PILE GROUPS WITH NEGATIVE SKIN FRICTION

Francesco Basile, Geomarc Ltd, Messina, Italy, tel +39 338 7038381, francesco.basile@geomarc.it

ABSTRACT

An efficient analysis method is presented for assessing the response of single piles and pile groups subjected to negative skin friction (NSF). The method involves two separate steps: (1) estimate of the free-field ground movements in the consolidating soil, and (2) analysis of the piles subjected to the computed free-field ground movements by means of PGROUPN, a computer program based on a 3D non-linear boundary element solution. The impact of various factors which affect the response of piles to NSF is examined, including the number of piles, the pile spacing, the additional presence of pile-head loading, and the influence of soil nonlinearity and pile-soil slip. Results provide insights into the correct understanding of NSF effects on pile behaviour and show the importance of considering such aspects as pile-soil slip and group interaction effects. In particular, it is demonstrated that the usual detrimental effect of group interaction among piles is diminished by the beneficial group effect for piles under NSF, thereby resulting in a lower values of drag force and drag settlement as compared to a single isolated pile.

Keywords: piles, pile groups, BEM, negative skin friction

INTRODUCTION

Pile foundations embedded within a consolidating soil layer are subjected to negative skin friction (NSF), i.e. negative shear stresses caused by the downward movement of the soil relative to the pile. Misconception remains among some pile designers that NSF will reduce the geotechnical axial capacity of a pile. However, it should be recognized that NSF pile design is mainly a settlement rather than a geotechnical capacity issue, and therefore it should be approached from a serviceability viewpoint by means of a pile-soil interaction analysis. Indeed, as pointed out by many researchers (e.g. Poulos, 2008; Fellenius, 2017), geotechnical failure implies that the entire pile moves past the soil and therefore NSF cannot be present at this point (given that NSF requires the soil to move past the pile). In such a case, NSF actually turns into positive shaft resistance and thus should generally not be considered in assessing the pile geotechnical capacity (unless there is significant strain softening of the interface friction, which is unlikely in the common situation of NSF in soft clay).

The main concerns for pile design in the presence of NSF are therefore two-fold:

1) ULS — the additional force ('*drag force*') induced in the pile by NSF at the neutral plane location (i.e. the depth at which the friction changes from negative to positive), and its effect on the pile structural strength;

2) SLS — the additional pile settlement ('*drag settlement*') caused by NSF, and its effect on the serviceability limit state design, i.e. settlements and differential settlement must remain within acceptable limits.

In addition, the pile geotechnical capacity must clearly be adequate for the head load (i.e. dead load and live load), while ignoring the NSF effect. In practice, the NSF effect on pile structural capacity only becomes significant for end-bearing piles (i.e. those which terminate in relatively impenetrable material such as rock or very dense sand or gravel), whereas, in the case of floating piles, the serviceability problem is usually prevailing and governs the design.

Current design practice for piles subjected to NSF is often unduly conservative because of the following reasons:

1) The drag force induced by NSF is incorrectly treated as an *external* unfavourable action (usually at the pile-head level) that reduces the pile geotechnical capacity; however, as discussed above, this is a faulty concept and can lead to an overconservative or even erroneous (Fellenius, 2017) design. The drag force should instead be regarded as an *internal* force associated with the development of static equilibrium of the pile-soil system. The use of "force" instead of "load" is intentional to make a distinction that the drag force should not be considered as an applied load on the pile head in design.

2) The drag force caused by NSF is usually evaluated on the assumption of full pile-soil slip, i.e. it is assumed that NSF is fully mobilized above the neutral plane regardless of the magnitude of ground movement. This can be an overconservative assumption when the ground movements are expected to be small, such as in a matured or treated (e.g. with vertical drains) reclaimed land. Similarly, the depth to the neutral plane (which actually is the result of a complex pile-soil interaction problem) is often determined in an empirical manner (e.g. at the base of the settling soil) and disregarding the magnitude of ground movements. As a result, the depth may be overestimated and thus the drag force would be further exaggerated.

3) Beneficial group effects are ignored — in contrast to conventional axial loading at pile-head level, pile-to-pile interaction effects have a favourable effect in the case of NSF, i.e., as compared with a single isolated pile, there is a tendency for reduced drag forces and drag settlements in the group, especially for the inner piles.

The above shortcomings may be overcome by adopting a more fundamental approach in which the NSF-induced ground settlement is actually treated as a 'geotechnical' action on the pile group and a pile-soil interaction analysis is carried out (clearly the presence of any direct loading at the pile head should also be included). This would allow a more realistic (and often more economic) assessment of the key features of pile response to be used in design, i.e. the pile-group settlement, the drag force, and the depth to neutral plane. It is encouraging to observe that such an approach is starting to become recognized (at least partially) by some modern codes, e.g. in Eurocode 7 the designer is given the option to allow for NSF by using pile-soil interaction analysis when the ground settlements are expected to be relatively small (and thus the "traditional" design for NSF would be too conservative).

A rigorous pile-soil interaction analysis including the NSF problem can be based on continuum approaches, such as the finite element (FEM) and the finite difference (FDM) methods. Such "complete" solutions allow for the proper modelling of the consolidation process, and hence the development of NSF-induced ground settlements is computed within the analysis by the application of surcharge loading (e.g. Comodromos and Bareka, 2005; Lee et al, 2009; Tan and Fellenius, 2016). However, even though FEM and FDM analyses are most powerful numerical tools, their use would be unpractical in the routine design of piles (and pile groups) under NSF due to the excessive complexity and high computational costs involved, particularly considering that accounting for pile-soil slippage and non-linear soil behaviour is essential. In the NSF problem, such limitations are further exacerbated by the increased computing time required by a consolidation analysis and also by the consideration that this level of complexity may not always be warranted because of the often limited available information related to ground movements and how they are developed (Poulos, 2017).

A more practical solution to deal with the analysis of piles under NSF is provided by simplified approaches involving initial separation of the soil and the pile domains so that the soil movements are first computed and then imposed on the piles. The approach is based on two separate steps:

(1) evaluation of the free-field downward ground movements in consolidating soil;

(2) analysis of the pile group subjected to the computed free-field ground movements (and to any direct load at the pile cap level).

