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The role of cap flexibility in pile group design

La flexibilité du radier dans la conception des groupes de pieux

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ABSTRACT: Pile groups may be connected through a cap which has a relatively small rigidity. In that case, the assumption of fully rigid cap adopted by some pile designers is incorrect and can lead to an underestimate of pile deformations and an overestimate of forces and moments in the outer piles of the group, especially for larger groups. This may partially be due to a lack of readily available numerical codes, given that commercial software for pile group design is mainly based on the rigid cap assumption (e.g. Repute, PIGLET, DEFPIG, GROUP). In an attempt to overcome this limitation, this paper extends the Repute analysis to the fully-flexible cap (i.e. individual piles not connected by cap), a feature which is particularly useful in the design of large pile groups. Validity of the approach is assessed through comparison with alternative numerical analyses and two case histories. In addition, a database of real cases is collated showing the relationship between the raft geometry and rigidity. Results provide guidance on the field of applicability of the rigid and flexible cap assumptions, indicating that most large rafts are likely to belong to the perfectly flexible category.

RÉSUMÉ: Certains les groupes de pieux ont une radier avec une petite rigidité. Dans ce cas, l'hypothèse d'une radier parfaitement rigide adoptée par certains concepteurs de pieux est incorrecte et peut entraîner une sous-estimation des déformations des pieux et une surestimation des forces et des moments dans les pieux extérieurs du groupe, en particulier pour les groupes de pieux plus grands. Cet article étend l'analyse du logiciel Repute à l'hypothèse de radier parfaitement flexible, caractéristique particulièrement utile dans l'analyse de grands groupes de pieux. Le validité de l'analyse proposée est démontrée par comparaison avec d'autres analyses numériques et deux cas d'expérience. En outre, une base de données est rassemblée, montrant la relation entre la géométrie et la rigidité du radier. Les résultats fournissent des indications sur le domaine d'applicabilité des hypothèses de radiers rigides et flexibles, indiquant que la plupart des grands radiers sont susceptibles d'appartenir à la catégorie des radiers parfaitement flexibles.

Keywords: Piles; Pile groups; Pile cap; Raft; Repute

1 INTRODUCTION

A crucial aspect in the design of pile groups is the proper assessment of the cap (or raft) stiffness. For pile groups of small to medium size (say up to 25 piles), the assumption of rigid cap is generally reasonable for practical purposes. However, as the pile-group size increases, the cap stiffness is reduced and the following aspects need to be considered:

- increase of differential settlements and angular distortion across the pile group;
- more uniform load and moment distribution among the group piles.

In current design practice, however, the above issue is not always correctly addressed and the assumption of fully-rigid cap is rather indiscriminately adopted. This may partially be due to a lack of readily available numerical

codes, given that commercial software for pile group design is mainly restricted to the rigid cap assumption, such as the Winkler-based approach adopted by GROUP (Reese et al. 2016), and the boundary element analyses used by Repute (Bond & Basile 2018), PIGLET (Randolph 2004), and DEFPIG (Poulos 1990). More sophisticated numerical analyses are also available, either based on the finite element (e.g. Plaxis 3D) or the finite difference (e.g. Flac-3D) methods. However, due to their complexity and high running costs (both in terms of computational time and efforts in data handling), these programs are not readily applicable to routine pile-group design and their use is confined to special cases (e.g. large piled rafts).

In an attempt to overcome the above limitation, this paper extends the Repute analysis (currently restricted to fully-rigid caps) to the case of a fully-flexible cap, i.e. individual piles not connected by cap. The main feature of the proposed method lies in its capability to provide a complete 3D boundary element (BEM) solution of the soil continuum, while incurring negligible computational costs. Accuracy of the proposed analysis is assessed through comparison with alternative numerical solutions and two case histories. In addition, in order to show the relationship between the raft geometry and rigidity, a number of well-documented real cases is examined.

2 PGROUPN ANALYSIS

Repute pile-group analysis is based on the 3D BEM formulation implemented in the computer program PGROUPN (Basile 2003). The analysis takes into account the simultaneous influence between all elements of all piles in the group, i.e. a "complete" solution of the soil continuum is provided. Pile-group effects are therefore evaluated as a matter of course, thereby overcoming the approximations of traditional interaction factor approaches and the fundamental limitations of Winkler models (based on empirical multipliers to account for group action). In addition, by retaining soil continuity, the input soil parameters have a clear physical meaning (e.g. the soil Young's modulus and strength properties) and can be measured directly in soils investigation. The basic influence of soil nonlinearity is modelled using a hyperbolic continuum-based interface model, while completely general 3D loading conditions (axial, lateral, and moments) on each pile of the group can be examined.

