Some Factors That Influence the Prediction of the Behaviour of Piled Rafts via Simplified (Numerical) Analyses

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ABSTRACT: This article evaluates two distinct external effects on the settlement results from standard analyses of piled raft foundation systems. The influence of the excavation level and the influence of the number of piles underneath the pile will be separately assessed by two independent analyses for two published case histories, respectively a house located in Gothenburg and another in Uppsala, Sweden. They have been initially presented by Hansbo (1993) and Hansbo and Källström (1983). Both structures were founded over a soft, highly plastic marine clay of varying thickness, where the foundation was designed by using the concept of "creep piling", i.e., piles in a state of full load mobilization. The analyses were carried out with the numerical tools DEFPIG and GARP, by considering a series of simplified assumptions for the load pattern, raft and pile characteristics and subsoil profile. The soil, pile and load characteristics have been considered, with analyses that allowed (and not) the effect of the excavation level (1st case history), and with variation (optimization) of the number of piles (2nd. case history). The exercise emphasizes the importance of the consideration of the excavation level for the proper assessment of the settlement pattern underneath the raft, and the beneficial aspect on the optimization of the number of piles in the piled raft design.

KEYWORDS: Piled Raft, Soil Excavation, Numerical Analysis, Foundation Design, Soft Clay, Optimization

1. INTRODUCTION

In the past decade, several papers have been published with emphasis on what are now called as "piled-rafts", i.e., pile groups in which the raft connecting the pile heads positively contributes to the overall foundation behavior (for example Ottaviani, 1975; Poulos, 1991; Hansbo, 1993; Burland, 1995; Ta and Small, 1996; Clancy and Randolph, 1996; Mandolini and Viggiani, 1997; Poulos, 1998 and Cunha et al., 2001). The International Society for Soil Mechanics and Foundation Engineering (ISSMFE) also focused the activities of one of its Technical Committees (ITC-18) on the study of piled raft foundations. This Committee gathered valuable information on case histories and on methods of analysis, having produced comprehensive reports on these activities (O'Neill et al., 1996).

In regard to the design philosophy of piled rafts, Randolph (1994) has defined the following approaches:

- The "conventional approach", in which the foundation is designed essentially as a pile group to carry the major part of the load, while making some allowance for the contribution of the raft. This is the conventional approach widely adopted in design;
- The "creep piling approach", as proposed by Hansbo and Källström (1983), in which the piles are designed to operate at a working load at which significant creep starts to occur, typically 70-80% of the ultimate load capacity. In this case, the pile cap or raft, contributes to the overall capacity;
- The "differential settlement approach", in which the piles are located strategically in order to reduce the differential settlements, rather than to substantially reduce the overall average settlement. The pile cap, or raft, also contributes to the overall capacity.

In general, the latter two approaches are more economical than the first one, but they can only be used under certain conditions, where either local standard allows, or differential settlements are the key design factor. Other papers have expanded upon these ideas, such as those by Cunha and Sales (1998) and Cunha et al. (2000a, b). Cunha and Sales (1998) presented a paper describing and discussing field loading tests carried out in small scale footings supported by a reduced number of piles. These tests were performed at the University of Brasília research site, and have confirmed that this design methodology has a large potential (although with some restrictions) to be adopted with the collapsible porous clay of the Federal District of Brazil. Cunha et al. (2000b) analysed a piled raft case history in the city of Uppsala, Sweden, on a prediction "class A" exercise. They have suggested an "optimized" parametric procedure for the preliminary design of both piled rafts and standard deep foundations. This optimization has proved that it is possible to obtain a considerable cost saving in the final design, without detriment to the original factor of safety of the foundation. The suggested procedure has been tested against another case history in Sweden (Cunha et al. 2000a), allowing the perception of the influence (in design) of one of the relevant variables that affect the behaviour of the foundation system, i.e., the number of piles underneath the raft. These papers established the basis of the presented numerical results, although the critical discussion and analyses have been considerably extended herein with information not available at that time, expanding their original scope.

The present paper initially explores the design of piled rafts, outlining the influence of the consideration (or not) of the excavation level on the proper assessment of the settlement of the piled raft system, including the excavation process in a very simplified manner. This technique is not new, and has already been adopted before by some authors as Sales et al. (2010) or Ibañez et al. (2014), among others. Sales et al. (2010) allowed the influence of the variation of the stress level on the piled raft behaviour by a rather complex manner, introducing the excavation sequence (stepwise) process on the numerical analyses in what has been called as a "compensated" piled raft analysis. On the other hand, Ibañez et al. (2014) considered the effect of the excavation with a more simplified procedure (like the one to be adopted here), simply by correcting the effective original stresses of the ground to the relief stress/reloading caused by both the extracted soil during excavation and the concrete raft molding. Both cases are simplified, and do not lead to perfect simulations of the real phenomena, but they can considerably improve the settlement pattern predicted by the numerical simulations. Of course, they cannot be precise given several other external aspects that are difficult (if not impossible) to input within the analyses, such as the stiffening and load distribution effects caused by the subsoil-superstructure interaction, the real rheological behaviour of the soil upon unloading and reloading stages, the concrete placement and curing of the raft, the sequential (floor by floor) loading stages of the building, geotechnical variability of the subsoil and variable foundation geometries, concrete creep effects, and temperature effects among several others.