(1) Estimation of ground movements

The magnitude of consolidation settlements, such as those induced by surcharge loading or dewatering, can be estimated using conventional settlement analysis, for example Terzaghi's one-dimensional consolidation theory. Although the time-rate of consolidation settlements may also be considered, end-of-consolidation settlements is often assumed in practice. Consolidation settlements are generally evaluated in free-field conditions, i.e. in the absence of the piles. This generally is a conservative assumption as the presence of the piles increases the soil stiffness, thereby reducing the induced ground movements (e.g. Mroueh and Shahrour, 2002).

(2) Analysis of pile response

The second step of the procedure can be carried out by a boundary element (BEM) analysis of the single pile or the pile group subjected to the NSF-induced ground movements evaluated above. BEM solutions are quite fitting for the NSF problem as they provide an effective means of retaining the essential aspects of pile interaction through the soil continuum, without incurring the high costs and complexity of FEM and FDM solutions. Examples include the simplified BEM approach for single piles and symmetrical pile groups proposed by Kuwabara and Poulos (1989) and a similar solution by Teh and Wong (1995). In this paper, a more general BEM approach is proposed which provides a complete 3D non-linear solution of the soil continuum, while retaining a computationally efficient code.

PGROUPN ANALYSIS

The proposed method is based on the two-step approach described above (as depicted in Figure 1) and is carried out within PGROUPN (Basile 2003), a numerical code for pile-group analysis which is commonly used in design through the software Repute (Bond and Basile, 2017). The analysis takes into account the simultaneous influence among 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 pile-soil slip and soil nonlinearity is modelled using a hyperbolic continuum-based interface model, while completely general direct loading conditions (axial, lateral, and moments) on the pile group can be examined, in addition to the profile of external ground movements which is computed separately as at Step (1) above.

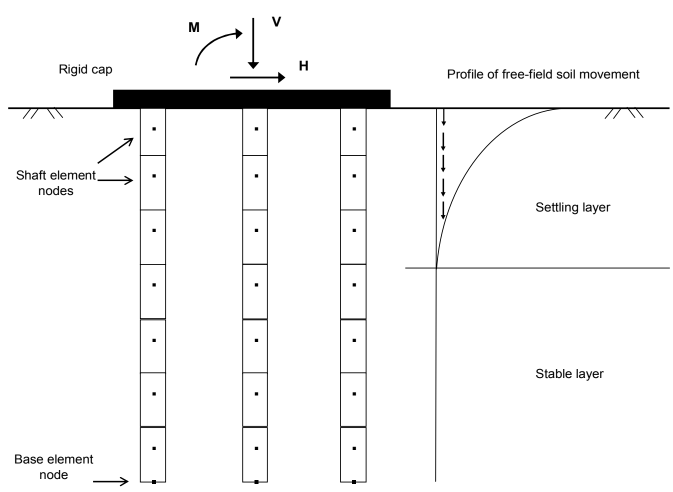

Fig. 1. BEM schematisation of the problem

It is noted that the PGROUPN analysis may be employed not only in the consolidation case described herein but in many circumstances in which pile foundations are subjected to “passive” loadings arising from vertical and/or horizontal movements of the surrounding ground. Examples include tunnelling (Basile 2014), kinematic effects induced by earthquakes (Basile 2012), slope movement, excavation, swelling of an expansive clay, and construction of adjacent piles or buildings.

The general capabilities of the PGROUPN code are summarized below:

● based on 3D complete BEM solution of the soil continuum;
● models all relevant interactions (i.e. pile-soil, pile-pile, raft-soil and pile-raft);
● piles in any configuration and having different characteristics within the same group (e.g. stiffness, length, rake, shaft and base diameter);
● piles connected by rigid and, if appropriate, ground-contacting cap;
● multi-layered soil profiles;
● linear elastic, elastic-plastic or non-linear continuum-based soil model;
● general 3D loading conditions, including any combination of vertical, horizontal, moment, and torsional loading acting on the pile cap;
● externally imposed vertical and/or horizontal ground movements;
● output includes the distribution of displacements, stresses, forces and moments along the piles, plus the normal stresses, displacements and rotations of the pile cap.

A detailed description of the theoretical formulation of PGROUPN has been presented elsewhere (Basile 2003, 2014) 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 (located at the mid-height of the element on the centre line of the pile), with the distributions of the stress components at the pile-soil interface which are assumed to be uniform over each element. The method employs a sub-structuring technique in which the piles and the surrounding soil are initially considered separately and then coupled by imposing the displacement compatibility ($u_s = u_p$) and stress equilibrium ($t_s = -t_p$) conditions at the pile-soil interface.

Soil domain

Assuming purely linear elastic soil behaviour, the soil displacements, arising both from the stresses caused by pile-soil interaction and the external source of ground movement, may be expressed as:

$$\{u_s\} = [G_s]\{t_s\} + \{u_e\} \quad \text{(Equ. 1)}$$

where u_s are the soil displacements, t_s are the soil stresses, G_s is the soil flexibility matrix obtained from Mindlin’s (1936) solution, and u_e are the external soil movements. It is noted that Mindlin’s solution is rigorously applicable to homogeneous soil conditions. In practice, however, this limitation is not strictly adhered to, and the influence of soil non-homogeneity can be approximated using a weighted average value of soil modulus at the influencing and influenced pile nodes (e.g. Yamashita et al., 1987; Poulos, 2009).

Pile domain

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

$$\{u_p\} = [G_P]\{t_p\} \quad \text{(Equ. 2)}$$

where u_p are the pile displacements, t_p are the pile stresses, and G_p is a matrix of coefficients obtained from the elementary (Bernoulli-Euler) beam theory.

Limiting stress and 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:

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

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 and compatibility 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.

NUMERICAL RESULTS

Validity of the proposed analysis is confirmed through a comparison with published results from well-established numerical solutions.