A detailed description of the theoretical formulation adopted in PGROUPN for the case of pile groups connected by a fully-rigid free-standing cap has been presented elsewhere (e.g. Basile 2003). The extension to the fully-flexible cap (i.e. individual piles not connected by cap) represents a special case of the above formulation and, hence, only a brief outline is given below. The analysis involves discretization of only the pile-soil interface into a number of cylindrical boundary elements, while the base is represented by a circular (disc) element, as illustrated in Figure 1. The behaviour of each element is considered at a node with uniform stress distribution assumed over the element. The method employs a sub-structuring technique in which the soil and pile domains are initially considered separately and then coupled by imposing the compatibility and equilibrium conditions at the pile-soil interface.

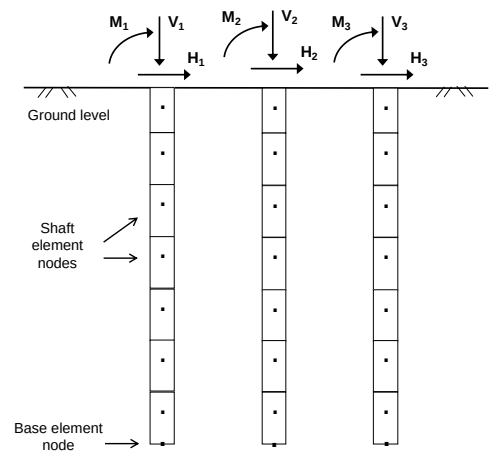

Figure 1. BEM schematisation of the problem

2.1 Soil domain

Under the assumption of purely linear elastic soil behaviour, soil stress (t_s) and displacements (u_s) at the pile-soil interface can be related via:

$$\{u_s\} = [G_s]\{t_s\} \tag{1}$$

where G_s is the soil flexibility matrix obtained from the solution of Mindlin (1936). It is noted that Mindlin's solution is rigorously applicable to homogeneous soil conditions; however, in practice, this limitation is not strictly adhered to, and the influence of soil non-homogeneity is often approximated using a weighted average value of soil modulus at the influencing and influenced pile nodes.

2.2 Pile domain

If the piles are assumed to act as simple (elastic) beam-columns which are free to rotate at their heads, the pile displacements may be written as:

$$\{u_p\} = [G_P]\{t_p\} + \{B\} \tag{2}$$

where u_p are the pile displacements, t_p are the pile stresses, B are the pile displacements due to unit boundary displacements and rotations of the piles, and G_p is a matrix of coefficients obtained from the elementary (Bernoulli-Euler) beam theory.

2.3 Non-linear soil behaviour

The foregoing procedure is based on the assumption that the soil behaviour is linear elastic. However, it is essential to ensure that the stress state at the pile-soil interface does not violate the yield criteria. This can be achieved by specifying the limiting stresses at the pile-soil interface, for example using the classical equations for the ultimate shaft and base resistance (e.g. Basile 2003). Non-linear soil behaviour is modelled by assuming that the tangent soil Young's modulus (E_{tan}) varies with the pile-soil interface stress (t) according to the common hyperbolic stress-strain law:

$$E_{tan} = E_i(1 - R_f t/t_{lim})^2 \tag{3}$$

where E_i is the initial soil modulus, R_f is the hyperbolic curve-fitting constant, and t_{lim} is the limiting value of pile-soil stress. Thus, the soil and pile equations described above for the linear response are solved incrementally using the modified values of soil Young's modulus of Equ. 3 within the soil matrix [G_s], while enforcing the conditions of yield, equilibrium ($t_s = -t_p$) and compatibility ($u_s = u_p$) at the pile-soil interface. For the pile-soil interface elements which have yielded, no more increment in stress is permitted and any increase in load is therefore redistributed between the remaining elastic elements until all elements have failed.

3 EFFECTS OF CAP FLEXIBILITY

In practical applications, a simple check on the assumption of cap flexibility may be performed by calculating the raft-soil stiffness ratio (K_{rs}) as defined by Horikoshi & Randolph (1997):

$$K_{rs} = 5.57\,\frac{E_r}{E_s}\frac{1-\nu_s^2}{1-\nu_r^2}\left(\frac{B_r}{L_r}\right)^{0.5}\left(\frac{t_r}{L_r}\right)^3 \tag{4}$$

where the subscripts r and s denote the raft and soil properties, respectively, E is the Young's modulus, ν is the Poisson's ratio, B_r is the raft width, L_r is the raft length (with $B_r \leq L_r$), and t_r is the raft thickness. In the case of layered soil profiles, E_s is the representative soil Young's modulus beneath the raft and may approximately be taken as the weighted average value over a representative depth equal to the raft equivalent radius (Poulos 2017), i.e.:

$$E_s = \frac{\sum_{i=1}^{n} h_i E_{si}}{\sum_{i=1}^{n} h_i} = \frac{\sum_{i=1}^{n} h_i E_{si}}{\sqrt{B_r L_r / \pi}} \qquad (5)$$

where h_i is the thickness of layer i, E_{si} is the soil Young's modulus of layer i, n is the number of soil layers within the representative depth, and $\sqrt{B_r L_r / \pi}$ is the raft equivalent radius. In the selection of the E_s value to be used in Equ. (4), it is also important to account for the appropriate strain level applied to the soil by the foundation. As an indication, following O'Brien (2012), a secant value of E_s equal to 70% of the small strain value may generally be adopted for pile groups designed with a typical factor of safety of about two.

