force is still the applied force at the pile head.

This simple example illustrates that potentially harmful effects can arise from tunnel construction near existing piles. In particular, large bending moments can be developed in the pile near the level of the tunnel invert, which may be below the usual cut-off level for bored pile reinforcement. Consequently, if the possibility of future tunnelling operations is recognized, then consideration should be given to the use of full-length reinforcement.

Only a single pile is considered in this example, but group effects can be slightly beneficial and result in a modest reduction of the moments and axial forces induced in the pile.

A further example of the effects of ground movements often being overlooked is in the design of pile-supported embankments and bridge abutments. Many designers focus on the vertical response of the piles to the embankment loading, but significant lateral ground movements can be developed also, and these can cause significant bending moments and shears in the piles, especially those towards the edges of the embankment or abutment. Such cases have been considered by Stewart et al (1992, 1994) and Ellis and Springman (2001), while and some approximate design charts have been presented by Poulos (1994). It is clear that one-dimensional thinking (considering only vertical movements and effects) should be avoided when designing deep foundations for embankment-induced ground movements.

7.1 IMPROVEMENTS IN PRACTICE

Improvements in practice to cope with imposed ground movements may be made by:

- Recognizing or anticipating situations in which the ground surrounding the foundation may be liable to movements due to external influences; in many cases, lateral as well as vertical ground movements may occur.
- Taking account of such movements in making assessments of the foundation movements, and the forces and actions within the foundation system. Methods that can be used include simple hand methods (for example, the effects of vertical ground movements in imposing negative skin friction on piles), chart solutions for lateral ground movements, for example, Chen and Poulos (1997), or detailed finite element or finite difference analyses in which the source of ground movement is included within the modelling process.

It should be emphasized that group effects in such cases can be beneficial, and so estimating the response of a single pile is often both adequate and conservative.

8 IGNORING KINEMATIC SEISMIC EFFECTS

As a structure responds to seismic loading, inertial lateral loads are imposed on a foundation system. The influence of such inertial effects on the seismic response of pile foundations is well-recognized, and depends on the frequency content of the earthquake and the natural period of the pile-soil-cap-mass system. However, an earthquake also imposes kinematic actions on a deep foundation system because of the movement of the ground in response to the seismic excitation, and the consequent pile-soil interaction.

Mylonakis et al (1997) have identified the following characteristics:

1. Inertial bending can be significant, especially in the upper part of the piles, when the dominant period of the earthquake is similar to the fundamental period of the soil-pile-structure system.
2. Kinematic bending can be significant when the dominant period of the soil motions is similar to the natural period of the soil strata.

The three most likely areas of damage of a pile are the pile head, interfaces between layers of different stiffness, and the pile toe. Pile head damage is most likely in homogeneous strata while damage at strata interfaces is most likely when there is a marked stiffness contrast between the soil layers. The kinematic bending strains at the pile toe may be significant when the toe is restrained.

To facilitate an understanding of the relative importance of inertial and kinematic effects, analyses have been performed on the fixed head single pile shown in Figure 26. The analysis has been carried out via the pseudo-static approach described by Tabesh and Poulos (2001), so that the results provide an envelope of maximum bending moments and shears along the pile. It is assumed that the site is subjected to the 1994 Northridge earthquake with a maximum bedrock acceleration of 0.2g.Three cases have been considered:

- A pile with no vertical load/cap mass;
- A pile with a lateral inertial load of 0.2MN;

- A pile with the same lateral inertial load as in the second case, but where the kinematic ground movements are included in the analysis.

Figure 27 shows the computed distributions of bending moment along the pile. Two key points emerge from this figure:

1. If kinematic effects are ignored, and only inertial (lateral load) effects are considered, the maximum moment at the pile head can be seriously under-estimated.
2. If only inertial effects are considered, the moment at depths in excess of about 7m becomes insignificant, but with the kinematic effects incorporated, there is a significant moment between depths of about 7 to 10m, i.e. in the vicinity of the interface between the softer upper layer and the stronger lower layer.