Comparison with Poulos (2006)

In order to illustrate the crucial difference between the effects of loading induced by external ground movements and direct applied load at the pile-head, a single pile in a two-layer soil profile is analysed under the following types of loading (Fig. 2):

(a) An axial load of 1 MN applied at the pile head;
(b) A downward ground movement profile linearly decreasing from 100mm at the ground surface to zero at a depth of 12m.

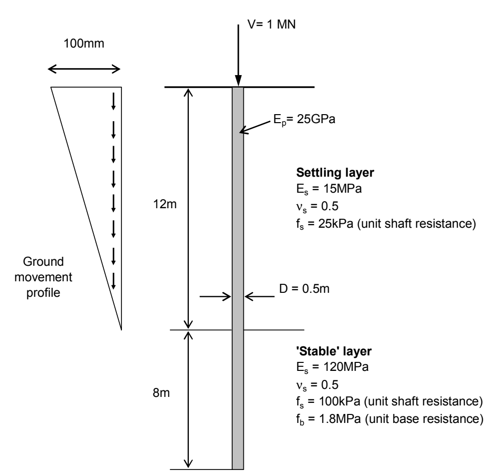

Fig. 2. Pile in two-layer soil system

The resulting axial force distribution computed by PGROUPN for loading types (a) and (b) is reported in Fig. 3, showing a favourable agreement with the results obtained by Poulos (2006) using a similar BEM approach with an elastic-perfectly plastic interface model (in practice corresponding to the use of R_f =0 in Equ. 3). Figure 3 also reports the distribution generated by the two type of loadings (a) and (b) acting together, and the distribution resulting from the addition of the two profiles of axial force obtained for loadings (a) and (b). The following features of behaviour are shown:

(a) The distribution of axial force in the pile due to direct loading is very different from that induced by ground movements. In the former case, the maximum axial force occurs at the pile head, whereas in the latter case, the maximum axial force (i.e. the drag force) occurs near the bottom of the layer subjected to ground movement. This consideration further shows the fallacy of treating the NSF as a force applied at the pile head, as still adopted by some pile designers.

(b) The simple addition of the two profiles of axial force obtained for loadings (a) and (b) would be unconservative, i.e. it yields axial forces which are less than those arising from the simultaneous application of loadings (a) and (b). This is because the downward ground movement has caused full slip at the pile-soil interface of the upper settling layer (i.e. the limiting unit shaft resistance is reached) and hence the superposition principle does not apply.

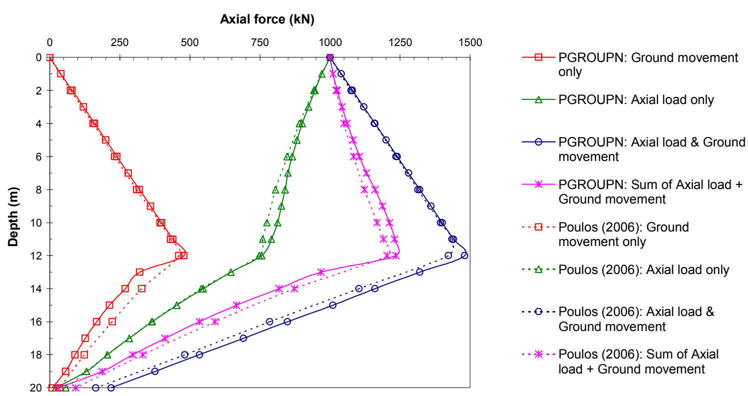

Fig. 3. Comparison of axial force distribution with Poulos (2006)

Effect of soil slip

Consideration of soil nonlinearity (including soil slip at the pile-soil interface) is essential in NSF pile design since soil slip can develop under relatively small ground movements. This results in a smaller drag force (and drag settlement) than that computed using a linear elastic (LE) model because only limited shear stress is permitted to transfer from the consolidating soil to the pile. Indeed, the LE model does not allow slip to take place at the pile-soil interface (i.e. it does not impose any limiting

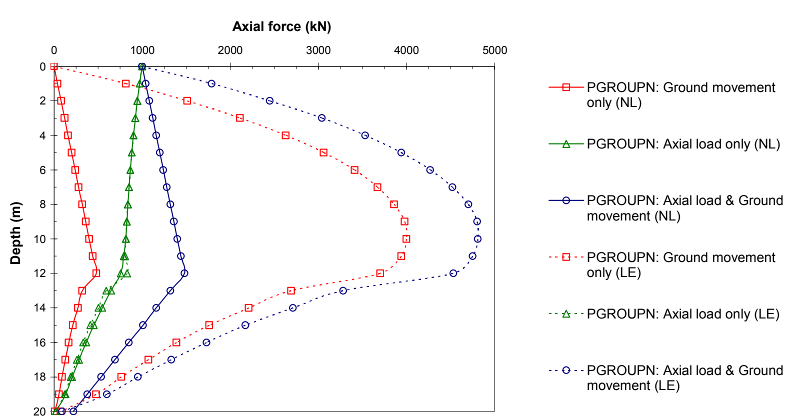

Fig. 4. Comparison of LE vs. NL analysis

values for the shear stress), thereby resulting in unrealistically large values of shear stress and thus drag force (and drag settlement). For the test case reported in Fig. 2 under loading (b) from ground movements only, the NL analysis computes that the unit shaft resistance is fully mobilized along the portion of the pile in the settling zone, thereby causing slippage at the pile-soil interface. Figure 4 shows that use of the NL analysis leads to a 88% reduction of the maximum drag force (i.e. from 3940 kN to 490 kN), as compared to the LE analysis.

Comparison with Poulos (2008)

Two hypothetical but typical problems reported by Poulos (2008) are analysed, as shown in Fig. 5. The first case involves an *end-bearing* pile embedded into a 20m thick settling soft clay layer, underlain by a "stable" stiff clay layer with a length of pile in the stable zone equal to L_c = 6m (giving a total pile length of 26m). The second case relates to a *floating* pile embedded into an identical settling layer as for the end-bearing pile, while the underlying stable layer is a medium clay layer (with significantly smaller strength and stiffness than in the first case) and the pile length in the stable zone is L_c = 18m (i.e. a total pile length of 38m). Both piles are subjected to an identical ground settlement profile linearly decreasing from a maximum value (S_o) at the ground surface to zero at a depth of 20m, whereas the applied loads (e.g. dead load) at the pile head are 1.5MN and 0.8MN for the end-bearing and floating pile, respectively.