The paper also outlines a parametric analysis in which the number of piles is varied underneath the raft, allowing the understanding of an "optimized" procedure where safety and economy can be simultaneously achieved.

2. CASE HISTORIES

2.1 House in Gothenburg

Sweden is the biggest country in the Scandinavian region, covering an area of 450,000 km^2. The dominant characteristics of the landscape can be attributed to glacial activity, with the rocky south-west coast along the Baltic Sea, and the Stockholm archipelago on the south-east coast, which is most notable for their fjords (as stated in the Lonely Planet web-page). Gothenburg is the Sweden's second city, being situated on the country's west coast in between Copenhagen and Oslo.

In the early 80's, two quite similar houses, one founded on conventional friction piles and the other on a "creep-piled" raft foundation were constructed in this city. These buildings, defined as "House 1" and "House 2", were located just 20 m apart. House 1 was designed in full accordance with the Swedish Building Code, meaning that the total load of the building was assumed to be carried by the piles. These piles were designed with a safety factor of 3 against a short-term (undrained) failure. House 2, on the other hand, was designed with the "creep piling" approach, as proposed by Hansbo and Källström (1983). In this approach, the piles are loaded to values close to, or equal to their creep load. They have the main purpose of reducing the settlement of the overall foundation structure, since the load of the building is partially counterbalanced by the contact pressure at the soil / raft interface (Hansbo, 1993).

House 2 was chosen to be analysed herein. This house was designed to be an apartment house with 4 stories. It had a plan area of approximately 1000 m^2 with total dimensions of 75 versus 12 m, as schematically presented in Figure 1. It was constructed with four levels and a basement, leading to a total design load of 61.5 MN. The whole building was cast-in-situ, with basement walls uniformly spread over the base area. It was also designed to rest on a piled raft with a 0.4 m thick raft foundation, directly resting on top of the local marine clay, i.e., no clear mention by Hansbo (1993) is made regarding the fact that the foundation raft was buried or not in the site although a "basement" unit has been mentioned. Nevertheless, some indications in this original reference indicates that the 4 story apartment houses 1 and 2 were very similar and both had a "basement", as clearly described in the text. Besides, this reference, that indicates the ground beams for House 1, depicts what appears to be a basement space below the ground level. Hence, supported by indirect evidence, it is very probable that the foundation raft was indeed buried in the site, at least to one story level (~ 2.5 to 3 meters) – and this is what it was assumed here.

In this piled raft foundation, 104 piles were used. They consisted of 0.3 m in diameter and 18 m long spliced timber "underpiles" with 8 m long 0.3 m diameter circular concrete piles on top. The total length of the composite piles was 26m, being driven in place and uniformly spread over the building. They were placed mainly beneath the basement walls, as depicted by the filled circles of Figure 1. It also shows the instrumentation that was placed prior to the casting of the raft. Pile load cells, contact pressure cells, bellows-hose (benchmark) settlement gauges, pore pressure gauges, and leveling stations were installed – as indicated in Figure 1.

It should be pointed out that this case history was proposed by Van Impe (1999) to be one of the examples of an international exercise on the predicted behaviour of piled rafts, via numerical programs. During this event, the instrumentation data was not made available to the participants, in order to characterize a "Class A" predictive exercise, although it was already known at that time that the settlement results of this case history had been previously published by Hansbo (1993).

Indeed, as will be further detailed, some effort to truly perform a "Class A" prediction was made by the authors of the present contribution, by analysing this case history solely on the basis of the data provided for the prediction exercise, together with the geotechnical characteristics of the site. Hence, "Class A" analyses are also presented and discussed in the present exercise, allowing it to highlight the importance of some of the parameters used for the numerical simulation of piled rafts, in particular the excavation level.

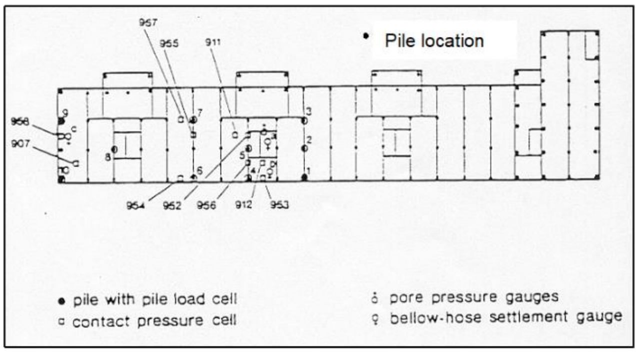

Figure 1 House 2 in Gothenburg (modified after Van Impe, 1999)

2.2 House in Uppsala

Uppsala is situated on the "Uppsala-slätten" plain, 70 km north of Stockholm. In 1984 and 1985 an apartment house was built in the northern part of the city in the town district of Svartbäcken. The house was placed in the middle of the site, and so the influence from adjacent buildings was negligible. It had a plan area of approximately 519 m^2 with dimensions of 38 x 14 m, as schematically presented in Figure 2. It was constructed with four levels and a basement, leading to a design load of around 29 MN. It was also designed to rest on a piled raft foundation, i.e., with piles under (or close to) full load capacity, according to the "creep piling approach" previously reported.