The importance of considering both kinematic as well as inertial effects is clearly emphasized in this example. As with the example of ground movements due to tunnelling, it is possible that full-length reinforcement would be required for bored piles, especially if significant layering of the soil profile exists and there is a distinct stiffness contrast between adjacent layers.

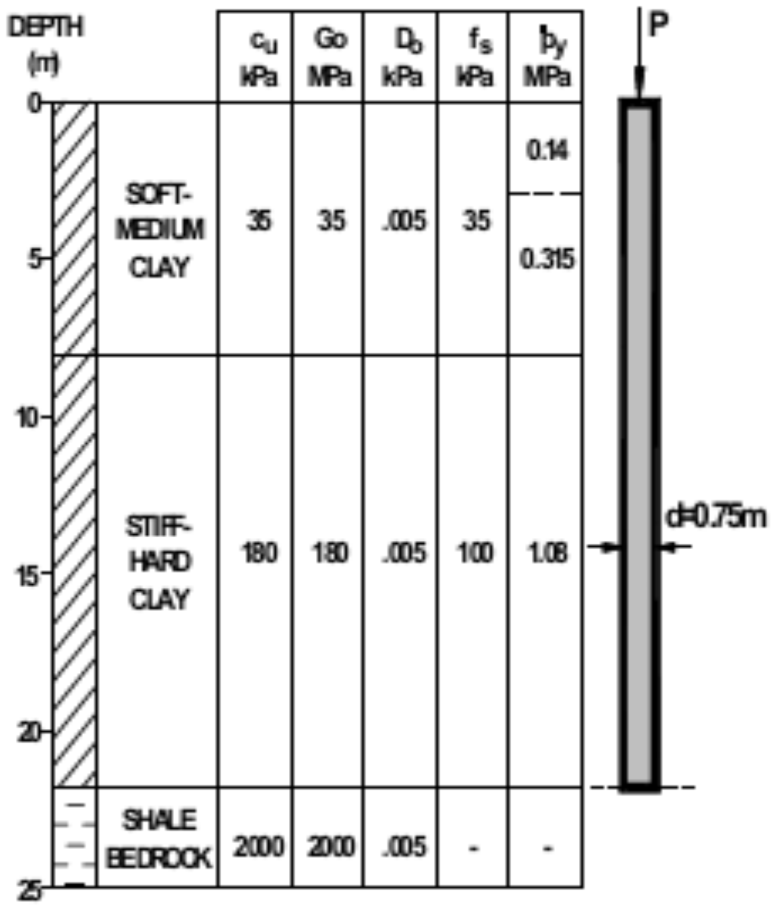

Figure 26: Example for effect of kinematic loading

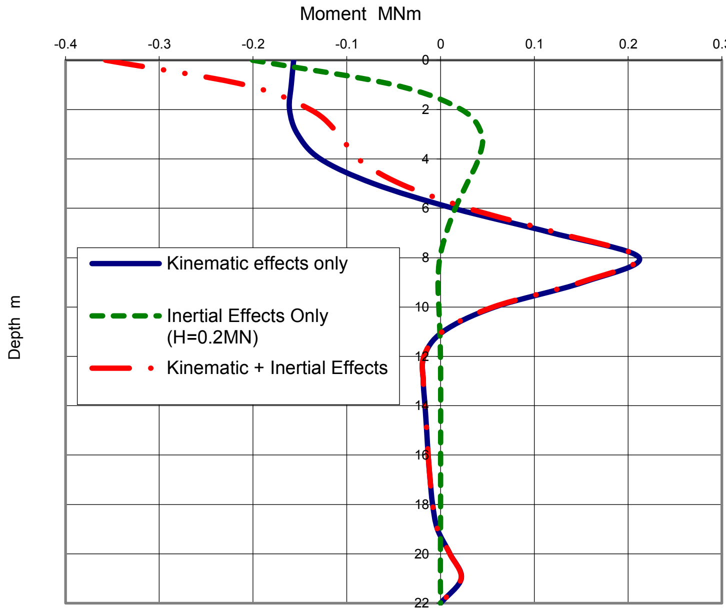

Figure 27: Computed bending moment distributions

A. IMPROVEMENTS IN PRACTICE

There are a number of relatively simple methods available for estimating the maximum moment in a pile due to kinematic soil movements. These include Nikolaou et al (2001), Di Laora et al (2012), and Dezi et al (2010) for single piles, and Dezi and Poulos (2016) for square pile groups. In all cases, the maximum moment tends to be at the interface between layers of differing stiffness. Such approaches are generally adequate to provide an indication of the potential for structural damage of the pile. If a more detailed investigation is warranted, then pseudo-static or full dynamic analyses are available.

9 ASSUMPTION OF ELASTIC BEHAVIOUR OF PILES