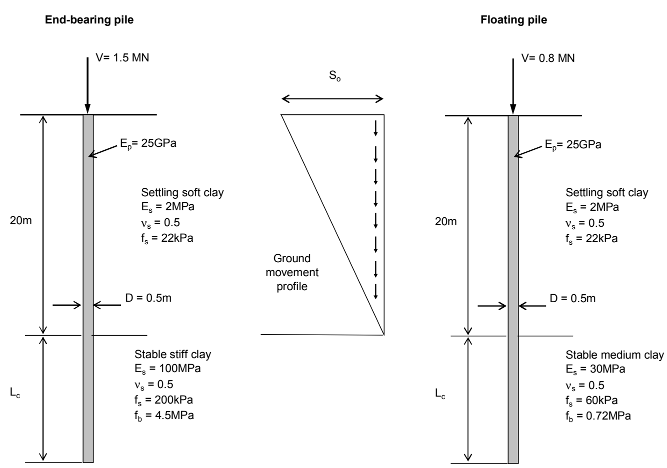

Fig. 5. End-bearing and floating pile in two-layer soil system

In order to emphasize the importance of accounting for the actual magnitude of the ground movements in the design of piles under NSF, attention is initially concentrated to the case in which the piles are only subjected to the ground settlement profile. Figure 6 shows the development of the maximum drag force and pile settlement (i.e. drag settlement) with increasing soil surface settlement (S_o). It is noted that, not only the maximum drag force, but also the pile drag settlement reaches a limiting value, regardless of the soil surface settlement — this is a most critical feature of behaviour which has to be recognized during the SLS check. Figure 6a also reports the limiting value f_sA_s = 691 kN (where A_s is the shaft area of the pile upper portion) which would generally be considered in a traditional NSF design under the assumption of full mobilization of the shear stress above the neutral plane (which is considered to be located at the base of the settling layer). It is observed that, only for soil surface settlements greater than about 200mm, the above assumption would be realistic. However, for smaller ground movements (e.g. S_o = 50mm), the maximum drag force is much less than that based on the simplistic full skin friction assumption. Figure 6 further proves that, in contrast to the case of conventional axial loading at the pile-head, the development of pile-soil slip is beneficial as it results in reduced drag forces and drag settlements as compared with a purely linear elastic interface condition. Indeed, pile-soil slip start to develop when the soil surface settlement is about 50mm and is almost complete at about 200mm. Thus, linear elastic theory would seriously overestimate the pile drag force and drag settlement for soil surface settlements in excess of about 50mm.

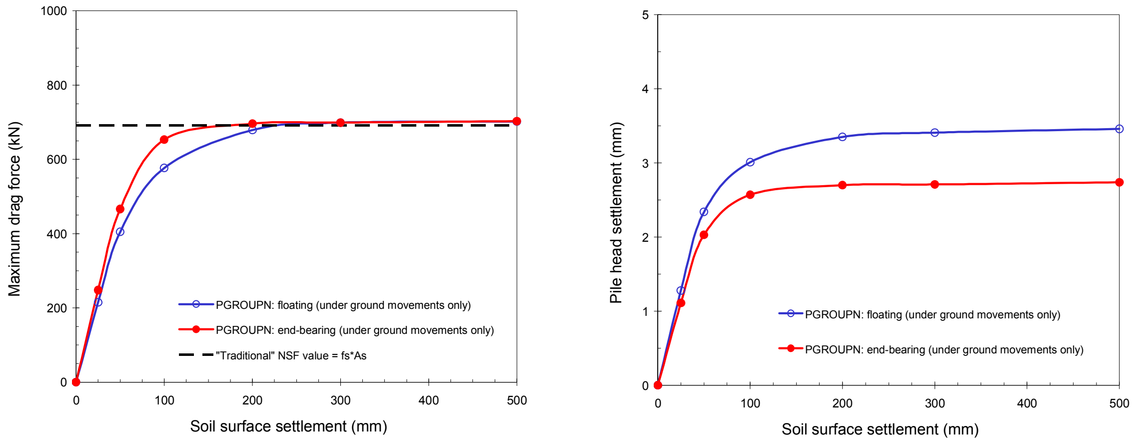

Fig. 6. Piles subjected to ground movement only: (a) Max. drag force; (b) Pile settlement

In Fig. 7, attention is turned to the more practical case in which the piles are subjected to both the ground settlement profile and the pile-head loads reported in Fig. 5. The non-linear correlation between pile drag force/settlement and ground settlement is similar to that discussed above, thereby confirming the value of accounting for soil slip via a pile-soil interaction analysis in order to achieve a more realistic assessment of pile response. For a soil surface settlement S_o = 100mm, Fig. 7b shows a good agreement of the pile head settlement values computed by PGROUPN with those reported by Poulos (2008) using the BEM approach described earlier.

Figures 8a-8c show the profiles of pile axial force, settlement, and interface shear stress, respectively, as obtained by PGROUPN for the floating pile subjected to the head load of 0.8MN and to two different ground settlement profiles (as reported in Fig. 8b). It is noted that the analysis is capable of consistently identifying the location of the neutral plane (NP), which can be defined as the plane where there is no relative movement between the pile and surrounding soil (i.e. where pile and soil settlement become equal). Indeed, the NP is also the location of the maximum force in the pile and where the direction of the pile shear stress reverses from negative to positive. Figures 8a-8c demonstrate that, for the same axial load, the larger ground settlements lead to an increased extent of soil slip along the pile, thereby resulting in a deeper NP location (i.e. from 17.5m to 19.5m depth), while the pile axial force and settlement are increased, as expected. It is noted that, due to the full mobilization of shear stress above the NP location, the transition zone from full NSF to positive shaft resistance becomes sharper and smaller for increasing ground settlement, as also obtained by Tan and Fellenius (2016) using a rigorous FEM analysis.