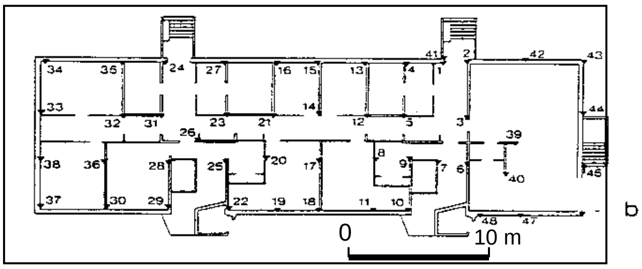

Figure 2 House in Uppsala (modified after Van Impe, 1999)

In the piled raft foundation, 48 piles were used. They consisted of 18 m long timber piles (spliced with concrete piles) of 0.3 m in diameter. The piles were uniformly spread over the building, and placed mainly along the basement walls, as depicted by the filled triangles of Figure 2. They were driven in January 1984, and the raft was cast in late August. Erection of the framework started in October and was completed in June 1985 (Van Impe, 1999).

Some instrumentation was placed prior to, and some after, the raft was cast in situ. Pile load cells, contact pressure cells, bellows-hose (benchmark) settlement gauges, pore pressure gauges, and leveling stations were installed. The bellows-hose settlement gauges are 30-35 m long, and consequently reached the firm strata below the clay. Pore pressure gauges were placed in the center of the building, at 5 m intervals, down to a depth of 35 m. Besides the detailed instrumentation, the authors of the present paper are unaware if such data has been published elsewhere.

3. GEOTECHNICAL CHARACTERISTICS

Geotechnical characteristics for both case histories are similar. The subsoil at both test sites consisted of soft, highly plastic marine clay of varying thickness. The clay was relatively homogeneous and contained two layers of silty clay, one at about 12 m and one just below 30 m, as graphically depicted in Figure 3. This clay layer extended down to a depth of 55m beneath the house at Gothenburg, and was underlain by rock. In Uppsala it extended to 34 m overlying a sand layer.

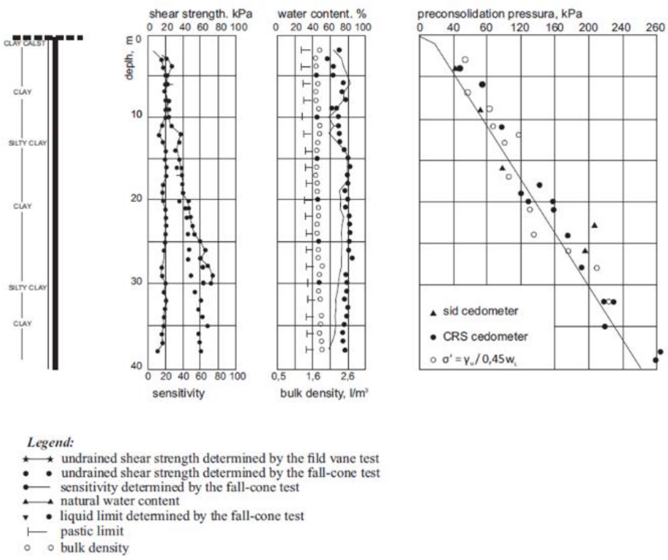

Figure 3 Main geotechnical parameters of the subsoil at Gothenburg (modified after Van Impe, 1999)

The undrained shear strength (Su) is fairly constant along the profile, down to 10 m depth, with a mean value of about 20 kPa. It then increases linearly with depth, at an approximate rate of 2 kPa/m down to around 40 m depth. The sensitivity is quite constant, being slightly less than 20. The natural water content varies from 60 to 80 %, and the liquid limit is usually somewhat higher. The plasticity index is typically about 50 %, and the bulk density or total unit weight is around 16.5 kN/m^3. The clay is slightly overconsolidated, given the fact that standard and CRS oedometer laboratory tests yielded preconsolidation pressures just above the values of vertical effective stress in the soil layer.

4. NUMERICAL TOOLS AND ASSUMPTIONS

The program GARP (Geotechnical Analysis of Raft with Piles, Poulos and Small, 1998) was adopted to evaluate the behaviour of both the rectangular piled raft foundations at Gothenburg and Uppsala, subjected to a distributed vertical loading. It is based on a simplified form of a hybrid program in which the raft is represented as a linear elastic plate (via finite elements) and the soil can be modelled either as an elastic layered continuum or as a "winkler" spring medium. The piles are represented by elastic-plastic springs that can interact with each other and with the raft. Limiting values of contact pressure (beneath the raft) and pile capacity can also be specified. By analysing the raft using the finite element technique, rather than via finite differences, it is possible to numerically simulate irregular shaped rafts, which can also be subjected to uniform or concentrated loads.

As mentioned above, GARP also considers "interaction factors" between the springs that represent the piles. Such factors are computed via the use of another well-established software program DEFPIG (Deformation Analysis of Pile Groups, Poulos, 1990). This latter program determines the deformations and load distribution within a group of piles subjected to general loading. It was specifically written for piles designed using the "standard approach", by considering a group of identical elastic piles having axial and lateral stiffness that are constant with depth. It also allows for the eventual slippage between the piles and the surrounding soil. The stress distributions are computed from the theory of elasticity, more specifically from Mindlin's solutions for an isotropic, homogeneous, linear elastic medium. It can also consider, although in a simplified manner, the soil non-homogeneity along the length of the pile (i.e., variation of the soil modulus with depth).