In designing piles for lateral loading, it is common to assume that a pile itself remains elastic during the entire loading process. While this may be a reasonable assumption for steel piles, it may be an over-simplification for concrete piles. A typical moment-curvature relationship for a bored pile is shown in Figure 28, and it can be seen that there is significant non-linearity in the relationship.

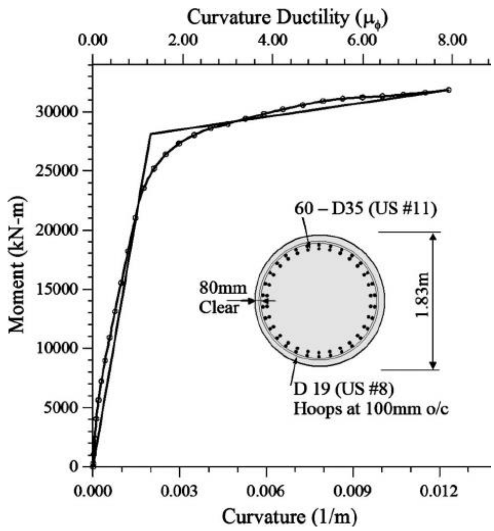

Figure 28: Typical moment – curvature relationship for bored pile

The effect of such pile non-linearity has been examined in a number of papewrs, for example, Ashour et al (2001), Kramer and Heavey (1988), Hsueh et al (2004). Kramer and Heavey have shown that even a simple elastic-plastic relationship for the pile can provide results that are in reasonable agreement with measurements of lateral pile behaviour. Figures 29 and 30 show an example of the improved agreement with observed behaviour when a simple non-linear elastic-plastic model is used for the pile.

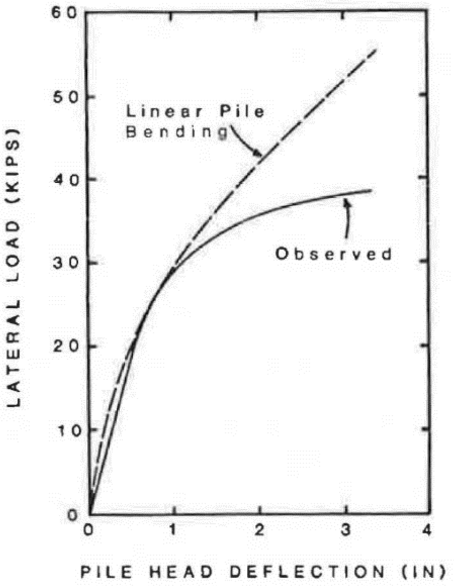

Figure 29: Comparison between linear model and observed behaviour (Kramer and Heavey, 1988)

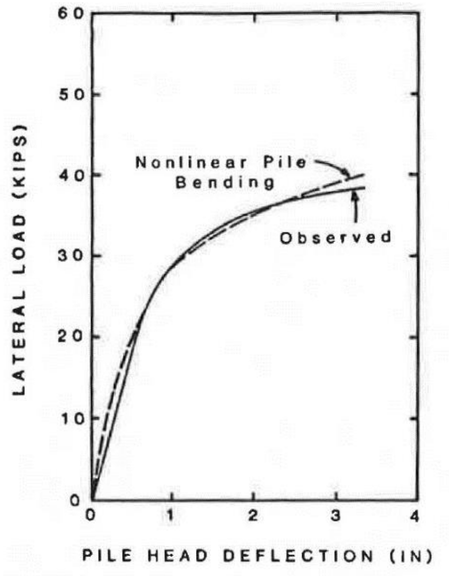

Figure 30: Comparison between non-linear model and observed behaviour (Kramer and Heavey, 1988)