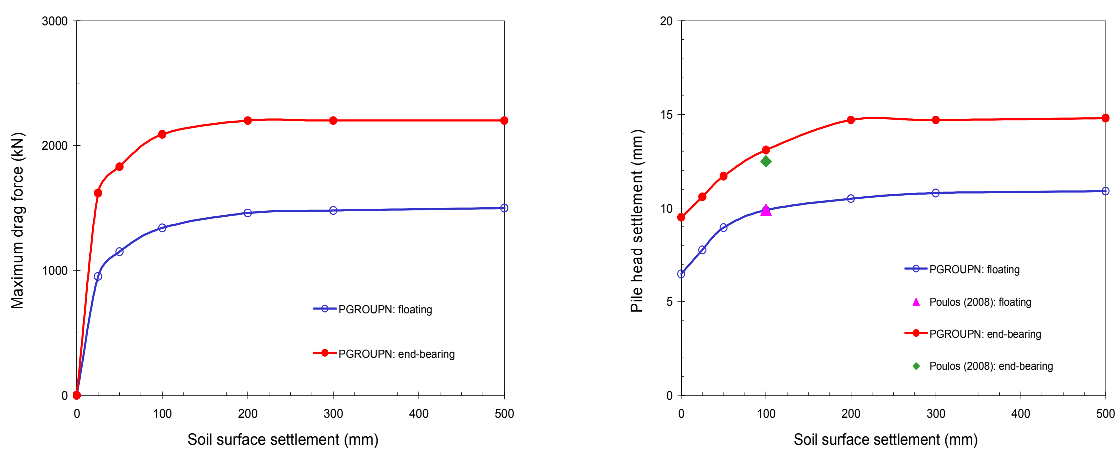

Fig. 7. Piles with head load & ground movement: (a) Max. drag force; (b) Pile settlement

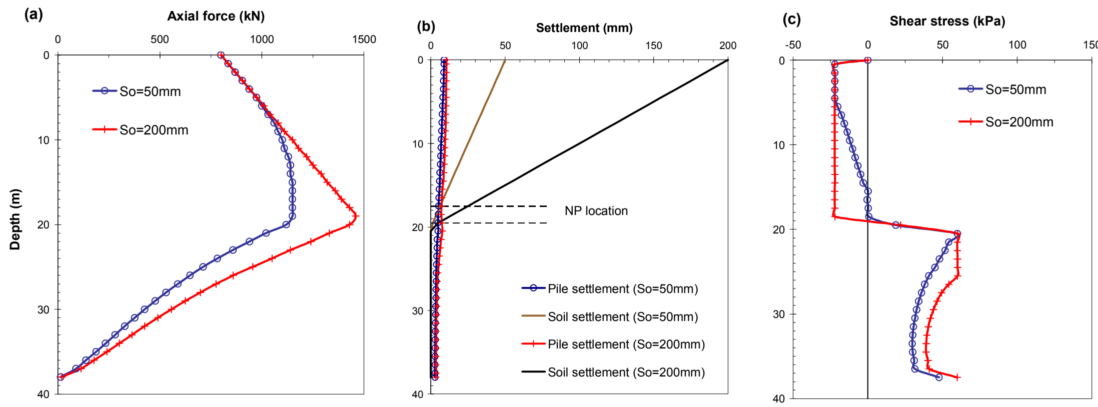

Fig. 8. Floating pile with head load & increasing ground movement

Comparison with Comodromos and Bareka (2005)

The behaviour of single pile and pile groups embedded in a multi-layered soil profile and subjected to direct head load and/or NSF-induced ground movements is investigated through a comparison of PGROUPN with the results obtained by Comodromos and Bareka (2005) using the more rigorous finite difference software FLAC-3D. The soil profile consists of a soft clayey silt 6m thick, underlain by highly plastic soft clay extending to a depth of 18.0m. Under this, a layer of medium stiff clay with a thickness of 24m, underlain by very dense sandy gravel extending to the end of the borehole at a depth of 80.0m. The soil profile and parameters used in the PGROUPN analysis are those reported by Comodromos and Bareka and are summarized in Fig. 9. The FLAC-3D non-linear analysis was based on secant values of soil modulus (E_{sec}), whereas the PGROUPN non-linear model adopts a soil modulus approaching the initial tangent value (E_i). Thus, in order to ensure consistency between analyses, a back-analysis of the single pile results has been carried out and a correlation equal to $E_i = 5E_{sec}$ is adopted (with hyperbolic factors of $R_f = 0.6$ for the shaft and $R_f = 0.99$ for the base, as per Equ. 3). In FLAC-3D, the consolidation settlement of the ground is computed within the analysis by means of the application of surface loading (corresponding to different heights of embankment), as reported in Fig. 12. The PGROUPN analyses have therefore been carried out using the ground movement profiles computed by FLAC-3D as an input.