Based on the work of Poulos (2016), O'Brien et al. (2012), and Horikoshi & Randolph (1997), the following limits for K_{rs} values resulting from Equ. (4) are recommended, as illustrated in Figure 2:

- For K_{rs} < 0.01, the raft behaves as "fully flexible", i.e. piles can be considered as not connected by cap (and hence interacting with each other only through the soil). In this case, the raft is unable to redistribute external load between the piles and hence the loads acting on each pile are equal to the actual loads applied on that pile. Assessment of differential settlements (and angular distortion) across the pile group is a key design issue. For design purposes, a higher limit for K_{rs} may be adopted and rafts with K_{rs} < 0.1 can generally be considered as "practically flexible".

- For 0.1 < K_{rs} < 1.5, the raft has an "intermediate" stiffness which should properly be considered in design via a suitable numerical analysis, given that the raft-soil interaction will have a dominant role on raft behaviour.

- For K_{rs} > 10, the raft behaves as "fully rigid" and is therefore able to redistribute external load between the piles. In this case, the loads acting on each pile can be determined by means of pile-soil-pile interaction analysis, while differential settlements across the pile group can be considered as negligible. Based on the consideration that the K_{rs} value of Equ. (4) does not include the additional stiffening contribution provided by the piles and by the superstructure which in effect increases the actual raft rigidity, a lower limit for K_{rs} may be assumed for design purposes and rafts with K_{rs} > 1.5 can generally be considered as "practically rigid" (Randolph 2003).