Both programs were used by adopting several simplified assumptions regarding the pile, raft and soil characteristics. These assumptions were necessary due to the simplified way in which these analyses were done, and also due to the lack of detailed information on both particular case histories, as previously mentioned.

The following assumptions were made in each case:

- Soil Profile in Gothenburg: 55 m of soft to medium clay (average Su ≈ 30 kPa varying from 20 to 60), overlying a rock surface. GARP took into consideration the soil parameters (Young's Modulus and Poisson's ratio) down to this lower limit of depth, where an extremely rigid (rock) surface was adopted. A constant drained Young's Modulus of 8200 kPa (lower limit) and 15000 kPa (upper limit) respectively were considered in the parametric analyses with the clay profile, together with a variable drained modulus (E′s) increasing from 6600 to 11800 kPa). These values are applicable to driven piles in clay, and were estimated by adopting the correlation between shear strength and modulus expressed in Poulos and Davis (1980). A drained Young's Modulus of 5000 MPa was adopted for the rock surface. All DEFPIG and GARP analyses considered an average clay Poisson's ratio of 0.4. The water level was initially assumed to be at the ground surface, being lowered to an average level of 1.5 m below ground surface to take into consideration excavation effects (to be described below). This was based on possible range of values adopted during excavation and dewatering at this site – the real values are unknown to the authors;
- Soil Profile in Uppsala: 34 m of soft to medium clay (su ≈ 30 kPa), overlying an assumed 16 m thickness sand layer. This latter layer overlies a rock surface; The soil deformations were limited to an extent approximately equal to the "bulb of pressure" (where the vertical pressure change $\Delta Pv/Pv = 10\%$) underneath the piled raft. This bulb was calculated via simplified "equivalent raft" analyses. This simplified analysis allowed a load spread of 1:4 and placed the raft at a distance (below surface) of 2/3 of the pile's length. GARP took into consideration the soil parameters (Young's Modulus and Poisson's ratio) down to the lower limit of depth, where an extremely rigid (rock) surface was adopted. A constant drained Young's Modulus of 9000 kPa was considered for the clay profile. This value is for driven piles in clay, and was estimated by adopting the correlation expressed in Poulos and Davis (1980). A drained Young Modulus of 50000 kPa was adopted for the sand profile. All DEFPIG and GARP analyses considered an average clay Poisson's ratio of 0.4 and sand Poisson's ratio of 0.3. The water level was assumed to be at the ground surface, yielding hydrostatic values within the soil;
- Pile Characteristics and Location: At Gothenburg the elastic modulus of the pile was considered as constant during GARP analyses, being obtained via DEFPIG analysis with the assumed soil profile and Young's moduli, and with the given pile characteristics. The 104 composite floating piles were considered to be vertical (and uniform) with a constant diameter of 0.3 m, and length of 26 m. They were also considered to be mainly of timber, with an assumed Young's Modulus of 18000 MPa and Poisson's ratio of 0.2. In all GARP analyses the piles were assumed to apply a uniform pressure to square elements of similar area (to the pile section) in the raft. In the case of Uppsala similar parameters/procedures were used, and

the 48 piles were considered to be vertical with the same shaft and base diameter of 0.3 m, and length of 18 m;

- Raft Characteristics and Location: The Young's Modulus of the raft was assumed to be 25000 MPa, its Poisson's ratio to be 0.2, and its thickness to be 0.4 m (Gothenburg) and 0.3 m (Uppsala). The base of the raft was assumed to be at the top level of the piles, with full contact with the underlying soil. An approximate dimension of 75 x 12 m (area ≈ 1000 m^2) was adopted in the analyses for Gothenburg, with uniform pressures of 61.5, 35 and 30 kPa (parametric analyses) evenly distributed around the top surface of the raft. In the case of Uppsala an approximate dimension of 38 x 14 m was adopted in the analyses, with a uniform pressure of 56 kPa evenly distributed around the top surface area of the raft;
- Bearing Capacity of Pile and Raft: The long-term bearing capacity of the piles was estimated and used, since the final, total settlement was desired. The point bearing capacity was calculated via a traditional effective stress approach. A drained friction angle of 25° was assumed for the clay in Gothenburg and 28° in Uppsala. The shaft resistance was calculated via the "Beta" method for drained soils, using the same angle of 25° and assuming a coefficient of lateral pressure K0 of 0.8 (OCR > 1) in Gothenburg and 0.65 in Uppsala. These are typical values found in the literature for marine clays (assuming that variations can possibly happen). The total depth of the piles was taken into consideration for the calculation of the vertical stress levels, assuming a bulk unit weight of 16.5 kN/m^3 for the soil layer. The group efficiency was estimated via the Poulos and Davis (1980) equation, which considers the sum of the ultimate capacities of individual piles and the ultimate load capacity of the "block" containing piles and soil. An efficiency very close to 1.0 was calculated, and a unit value was therefore assumed. A drained bearing capacity was also calculated for the raft, adopting the Terzaghi (1943) equation again using aforementioned friction angles for the clay;
- Interaction Factors: These were obtained via the DEFPIG analysis, and used within GARP for pile/pile and pile/raft settlement interactions – given the lack of pile load tests on the site (none was found or apparently published). These interactions were limited to a horizontal spacing equal to the total length of the pile, i.e. 26 m and were assumed to be zero for greater spacings. The raft/raft and raft/pile interactions were obtained via the Boussinesq elastic equations, assuming an elastic continuum model and making approximate allowance for soil layering (E′ variable);
- Pile and Raft Discretization: In the case of Gothenburg, a non-symmetrical mesh with 940 nodes and 855 elements was assumed for the raft in the GARP analyses, as depicted in Figure 4. The piles were introduced in the nodes (crossings) of this same figure, following the real disposition depicted in Figure 1. In the DEFPIG analysis each pile was divided into 52 elements each 0.5 m in length. In the case of Uppsala the mesh had 760 nodes and 702 elements, and each pile was divided into 36 elements with 0.5 m in length;
- Parametric Analysis of Gothenburg: A drained Young's Modulus of 8200 kPa and 15000 kPa was adopted for the clay. A calculated raft pressure of 61.5 kPa was adopted for the long-term settlement analysis as presented in Table 1 – Cases 1 and 4. These cases were followed by another analysis with a variable Young Modulus, keeping the same raft pressure – Case 6. Six cases were then analysed according to this table. The soil Young's Modulus was considered as constant (8200 and 15000 kPa – cases 1 to 5) with depth or variable (see table – case 6). The pressure on top of the raft was considered in accordance to the published total load of the house (61.5 MN) and raft area of around 997 m2 in accordance to cases 1, 4 and 6. The final net effective pressures of 30 and 35 kPa, adopted in cases 2, 3 and 5 were calculated by considering the excavation process of the subsoil;