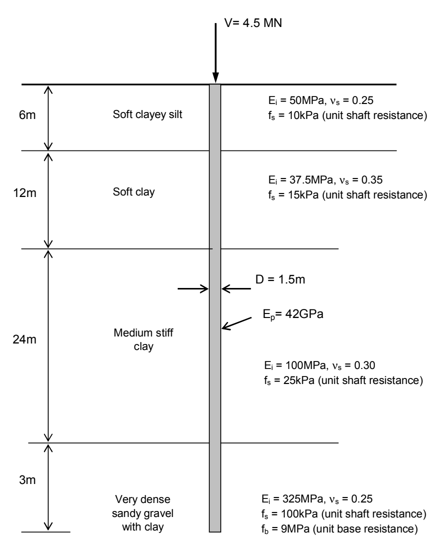

Fig. 9. Soil profile and design parameters used in PGROUPN analysis

Figures 10, 11, and 12 report the profiles of pile shear stress, drag force, and drag settlement, respectively, predicted by PGROUPN and FLAC-3D. A substantial agreement between results is found for all three types of loading conditions, i.e. direct head load (V = 4.5 MN), surface loading (S.L. = 5, 100 kPa), and a combination of V and S.L. Results show that larger surface loads (and thus larger ground settlements) result in a deeper NP location, while the pile drag force and drag settlement are increased, as expected. It is important to note that, even for the relatively small surcharge load S.L. = 5kPa (corresponding to a ground surface settlement of only about 8mm), negative shear stress are developed along a considerable portion of the pile length (the NP is over 30m deep), while the drag force attains a value of nearly 1MN. With increasing surcharge load to S.L. = 100kPa, NSF is fully mobilized for almost the entire pile length and the drag force reaches a value of about 3.5MN, while the pile approaches the end-bearing configuration (i.e. NP location at about 40m depth). In addition, Fig. 13 shows the combined effect of head load (V = 4.5 MN) and surcharge load S.L. = 50kPa (corresponding to the surcharge load of the bridge approach embankment), resulting in a maximum axial force from PGROUPN of 7.4MN with a NP location at 33m depth. Thus, the additional effect of head load is an increased value of the maximum axial force and a reduced depth to NP location, as compared to the case in which only the surcharge load S.L. = 50kPa is acting (refer to Fig. 11).

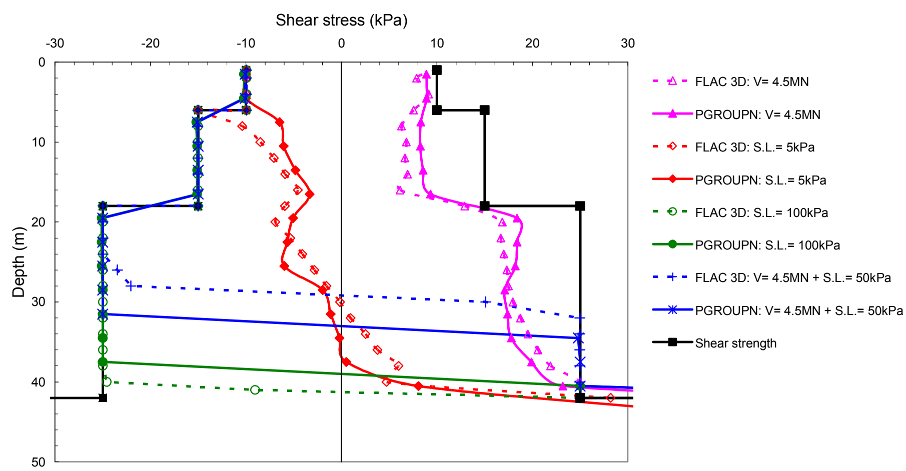

Fig. 10. Shear stress along single pile for various surface loads (S.L.) and/or head load (V)

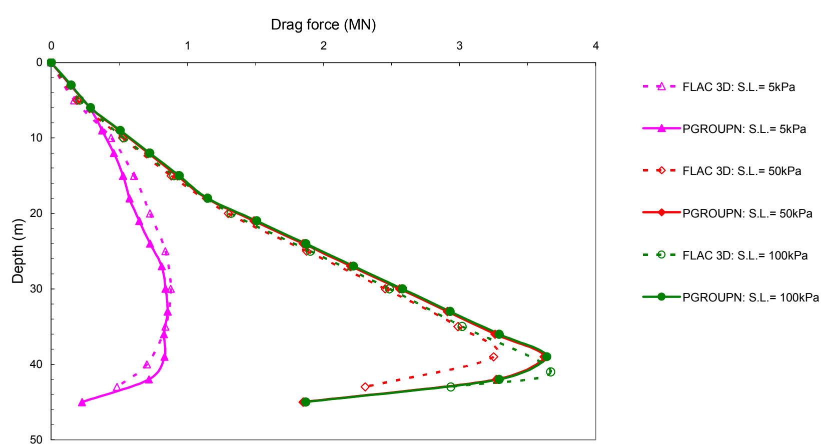

Fig. 11. Drag force along single pile for various surface loads (S.L.)

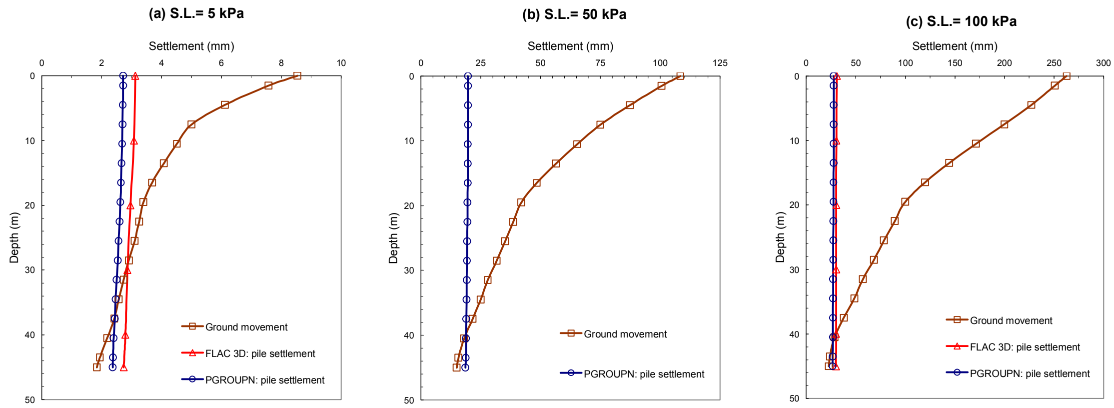

Fig. 12. Pile and ground settlement for various surface loads: (a) 5kPa, (b) 50kPa, (c) 100kPa

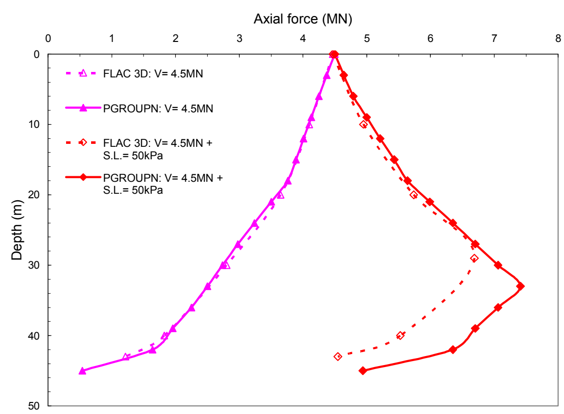

Fig. 13. Axial force along single pile for head load (V) and/or surface load (S.L.)