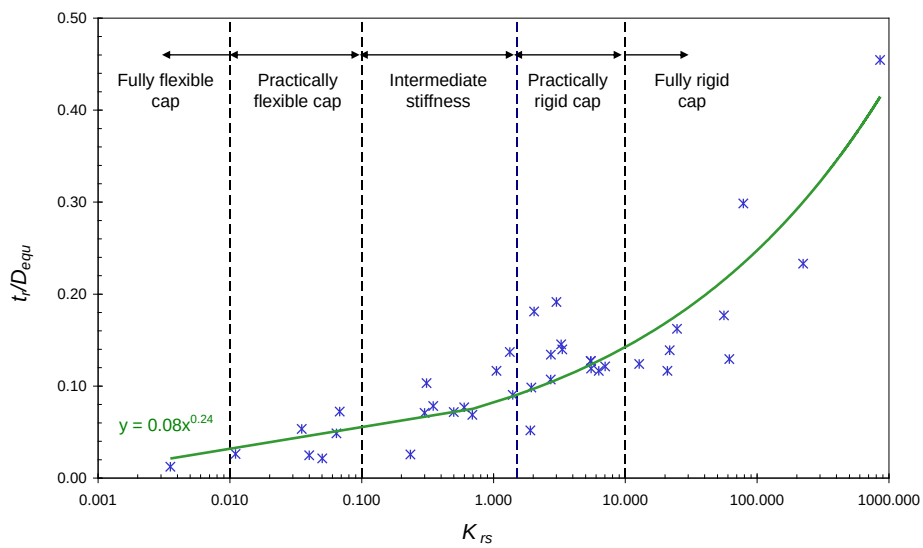

Figure 2. K_{rs} database and relationship with t_r/D_{equ}

Table 1. Details of real cases considered in K_{rs} database

Case	Reference	Description	n	L_p (m)	s/D	L_r (m)	B_r (m)	D_r (m)	t_r (m)	t_r/D_{equ}	E_s (MPa)	K_{rs}
1	Author's database	Viaduct	20	36.0	3.0	15.5	13.2	-	2.00	0.12	23	12.8
2	Author's database	Viaduct	12	22.1	3.4	19.5	5.1	-	1.50	0.13	14	2.72
3	Author's database	Building	220	18.0	6.1	42.6	22.3	-	0.91	0.03	88	0.01
4	Author's database	Building	18	13.0	5.0	25.1	6.3	-	0.76	0.05	57	0.04
5	Author's database	Building	159	31.0	2.5	54.1	34.1	-	3.50	0.07	600	0.07
6	Author's database	Viaduct	10	15.1	5.2	6.3	1.4	-	1.00	0.30	4	78.8
7	Author's database	Building	36	15.0	-	19.8	9.8	-	2.00	0.13	26	5.49
8	Author's database	Tank	40	9.0	-	-	-	20.6	2.40	0.12	31	20.9
9	Author's database	Viaduct	8	16.0	3.1	6.2	2.2	-	0.80	0.19	50	3.01
10	Author's database	Wind turbine	26	7.5	-	-	-	17.2	2.50	0.15	106	3.27
11	Author's database	Bridge	4	5.0	2.5	2.7	2.7	-	1.40	0.45	26	859
12	Author's database	Building	45	10.0	3.1	24.0	12.0	-	1.50	0.08	83	0.35
13	Author's database	Wind turbine	16	25.0	-	-	-	17.5	1.20	0.07	92	0.69
14	Author's database	Building	35	25.0	3.0	5.9	4.1	-	0.90	0.16	21	24.7
15	Badellas et al. (1988)	Tank	112	40.7	3.2	-	-	37.6	0.80	0.02	35	0.05
16	Rampello et al. (2004)	Chimney	74	56.7	2.7	-	-	30.5	3.00	0.10	90	1.94
17	Liew et al. (2002)	Tank	137	27.9	4.3	-	-	19.5	0.50	0.03	11	0.23
18	Borsetto et al. (1991)	Chimney	281	25.0	3.3	-	-	30.4	4.25	0.14	154	3.33
19	Author's database	Viaduct	20	23.0	3.0	17.2	12.9	-	2.00	0.12	38	5.51
20	Author's database	Building	288	17.3-37.0	2.9	64.6	46.4	-	3.00	0.05	220	0.06
21	Author's database	Building	15	9.0	2.6	14.2	5.2	-	1.00	0.10	80	0.31
22	Author's database	Building	93	23.0	3.0	10.4	8.0	-	1.20	0.12	175	1.05
23	Author's database	Bridge	30	15.0	3.0	20.0	6.0	-	1.50	0.12	6	7.04
24	Author's database	Building	36	15.0	2.5	19.8	9.8	-	2.00	0.13	26	5.46
25	Author's database	Wind turbine	40	8.5	-	-	-	20.6	2.40	0.12	34	6.30
26	Author's database	Wind turbine	38	16.7	-	-	-	16.4	2.90	0.18	21	56.1
27	Author's database	Bridge	5	18.9	1.5	11.1	6.6	-	2.25	0.23	5	224
28	Author's database	Building	12	10.0	3.8	12.8	7.3	-	1.50	0.14	166	1.33
29	Sommer et al. (1985)	Building	42	20.0	3.6	24.5	17.5	-	2.50	0.11	49	2.72
30	Tazoh et al. (1987)	Bridge	64	22.0	2.5	12.0	12.0	-	1.75	0.13	7	61.5
31	Reul & Randolph (2004)	Building	25	22.0	-	-	-	-	3.50	0.07	-	0.30
32	Reul & Randolph (2004)	Building	64	31.0	-	58.8	58.8	-	6.00	0.09	-	1.40
33	Reul & Randolph (2004)	Building	40	30.0	-	-	-	-	4.70	0.08	-	0.60
34	Reul & Randolph (2004)	Building	47	25.0	-	-	-	-	2.00	0.02	-	0.04
35	Reul & Randolph (2004)	Building	54	14.0	-	37.1	37.1	-	3.00	0.07	-	0.50
36	Author's database	Viaduct	11	19.0	3.2	13.8	8.4	-	2.20	0.18	287	2.04
37	Kakurai et al. (1987)	Tank	5	22.7	-	-	-	11.6	0.60	0.05	14	1.91
38	Author's database	Wind turbine	24	31.0	-	-	-	18.0	2.50	0.14	27	21.7
39	Van Impe et al. (2015)	Tank	422	21.5	4.8	-	-	48.8	0.60	0.01	105	0.004

Note: n = No. of piles, L_p = pile length, s/D = pile spacing, L_r = raft length, B_r = raft width, D_r = raft diameter, t_r = raft thickness, D_{equ} = raft equivalent diameter, E_s = soil Young's modulus, K_{rs} = raft-soil stiffness ratio

The above definition of K_{rs} has been used to verify the raft stiffness in No. 39 real cases including both published case histories and projects from the author's database. The main features for each case are listed in Table 1, ranging from small pile groups for bridges to large piled rafts for high-rise buildings, so that they are believed to be representative of a broad range of practical problems.