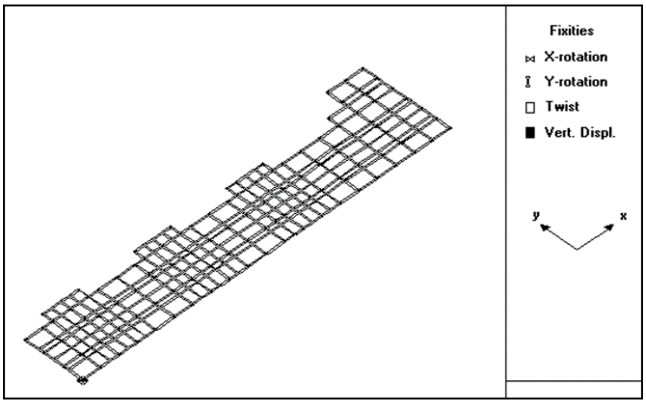

Figure 4 Discretization of the finite element mesh of the foundation in Gothenburg (raft and piles)

Table 1 Parametric analyses adopted in Gothenburg´s house

Case	E′s (kPa)	ΔP raft (kPa)	Raft Position
1	8200	61.5	Surface
2	8200	35	Buried
3	8200	30	Buried
4	15000	61.5	Surface
5	15000	30	Buried
6	Variable*	61.5	Surface

- Parametric Analysis of Uppsala: In the parametric analysis, 3 cases were studied, i.e., the original case with 48 piles (Case 1), a reduction of the number of piles from 48 to 24 (Case 2), and from 24 to 12 (Case 3). Case 2 involved 24 piles distributed along the 2 central "rows" of the raft, while Case 3 involved 12 piles centrally located within the raft (6 outer piles were simply removed from each of the sides of the raft adopted in Case 2);
- Excavation sequence at Gothenburg: By analysing the assessed information of the original Hansbo (1993) paper, it appears that the raft was not cast on the deposit´s surface, but rather buried on the subsoil. It was then estimated to have been constructed within an excavation of 2.5 m with the final water table at 1.5 m below ground surface. A simplified approach (hand calculation) was carried out to evaluate the net final effective pressure at raft level (as stated before, around 30 to 35 kPa). This approach adopted the same qualitative soil behaviour as numerically found by Hsi and Small (1992) when simulating a 1-D excavation in a poro-elastic material. It was considered that the raft load was applied immediately after the soil excavation, i.e., before any pore pressure change from negative to positive values – which is obviously another simplified assumption. At Uppsala the raft was considered at the surface, in a simple manner.

5. RESULTS AND DISCUSSIONS

5.1 Effect of the Excavation Level (Gothenburg Case)

5.1.1 Overall results

The results of the numerical analyses were compared in terms of the extreme (maximum and minimum) values of settlement and moment (in both x and y directions). The load sharing between the piles and the raft was also computed and compared. These results are presented in Table 2, while Figures 5 to 7 depict the contours of vertical settlement respectively obtained for Cases 1, 4 and 2. The discussion on the results come in an itemized manner after that.