In order to investigate the effects of group interaction, the single-pile analysis is extended to a fixed-head group of piles in a 3x3 configuration at a pile spacing of 3D (see inset to Fig. 14). The pile cap is assumed to be fully rigid and, owing to the ground settlement, not interacting with the ground. In order to concentrate our attention to the NSF problem only, it is initially assumed that no external load is acting on the pile cap, while the NSF-induced ground movements are those corresponding to a surface load of 50kPa (refer to Fig. 12b).

Figure 14 shows that, in contrast to conventional axial loading at pile-cap level, group effects stemming from pile-to-pile interaction have a beneficial effect in the case of NSF problems, resulting in a reduction of the maximum drag force by 14% in the corner pile and by 31% in the centre pile, as compared to the corresponding single pile. The above results and trend of behaviour from PGROUPN are in good agreement with those predicted by the FLAC-3D analysis. The reduction of the maximum drag force becomes more marked with increasing the number of piles in the group, leading to a 25% reduction on the corner pile, as compared to the single pile, in the case of a 4x4 pile group. It is also noted that, due to the assumption of fully rigid cap, all piles in the group have developed a NP at the same depth (as correctly modelled by PGROUPN), with the NP location moving upward from 39m for the single pile to 36m for the 3x3 pile group. The above features of behaviour therefore offer the prospect of a more economical design for piles in a group given that the reduced maximum drag force (for example that on the corner pile) and the reduced depth of NP location, as compared to a single isolated pile, may be used for the ULS check of pile structural strength. It is also noted that some tensile force is developed at the corner pile near the pile-head, a behavioural feature which has also been reported by several researchers (e.g. Kuwabara and Poulos, 1989, Teh and Wong, 1995; Lee et al, 2009). In practical cases, however, the tensile force will be eliminated by the usually greater effect of the dead load, as numerically illustrated in Fig. 16.

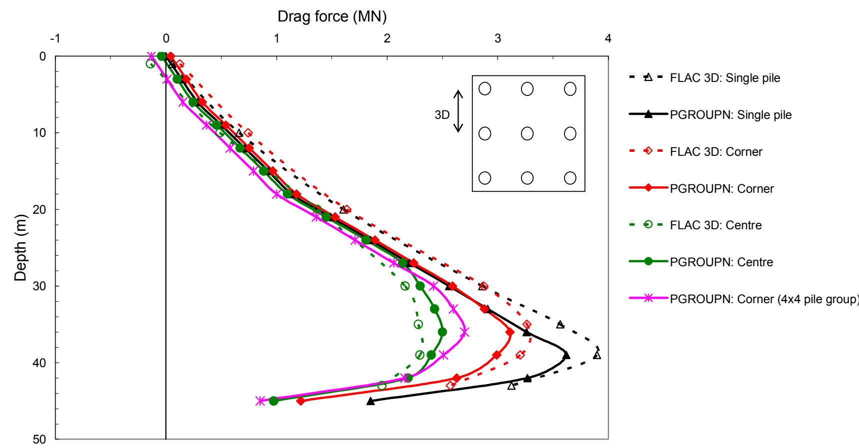

Fig. 14. Drag force distribution for 3x3 pile group with 3D spacing and surface load of 50kPa

Figure 15 shows that the beneficial group effects for piles under NSF decrease with increasing pile spacing (due to the reduced pile-to-pile interaction) and become almost negligible for the group at wider pile spacing (6D), where the reduction in maximum drag force on the corner pile is only 11%, as compared to the single pile.

In order to investigate the more practical case in which both the head load and the NSF are acting, the 3x3 group at 3D spacing is subjected to a total load at pile-cap level of 40.5MN (i.e. an average load of 4.5MN per pile) in addition to the surcharge load S.L. = 50kPa. Figure 16 confirms (even though reduced) the beneficial NSF group effects observed in Fig. 14 in the absence of external cap load, leading to a reduction of the maximum axial force by 4% in the corner pile and by 22% in the centre pile, as compared to the corresponding single pile. Similarly to the single-pile results, the additional effect of external cap load is an increased value of the maximum axial force and a reduced depth to NP location, as compared to the case in which only the surcharge load is acting.

Finally, it should be observed that, even for the larger 4x4 pile group, the computational time required by the PGROUPN analysis is a matter of only a few seconds on an ordinary computer, thereby confirming the value of PGROUPN as a practical numerical tool for the design of pile groups subjected to NSF conditions.

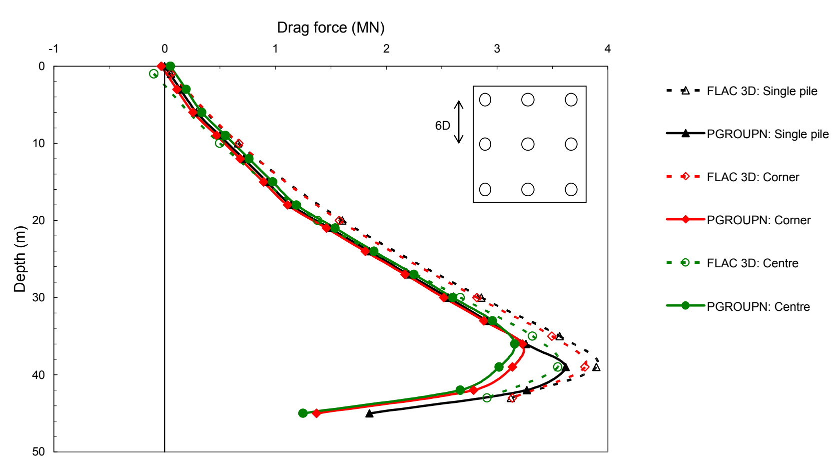

Fig. 15. Drag force distribution for 3x3 pile group with 6D spacing and surface load of 50kPa