The computed K_{rs} values for each case are plotted on a logarithmic scale in Figure 2, varying from values of about 0.004 to 1000 (the best-fit power curve of the data set is also shown). In addition, in order to provide a simple geometric parameter purely based on the raft dimensions and independent of the raft and soil Young's modulus, the ordinate axis reports the corresponding t_r/D_{equ} values (where $D_{equ} = 2\sqrt{B_r L_r/\pi}$ is the raft equivalent diameter). It may be observed that, for $t_r/D_{equ} > 0.10$, the raft generally tends to behave as a rigid raft, while for $t_r/D_{equ} < 0.05$, the raft is likely to belong to the flexible category. The latter result confirms the findings of Poulos (2016), i.e. large rafts tend to behave as fully flexible rafts. The above limits for t_r/D_{equ} may therefore be used for a preliminary indication of the raft stiffness in the absence of any information on the raft and soil Young's moduli (which is required for a more rigorous assessment of the raft stiffness using Equ. 4).

4 NUMERICAL RESULTS

4.1 Comparison with PIGLET and DEFPIG

Accuracy of the proposed PGROUPN solution is assessed through a comparison with the numerical results obtained by two established computer programs for pile-group analysis, i.e. PIGLET (Randolph 2004) and DEFPIG (Poulos 1990). A simple hypothetical case of a 9-pile group under vertical loading is analysed, as sketched in Figure 3a. The piles are assumed to be 20 m long, 1 m in diameter, with a Young's modulus of 30 GPa, and arranged in a square grid with a spacing of 3.0 m. The piles are embedded in a deep homogeneous elastic soil layer having a soil Young's modulus of 50 MPa and a Poisson's ratio of 0.5. In order to investigate the effect of pile cap rigidity, two limiting conditions for the cap are considered, i.e. a fully-rigid (free-standing) cap in which the total applied load is equal to 9 MN (i.e. an average of 1 MN/pile), and a fully-flexible cap in which the applied load on each pile head is equal to 1 MN.

Results are reported in terms of the maximum pile settlement, maximum pile differential settlement, and normalised maximum pile force (P_{max}/P_{ave}) in Figures 3b, 3c, and 3d, respectively, showing a favourable agreement between programs. From these figures, the following observations can be made:

- The fully-flexible cap results in a larger (by about 12%) and hence more conservative value of maximum pile settlement as compared to the fully-rigid cap (clearly in the latter case the settlements of all piles are equal);
- The fully-flexible cap allows the evaluation of differential settlements (and angular distortion) across the pile group;
- In the case of fully-rigid cap, the cap redistributes external load between the piles, thereby resulting in a non-uniform load distribution, with the corner pile carrying the greatest proportion of load and the centre pile carrying least;
- For caps of intermediate stiffness, the two limiting conditions of the cap stiffness will provide lower and upper bounds for the maximum values of pile settlement, differential settlement and force.

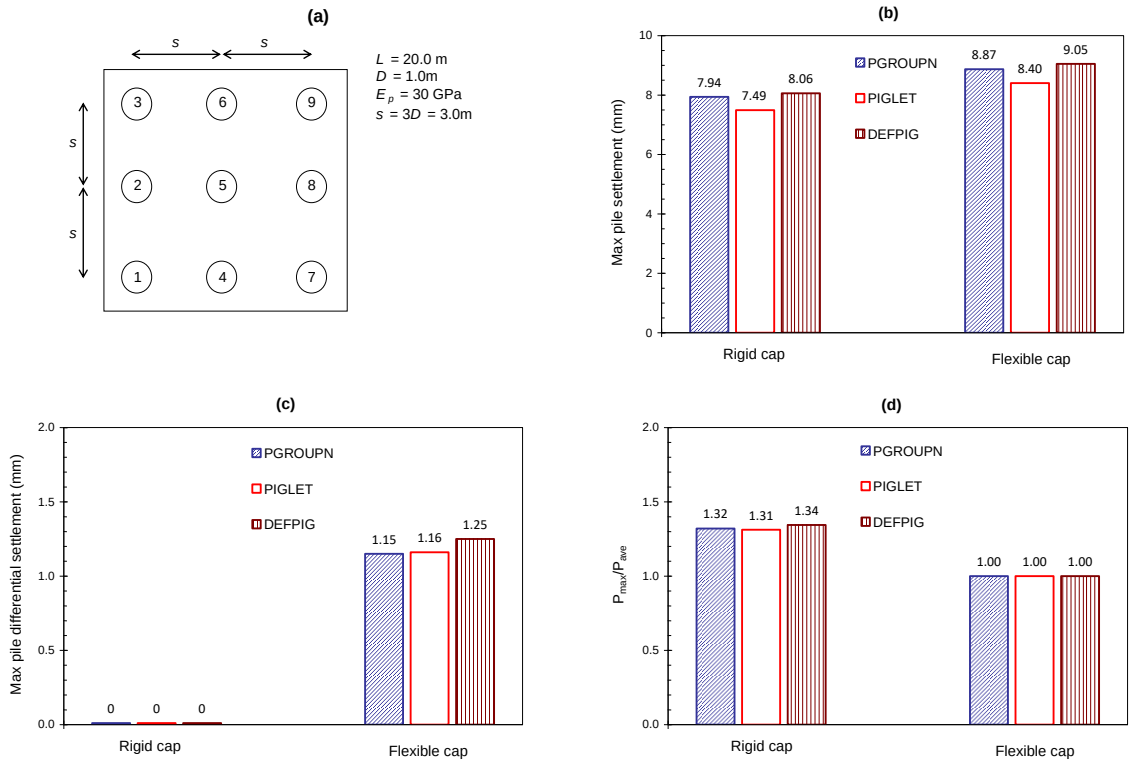

Figure 3. (a) Group layout; (b) Pile settlement; (c) Pile differential settlement; (d) Pile force