Table 2 Settlement, moment and load sharing results at Gothenburg

Case	Settlement (cm)		Moments (kN.m)				Load Sharing (%)	
			Mx		My		Piles	Raft
	max	min	max	min	max	min		
1	8.3	3.0	128	-237	208	-113	75	25
2	4.7	1.7	73	-133	117	-63	75	25
3	4.1	1.5	62	-114	100	-54	75	25
4	5.3	1.9	100	-197	163	-107	66	34
5	2.6	0.9	49	-96	80	-52	66	34
6	7.4	2.6	129	-243	202	-118	77	23

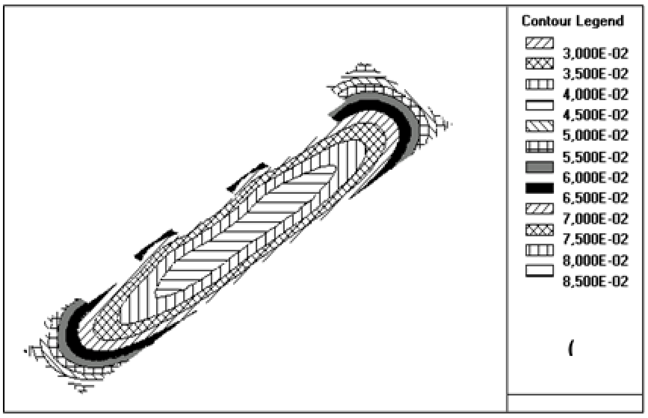

Figure 5 Contours of vertical settlement (m) – Case 1 – surface raft

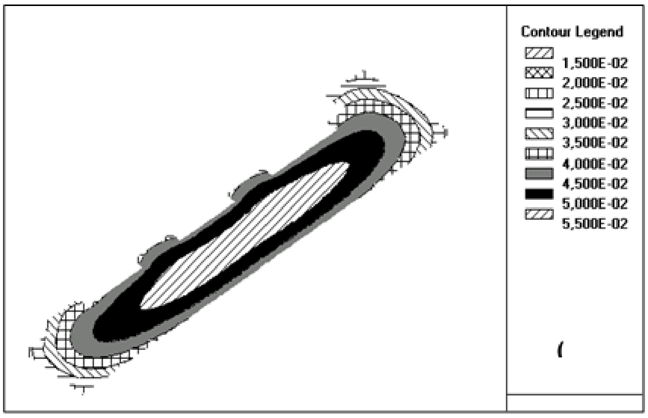

Figure 6 Contours of vertical settlement (m) – Case 4 – surface raft

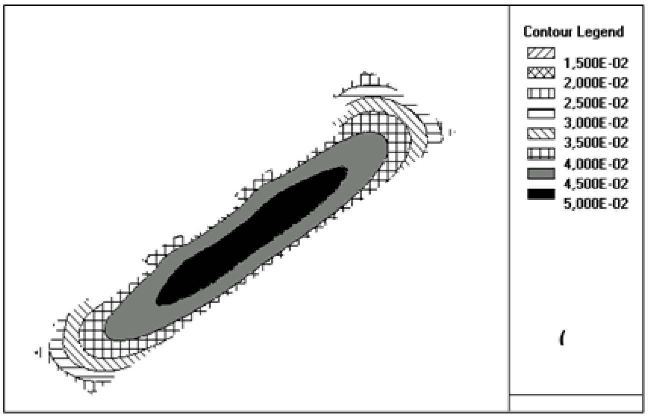

Figure 7 Contours of vertical settlement (m) – Case 2 – surface raft

The contour legend represents the limits (in meters) for the obtained settlement results. For instance, in Figure 7 the black (central) region represents the area of the raft with derived settlements between 4.5 and 5 x 10-2 m, i.e., 4.5 to 5 cm.

The comparison in terms of absolute extreme values is useful to indicate general behavioural tendencies from the different input parameters adopted with this particular piled raft foundation.

Hence, some general observations can be drawn from the results of Table 2 and from Figures 5 to 7:

- By increasing the Young's Modulus of the soil, while keeping all other variables constant, there is a tendency for both (max./min.) settlements and moments to decrease, as expected. There is also a slight tendency for the load carried by the raft to increase, reducing the load carried by the piles;
- For a variable E′s rather than a constant E′s, there is a tendency for the settlement values to be intermediate between those obtained with a constant E′s. Nevertheless, similar results for moments and load division were obtained for both Cases 1 and 6;
- By decreasing the distributed pressure on top of the raft, while keeping all other variables constant, there is a tendency for both (max./min.) settlements and moments to decrease, again as expected. However, there was no variation in the load sharing between raft and piles.

It is also noticed that similar contours of total settlement were obtained in Figures 5 to 7 for each of the cases analysed, although the magnitude of the settlements varied from one case to another. This indicates that all cases tended to develop similar pattern of settlements, albeit distinct values of input E´s, raft position (buried or not) and raft pressure. Indeed, given the homogeneity of the subsoil, and linearity of loads and responses (structure and soil´s modulus), such similarities were already anticipated.

In summary, the analyses demonstrated the important influence of the assumed drained Young's Modulus of the clay, and the distributed load, on the predicted values of settlement and moment. The Young's Modulus has also some influence on the total load sharing between raft and piles, but to a lesser extent than the influence on the settlement.

5.1.2 Class A Analyses and the Assessment of the Excavation

It was mentioned before that, despite the fact that this particular case history has already been published elsewhere (and the final total settlements are supposedly known), some effort has been made to characterize the numerical analyses as truly "Class A" predictions. That means, to check the predictions against the (unknown during analysis) measured values. Hence, Cases 1, 4 and 6 (initial numerical analysis with the raft at surface) were analysed solely on the basis of the information provided by Van Impe (1999) in the international exercise, without referring to the Hansbo (1993) or Hansbo and Källström (1983) papers.