A further example of the effects of using a non-linear pile model are shown in Figure 31 for a 0.76 m diameter bored pile in stiff clay (Ashour et al, 2001). Here, the load-deflection curve for a non-linear pile model is shown together with solutions for various values of the bending stiffness (EI) of an elastic pile. A would be expected, at low loads, the solution for the non-linear pile is the same as that for a linear pile with the same initial bending stiffness. However, as the load increases, there is a gradual transition from the initial curve for a linear pile across curves for decreasing values of EI.

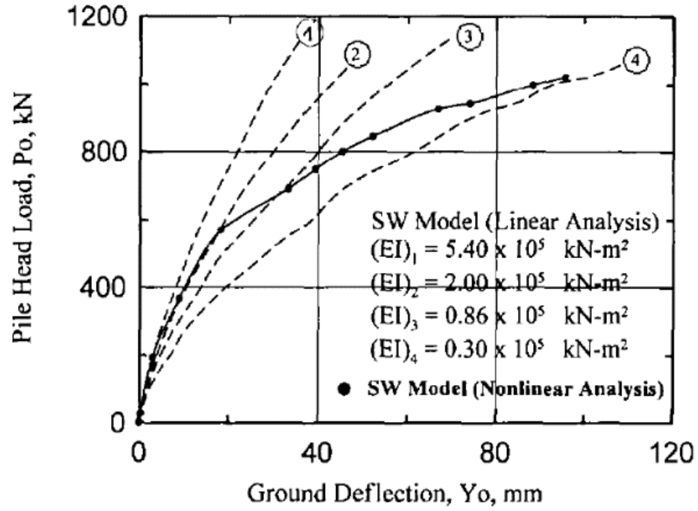

Figure 31: Load-deflection curves for non-linear pile model and for various EI values for a linear model (Ashour et al 2001)

9.1 IMPROVEMENTS IN PRACTICE

As a practical approximation, it may be feasible to use a series of computed load-deflection curves, for different pile stiffness values, to construct a non-linear curve, by starting the transition once the initial cracking moment of the pile section is reached, and then gradually moving across the curves for decreasing bending stiffness. From Figure 36, it may be expected that, for relatively low levels of lateral load, the use of a concrete modulus perhaps 1/3 to 1/2 of the small-strain value might provide an adequate result, but in any case, a measure of judgement is required to adopt an equivalent modulus to represent the non-linearity of the pile behaviour in bending.

For more detailed assessments of pile non-linear behaviour, commercial software packages are available, for example later versions of the programs LPILE and GROUP8 (Ensoft, 2015).

10 CONCLUSIONS

This paper has examined a number of aspects of design which are considered by the author to be inadequate, or else in need of improvement. These commonly-employed aspects have the potential to lead to inaccurate predictions of deep foundation behaviour.

The following factors are found to have the potential to lead to unconservative designs:

1. Ignoring foundation interactions;
2. Ignoring the effects of ground movements;
3. Ignoring the kinematic effects of ground movements in seismic design.

Conversely, the following factors may lead to conservative, and hence unnecessarily expensive, design outcomes:

1. Ignoring the beneficial effect of the raft;
2. Assuming a rigid cap or raft;
3. Over-simplification of the ground profile;
4. Ignoring the beneficial effects of basement walls;

With the design tools that are now available, all of the above perceived inadequacies can be addressed satisfactorily. There is however ample scope for innovation to be employed to develop effective and economical means of designing deep foundations and incorporating the key features of a particular project. The main challenges that remain in relation to foundation design are the recognition and modelling of the factors that can influence deep foundation behaviour, and the ever-present challenge of appropriate assessment of the relevant geotechnical parameters.

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