It should be observed that the above results are based on the assumption of linear elastic soil behaviour (due to a limitation of the numerical codes used for comparison). However, in order to model a more realistic response, it is essential to consider the effects of soil nonlinearity. For this purpose, the same pile group shown in Figure 3a is reanalysed using PGROUPN non-linear soil model for the ground conditions reported in Table 2. The resulting load-settlement response for the two limiting conditions of the cap stiffness is shown in Figure 4 (together with the corresponding single-pile response), thus confirming that the fully-flexible cap assumption results in a larger value of maximum pile settlement as compared to the fully-rigid cap.

Table 2. Ground model for non-linear analysis

Depth range (m)	E_s (MPa)	ν_s	f_s (kPa)	f_b (MPa)
0-10	50	0.2	50	-
10+	200	0.2	100	5.0

Note: E_s = initial Young's modulus, ν_s = Poisson's ratio, f_s = shaft friction, f_b = ultimate base resistance

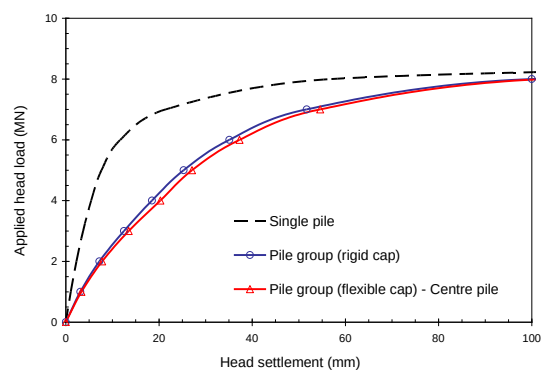

Figure 4. PGROUPN load-settlement response

4.2 Assessment of pile spring stiffness values

Structural design software usually offers the facility to model the foundation system as a raft slab supported by springs representing the piles. The stiffness of these springs (defined as load divided by displacement) should be assessed by the geotechnical engineer via a pile-group analysis which takes into account essential aspects such as pile-soil-pile interaction and soil nonlinearity.

In order to determine the pile spring stiffness values, an iterative procedure between the structural analysis of the superstructure and the geotechnical pile-group analysis is commonly carried out. This requires the structural designer adjusting the spring stiffness values obtained from the geotechnical analysis until a reasonable match between calculated displacements of the structural and geotechnical analyses is obtained. The revised column loads are then provided back to the geotechnical engineer and the process repeated until an acceptable level of convergence is achieved (usually within two-to-four iterations). Different spring stiffness values are computed for each component of load (i.e. vertical, lateral and moment); although attention is herein focussed on the vertical spring stiffness values, a similar approach can be adopted for assessing the lateral and rotational spring stiffnesses. However, vertical, lateral and moment loads should be considered separately in order to avoid unrealistic stiffness values that can arise under some combinations of vertical, lateral and moment loading (Poulos 2018).

It is worth noting that, in the assessment of pile spring stiffness values via a pile-group analysis, the assumption of fully-flexible cap is often a more reliable approach than that of fully-rigid cap (Poulos 2017); thus, the proposed PGROUPN extension to the fully-flexible cap can have a useful role in design practice. In order to illustrate the application of the above approach, the 9-pile group previously described in Figure 3a with the ground profile reported in Table 2 is considered. Table 3 and Figures 5a-b report the vertical pile-head stiffness values (i.e. applied load divided by displacement) computed for the two limiting conditions of the cap stiffness. Considering that the ultimate pile capacity results in 8.6 MN, two loading conditions are selected, i.e. an average serviceability (SLS) load of 3.5 MN/pile (i.e. a total load on the group of 31.5 MN) and an average ultimate (ULS) load of 5.0 MN/pile (i.e. a total load on the group of 45.0 MN). The following features of behaviour are observed:

- Different stiffness values are obtained for each pile location within the group, with the corner piles showing the largest stiffness and the centre pile (No. 5) the smallest one. This demonstrates the shortcomings of ignoring pile-soil-pile interaction effects and assuming constant spring stiffness values across the pile group (as incorrectly adopted by some structural engineers);
- The stiffness values for the fully-flexible cap assumption show a more uniform distribution across the pile group than those based on the fully-rigid cap assumption;
- The group stiffness values are significantly lower than the single-pile stiffness value, thereby confirming the importance of considering pile-soil-pile interaction effects;
- The stiffness values for the ULS case are significantly lower than those for the SLS case, thus attesting the value of accounting for soil nonlinearity effects.