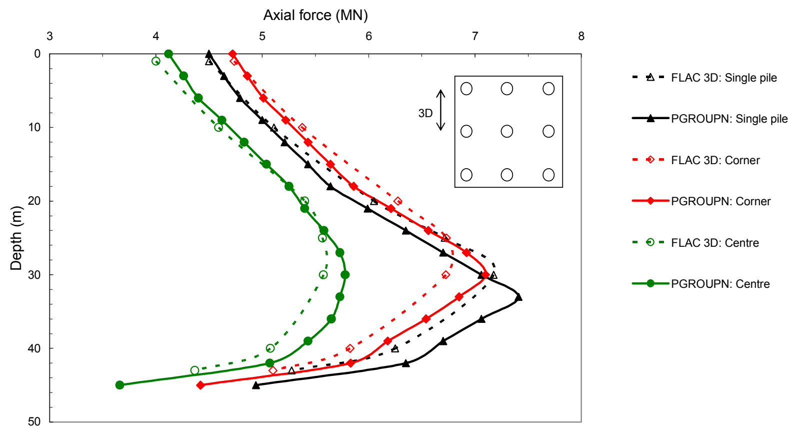

Fig. 16. Axial force in 3x3 pile group under total cap load of 40.5MN and surface load of 50kPa

CONCLUSIONS

Pile design in NSF conditions is mainly a settlement rather than a geotechnical capacity issue, and therefore it is best approached from an SLS viewpoint by means of a pile-soil interaction analysis. For this purpose, this paper has introduced a practical two-stage procedure, based on the 3D non-linear BEM solution implemented within the PGROUPN code, for estimating the response of piles and pile groups subjected to NSF-induced ground movements and to any direct load at pile-head level. Based on the results presented in the paper, a number of considerations may be made:

● The proposed numerical approach is capable of generating reasonable predictions of pile-group response under NSF with negligible computational costs, thus offering a practical compromise between the simplistic full skin friction assumption generally invoked in routine practice and the complexity and time-consuming nature of 3D FEM and FDM solutions.

● PGROUPN is able to model with sufficient accuracy the partial shear stress mobilization corresponding to the actual NSF-induced ground movements, thereby resulting in a reduced pile drag force and drag settlement as compared to the traditional full mobilization assumption. It should again be emphasized that drag force is an *internal* force associated with the development of static equilibrium of the pile-soil system, and therefore it is only a matter for the pile structural strength, i.e. it should not be lumped in with the load from the structure for the assessment of pile geotechnical capacity.

● Consideration of soil nonlinearity (including soil slip at the pile-soil interface) is advantageous in NSF pile design since soil slip can develop under relatively small ground movements, thereby resulting in reduced pile drag forces and drag settlements as compared with a purely linear elastic interface condition.

● In contrast to conventional axial loading at pile-cap level, group effects stemming from interaction among piles have a beneficial effect in the case of NSF problems. As compared with a single pile, there is a tendency for reduced drag forces and a reduced depth of NP location in the group piles, thereby offering the prospect of a more economic and theoretically sound design for piles in NSF conditions.

REFERENCES

Basile, F., 2003. Analysis and design of pile groups. In Numerical Analysis and Modelling in Geomechanics (ed. J.W. Bull), Spon Press, pp. 278-315.

Basile, F., 2012. Pile-group response under seismic loading. Proc. 2nd Int. Conf. on Performance-Based Design in Earthquake Geot. Eng., Paper No. 11.01, Taormina, Italy, pp 1185-1200.

Basile, F., 2014. Effects of tunnelling on pile foundations. Soils and Foundations, Vol. 54, No. 3, pp. 280-295.

Bond, A.J. and Basile, F., 2017. Repute 2.5, Software for pile design and analysis. Reference Manual, Geocentrix Ltd, Surrey, UK, 52 p.

Comodromos, E.M. and Bareka, S.V., 2005. Evaluation of negative skin friction effects in pile foundations using 3D nonlinear analysis. Computers and Geotechnics, Vol. 32, pp. 210-221.

Fellenius, B.H., 2017. Basics of foundation design – a textbook. Pile Buck International, Inc., Vero Beach, FL, Electronic Edition, www.fellenius.net, 464 p.

Kuwabara, F. and Poulos, H.G., 1989. Downdrag forces in groups of piles, ASCE J. Geot. Eng. Div., Vol. 115, No. GT6, pp. 806-818.

Lee, C.J., Ng, C.W.W., and Jeong, S.S., 2009. The effect of negative skin friction on piles and pile groups. In Linear and Non-linear Numerical Analysis of Foundations (ed. J.W. Bull), Spon Press, pp. 181-230.

Mindlin, R.D., 1936. Force at a point in the interior of a semi-infinite solid. Physics, Vol. 7, pp. 195-202.

Mroueh, H. and Shahrour, I., 2002. Three–dimensional finite element analysis of the interaction between tunnelling and pile foundations. Int. J. Numer. Anal. Meth. Geomech., Vol. 26, pp. 217-230.

Poulos, H.G., 2006. Ground movements – A hidden source of loading on deep foundations. John Mitchell Lecture, Proc.10th DFI Int. Conf. on Piling and Deep Foundations, Amsterdam, pp. 2-19.

Poulos, H.G., 2008. A practical design approach for piles with negative friction. Proceedings of the Institution of Civil Engineers, UK, Vol. 161, No. 1, pp. 19-27.

Poulos, H.G., 2009. Program CLAP (Combined Load Analysis of Piles). Users Manual, v. 6.2, Coffey Geotechnics Pty. Ltd, 45 p.

Poulos, H.G., 2017. Tall Building Foundation Design. CRC Press, 532p.

Tan, S.A. and Fellenius, B.H., 2016. Negative skin friction pile concepts with soil-structure interaction. ICE Geotechnical Research Journal, UK. Paper 16.00006. pp. 1-11.

Teh, C.I. and Wong, K.S., 1995. Analysis of downdrag on pile groups, Géotechnique, Vol. 45, No. 2, pp. 191-207.

Yamashita, K., Tomono, M. and Kakurai, M., 1987. A method for estimating immediate settlement of piles and pile groups. Soils and Foundations, Vol. 27, No. 1, pp. 61-76.