The predicted settlements, presented in Table 2, varied from about 5 to 8 cm (maximum) and about 2 to 3 cm (minimum). In fact, as noted before, these settlements were strongly related to the assumed drained modulus of the clay, and the other assumptions in terms of raft pressure and position adopted for the input parameters. By comparing the predicted results with those from Hansbo (1993), depicted in Figure 8, it was noticed that they somewhat different from the measured "real" published settlements, which ranged from 2.5 (min.) to 4.2 cm (max.). That means, the experimental values were well below the predicted values at class A prediction from Table 2, indicating that either the model did not accomplish the nuances of the rheological phenomena or the input elastic parameters were slightly lower ("softer") than those from the real subsoil (or something else, as the actual stiffening effect of the basement beams on the overall displacement pattern).

In order to understand the possible reason for such discrepancies, the authors carefully reviewed the Hansbo (1993) paper. This extra exercise revealed that an excavation was probably carried out before the construction of the piled raft (as suspected by aforementioned comments). This excavation was not considered in the initial numerical analyses with Cases 1, 4 and 6, given the lack of detailed information about this particular case history. Indeed, the excavation process prior to the raft placement may have a large effect on the final settlement results, since it considerably changes the original stress state (see for instance Hsi and Small, 1992). According to Sales et al. (2010), the stress relief and preloading process that takes place during the excavation and casting of the foundation raft must be taken on account to properly simulate the loading pattern, and hence to accurately forecast the settlements at working load.

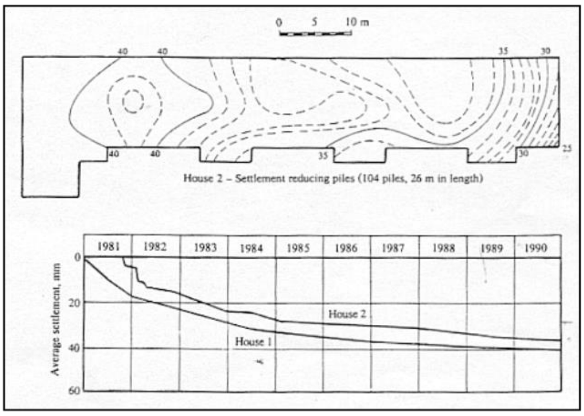

Figure 8 Measured settlement contours in House 2 of Gothenburg (modified after Hansbo, 1993)

In order to account for this effect, at least in a reasonable but simplified manner (as proposed by Ibáñez et al., 2014), extra parametric analyses were made considering the excavation. The new assumption resulted in the final net effective pressures of 30 and 35 kPa below the raft, as adopted for Cases 2, 3 and 5 in which the raft was considered to be buried. Off course, in these particular cases, it is readily noticed that they consisted of "Class C" predictions for this particular exercise (as the authors already knew the "target" values), rather than "Class A" predictions as done before.

The new effective pressures were estimated by simple hand calculations where the effective vertical pressures below the raft were determined with the excavation effect on the groundwater pressure. That means, immediately after 2.5 m (1D) excavation the positive water pressure (with water table at ground surface) became negative in accordance to Hsi and Small (1992) advocated technique. Once the raft load is applied, by casting of the foundation, the water pressure changes again to a positive value.

At this stage, it was assumed that two things took place simultaneously: The lowering of the water table from ground surface to 1.5 m below ground (dropped 1 m in height) and full dissipation of the excess pore pressures (positive excess in the previous step) to take on account long term effects. The final variation of net effective vertical pressures is simply the difference between initial and final values below the raft. Perhaps this procedure can be better visualized with the values of Table 3.

Within the context of the analyses carried out for Cases 2, 3 and 5, a reduction in the maximum settlement as high as 50 % can be noticed in Table 2. It is also noticed that a variation on the effective Young's Modulus was also tried out from cases 2, 3 to 5, in order to improve even more the predictions (again, a typical "Class C" analysis since the measured results were known in advance allowing the optimization of the numerical output). The important point to note, however, relates to the fact that even by knowing beforehand the results, the present authors were unable to accurately and definitively predict the settlement pattern presented by Hansbo (1993), as depicted in Figure 8. See, for instance, the marked differences in the pattern of vertical settlement from Figures 7 to 8.

Table 3 Variation of pore, effective and total vertical pressures underneath the raft

Condition	Pore Pressure (kPa)	Effective Vertical Pressure (kPa)	Total (*) Vertical Pressure (kPa)
Initial state before excavation	25	16.25	41.25
Immediately after 2.5 m excavation. GWT at surface	-16.25 (**)	16.25	0
Application of raft load - molding	--	--	61.5
After curing. GWT at surface	45.25	16.25	61.5
GWT at 1.5m below surface and full consolidation	10	51.5	61.5
ΔEffective Net Vertical Pressure =	51.5 - 16.25 = ~35 kPa		

* Considering a total unit weight of around 16.5 kN/m3
**Considering Hsi and Small (1992) approach

Although Figure 7 presents settlement values close to the right magnitude, they were predicted to vary in a concentric "dishing" manner within the raft, which differs from the measured (real) pattern depicted in Figure 8. This figure portrays higher settlements at the left side of the house, which could be due to some possible factors, as follows: uneven distribution of raft pressures (due to concentrated loading from existing columns), a variable clay layer profile and/or the stiffening effect of the foundation beams (in what is called as a "soil-superstructure effect"). Neither conditions were considered herein, given the lack of detailed information on these parameters, as well as software capabilities.