Table 3. Pile-group stiffness values

Pile location	Pile-head stiffness values (MN/m)			
	SLS case		ULS case	
	Rigid cap	Flexible cap	Rigid cap	Flexible cap
Corner	261	232	218	203
Mid-side	210	220	189	194
Centre	149	208	151	185

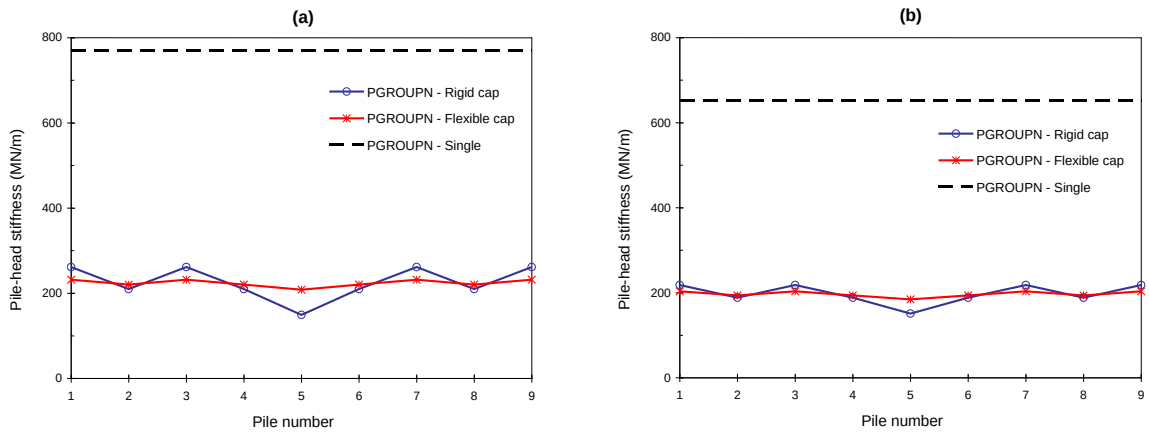

Figure 5. Pile-head stiffness values: (a) SLS case; (b) ULS case

5 CASE HISTORIES

Attention is turned to the application of PGROUPN to two published case histories involving pile groups where the assumption of fully-flexible cap can reasonably be adopted.

5.1 Liquid storage tank (Thessaloniki, Greece)

The case history described by Badellas et al. (1988) relates to a liquid ammonia storage tank constructed in the city of Thessaloniki (Greece). The tank is founded on a circular concrete raft, 1.30 m above ground level, with a diameter of 37.6 m and a thickness of 0.8 m. The raft is supported by a group of 112 bored piles with an embedded length of 40.7 m, a diameter of 1.0 m, an assumed Young's modulus of 30 GPa, and an average pile spacing of 3.15 m, as illustrated in Figure 6.

The foundation soil is mainly composed of very soft to medium stiff silty clay and clayey silt. The three-layer soil profile and undrained shear strength (C_u) values reported by Georgiadis et al. (1989) have been adopted, while the pile adhesion factor (α) has been computed based on Poulos (2017), as summarised in Table 4. The initial value of soil Young's modulus (E_s) to be used in the PGROUPN non-linear analysis has been derived by back-analysis in order to match the maximum measured settlement of the central piles (i.e. 33.9 mm). This has resulted in a E_s/C_u ratio of 1400 and the corresponding E_s values are reported in Table 4. It is noted that the computed value of raft-soil stiffness ratio K_{rs} from Equ. (4) is equal to 0.05, thus confirming the validity of the fully-flexible raft assumption. In Equ. (4), the raft Young's modulus (E_r) is assumed to be 30 GPa, while the soil Young's modulus (E_s) is equal to 35.3 MPa, as calculated on the basis of Equ. (5) with a secant value taken as 0.7 of the initial (small strain) value.

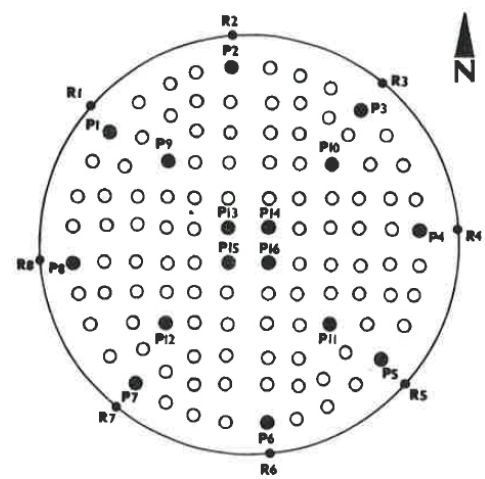

Figure 6 Group layout (after Badellas et al. 1988)

The tank was hydrotested with a weight of 15,000 t of water which is assumed to be uniformly distributed among the 112 piles (i.e. 1.31 MN/pile). During the hydrotest, the settlements of 16 piles were monitored with the maximum settlements measured in the four central piles (with a mean value of 33.9 mm), while the piles located around the slab periphery settled between 23.5 and 27.5 mm (with a mean value of 25.6 mm), as shown in Figure 7. The pile settlements computed by PGROUPN are also reported in the figure, showing the ability of the code to predict the pile differential settlements across the pile group.

Table 4. Ground profile and parameters

Layer	Depth (m)	E_s (MPa)	ν_s	C_u (kPa)	α
Silty clay/Clayey silt	0-21	50.4	0.5	36	0.70
Silty clay	21-30	103.6	0.5	74	0.42
Silty clay	30+	161.0	0.5	115	0.33

Note: E_s = initial Young's modulus, ν_s = Poisson's ratio, C_u = undrained shear strength, α = pile adhesion factor

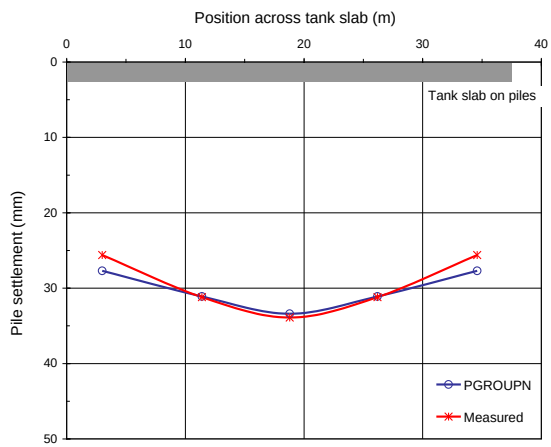

Figure 7 Measured and predicted pile settlements for the Thessaloniki tank

5.2 Liquid storage tank (Menstrie, UK)

The molasses storage tank described by Thorburn et al. (1983) has a diameter of 12.5 m and is supported by a group of 55 precast concrete driven piles with a length of 29.0 m, a square section of 0.25 m, a Young's modulus of 26 GPa, and laid out on a triangular grid at a spacing of 2.0 m. The piles were not connected by a cap and a 2.0 m thick pad of dense granular material (incorporating a 150 mm thick reinforced concrete membrane) was constructed over the pile heads. Following Randolph (1994), the foundation soil can be modelled as a unique cohesive layer with parameters linearly increasing with depth, i.e. an undrained shear strength given by the expression $C_u = 6 + 1.8z$ kPa and an initial value of soil Young's modulus (E_s) based on a E_s/C_u ratio of 750. The soil Poisson's ratio is 0.5 while the average pile adhesion factor (α) can be calculated as 0.82 based on Poulos (2017).

Under a total applied load of 2,000 t (i.e. an average load of 357 kN/pile assuming uniform distribution among the 55 piles), measured settlements were in the range 29-30 mm for the piles located around the tank periphery, while the measured differential settlements across the pile group were less than 10 mm (i.e. within the accuracy of the instrumentation used to measure such settlements). The above measurements compare well with those predicted by PGROUPN which computes a settlement in the range 29-31 mm for the edge piles and a maximum differential settlement of 7 mm between the centre and edge of the pile group. Finally, it should be observed that the computational time required for the 55-pile group analysis (using the non-linear soil model) is a matter of a few minutes on an ordinary computer, thereby confirming the value of PGROUPN as a practical numerical tool for the design of pile groups.

6 CONCLUSIONS

The paper has described an efficient analysis method, based on the 3D BEM solution of the code PGROUPN (i.e. the calculation engine of Repute software), for determining the non-linear response of pile groups with a fully-flexible cap

(i.e. individual piles not connected by cap). As compared to the analysis with fully-rigid cap, the proposed extension to the fully-flexible cap can be useful in practice given that:

- it allows computation of pile differential settlements and angular distortion across the pile group;
- it avoids overestimation of forces and moments in the outer piles of the group;
- it computes a larger (and hence more conservative) maximum pile settlement;
- it can be valuable in the assessment of pile spring stiffness values for the superstructure analysis.

The proposed method has been successfully compared with the results from alternative numerical solutions and field measurements which have confirmed the value of PGROUPN as a useful tool for pile group analysis. In addition, a database of real cases has been collated showing the relationship between the raft geometry and rigidity. Results provide insight into the field of applicability of the rigid and flexible cap assumptions, indicating that most large rafts are likely to belong to the fully-flexible category.

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