5.1.3 Main Points Learned from the Analyses

The main observation of the Gothenburg series of analyses is that the problem lacked a great deal of important information, even for a proper "Class C" prediction (see previous comment related to cases 2, 3 and 5). Besides the shortcomings of these analyses, the present exercise was valuable in showing possible deficiencies in standard numerical design analyses, and the importance that the excavation effect, among others, has in fine tuning the settlement results. It has shown that the detailed knowledge of some input variables is essential for the proper numerical simulation of real engineering projects, and, besides all, that some extra external influential variables (nowadays considered as circumstantial aspects of the problem) should be taken on account in future numerical predictions of piled rafts.

Indeed, in 2016 for the Brazilian national geotechnical conference (XVIII COBRAMSEG) colleagues from the geotechnical and structural area have agreed in a special session of the event (soil-superstructure effects) that future analyses must deal somehow with this aspect (as done in a simplified manner by Cunha and Cámbar, 2011), either in terms of the building structural modelling or in terms of the foundation discretization. This opinion is also shared by Russo et al (2013) when reassessing the foundation settlements for the Burj Khalifa foundation project.

Unfortunately, the designer is still far from reaching an agreement on how to do that in practice, and with what tools (and parameters). As a final piece of information given in this same conference, some "educated" accounts given by colleagues on the budgetary aspects of the design foresee an increase of 0.04% on the final price of the elaboration of the geotechnical design, i.e., this component of the project would increase from around 0.20% of the total budget of the building construction to around 0.24%. Is the price increase worthy of the benefits of a better understanding of the whole superstructure – foundation system (hence more accurate analyses)? The future will hopefully provide answer.

5.2 Optimization of the Number of Piles (Uppsala Case)

The comparison in terms of absolute extreme values of the output variables for the Uppsala case history, where the raft was not considered buried, is useful to indicate general behavioral tendencies from the different designs adopted with this particular piled raft foundation. Hence, Table 4 presents the final predictive results for all cases studied herein. The following observations can be drawn:

- By reducing the number of piles there is a tendency to increase the load carried by the raft. This increase was in the order of 75 %, and it caused an increase of 113 % in the average contact pressure underneath the raft. However, the predicted average pressures are well below the long-term bearing capacity of the soil underneath the raft, estimated at 480 kPa;
- By reducing the number of piles there is a slight tendency for the global factor of safety (against rupture) of this foundation to decrease. Nevertheless, the estimated values are well beyond 3, clearly indicating that the purpose of inclusion of the piles was to reduce total settlements, rather than to increase the capacity of the foundation system;
- By reducing the number of piles there is a slight tendency for the factor of safety of the piles (alone) to decrease. A decrease of 20 % was noted when the number of piles was reduced from 48 to 12, and this was caused by the fact that the number of piles at the limit of their capacity (estimated to be 380 kN from a drained analysis) has considerably increased from Cases 1 to 3. For instance, with 48 piles (Case 1) 48 % of the piles were fully mobilized, while 100 % of the piles were in this condition by adopting 12 piles (Case 3);
- By reducing the number of piles there is a marginal increase on the maximum and minimum settlements of the raft.

Table 4 Predicted results from the parametric analyses at Uppsala

Max. Δ (cm)	**No. of Pile**	**Load Division (%)**		**Avg. Contact Pressure (kPa)**	**Factor of Safety**	
		Piles	Raft		Global	Piles
6.8	48	51.6	48.4	27.7	8.9	1.2
6.9	24	30.1	69.9	51.0	8.6	1.1
7.2	12	15.3	84.7	59.1	8.5	1.0

5.2.1 Main Points Learned from the Analyses

The main observation from the Uppsala series of analyses is that the piled raft system would not fail by decreasing the number of piles from 48 to 12, since the global factor of safety would still stay well beyond 3. All the piles would be fully mobilized, and the system would have a marginal increase in the maximum settlement of 5 % with respect to the value obtained in the original design. Moreover, a great cost saving would be obtained by optimizing the design in such manner, since 75 % of the piles from the original piled raft configuration would be removed.

Such results do highlight the fact that an “optimized” parametric procedure should be considered in practice for the design of both piled rafts and standard deep foundations, so that economy and safety can be simultaneously taken on account.

6. CONCLUSIONS

The results of the present exercises highlight the fact that in practice it is extremely important to understand all input parameters which are required in the design of both piled rafts and standard (conventional) groups of piles. This is so regardless of the “accuracy” and capabilities of the numerical program in question.

This contribution has shown that it is not possible to precisely predict the behaviour of piled rafts and group of piles, in both “Class A and C” analyses, without a full understanding of the problem and knowledge of its important input parameters, such as the raft geometry and load distribution (magnitude, pattern, variability with time), the excavation depth and sequence, the seasonal variability of the water level, the soil profile (depth, variability, layering), and, finally, without a comprehensive laboratory or in situ testing program. It has also allowed a clear perception of the possible influence of the lack of information of some of the above variables in the final numerical assessment of a published piled raft case history, particularly the excavation process (simulated in a simplified manner) and the soil-superstructure stiffening influence (not taken on account).

It is also important to provide alternative analyses, in order to rationalize the project, enhancing its economy with lesser use of piles, nevertheless keeping the necessary safety factors.

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