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Effects of tunnelling on pile foundations

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Abstract

An efficient analysis method is presented for estimating the effects induced by tunnelling on existing pile foundations. The method is based on a two-stage procedure: (1) an estimate of the free-field ground movements caused by the tunnel excavation, and (2) an analysis of the pile group subjected to the computed free-field ground movements. The first step may be carried out using alternative approaches, ranging from empirical methods to 3D numerical analyses. The second step is performed by PGROUPN, a computer program for pile-group analysis based on a non-linear boundary element solution. The validity of the approach is assessed by comparing it with alternative numerical solutions and field measurements. The results indicate that the method is capable of generating reasonable predictions of pile response for many cases of practical interest, thus offering substantial cost savings over a complete 3D analysis of tunnel–soil–pile interaction.

Keywords: Piles; Pile groups; Tunnels

1. Introduction

Tunnelling in soft grounds inevitably causes ground movements, both vertical and lateral, which may have an impact on existing pile foundations. In such cases, at least two important aspects must be considered by the designer:

(1) The movements of the piles caused by the ground movements in order to ensure structural serviceability;
(2) The additional forces and/or bending moments induced in the piles by the ground movements in order to ensure structural integrity of the piles.

Current analysis methods to evaluate the effects of tunnelling on existing pile foundations belong to two categories:

(a) Simplified two-stage approaches involving the initial separation of the soil and the piles so that the soil movements are first computed and then imposed on the piles;
(b) Complete numerical analyses including simultaneous modelling of the piles, the soil, and the tunnel excavation.

The latter category is generally based on three-dimensional finite element (FEM) or finite difference (FDM) analyses which provide a complete solution to the tunnel–soil–pile interaction (e.g., Mroueh and Shahrour, 2002; Zhang and Zhang, 2013). While such solutions are the most powerful

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numerical tools currently available, they are very expensive in terms of data preparation (pre- and post-processing) and computational time. The cost of such analyses may become prohibitively high if non-linear soil behaviour and complicated construction sequences are to be taken into account. In addition to the computational requirements, complete 3D numerical analyses are complex when use for design purposes, particularly when non-linear behaviour is to be considered. Major difficulties are related to the construction and the interpretation of the 3D model (modelling errors are easily overlooked), the high mesh dependency, the uncertainty in assigning mechanical properties to the pile–soil interface elements, the interaction with adjacent structures, and the modelling of the excavation sequence (e.g., Poulos, 2001; Brinkgreve and Broere, 2003). Thus, a complete 3D analysis is more suitable for obtaining the benchmark solutions (against which simpler analyses can be checked) or for obtaining the final design solution for major projects, than for use as a practical tool for less demanding problems or in the preliminary design stages (in which multiple tunnel configurations and scenarios have to be examined).

In order to overcome the above shortcomings, simplified approaches have emerged (e.g., Chen et al., 1999; Xu and Poulos, 2001; Loganathan et al., 2001; Kitiyodom et al., 2005; Surjadinata et al., 2006). Such approaches are based on a two-stage procedure:

(1) evaluation of the free-field ground movements caused by the tunnel excavation;
(2) analysis of the piles subjected to the computed free-field ground movements.

In simplified approaches, the tunnelling-induced ground movements are generally evaluated in free-field conditions, i.e., in the absence of piles. This generally is a conservative assumption as the presence of piles increases the soil stiffness, thereby reducing the induced ground movements, as demonstrated numerically by Mroueh and Shahrour (2002).

1.1. Estimation of soil movements

Estimation of tunnelling-induced ground movements can be carried out using alternative procedures, namely, empirical methods, analytical expressions, and numerical analyses. Each method has its own strengths and weaknesses.

Empirical methods are based on a Gaussian error function (Peck, 1969; Mair et al., 1996) and are widely employed in engineering practice. The main limitations are related to their applicability to different tunnel geometries, ground conditions, and construction techniques, and in the limited information they provide about horizontal movements and subsurface settlements.

In light of the above limitations, a number of closed-form analytical solutions have been proposed (Sagaseta, 1987; Verruijt and Booker, 1996). In particular, the analytical expressions developed by Loganathan and Poulos (1998) for the estimation of surface settlements, subsurface vertical movements, and subsurface horizontal movements, even though strictly valid for a linear elastic half-space, have the advantage of being able to take into account the various construction methods and the non-linear ground movements (due to an oval-shaped gap) around the tunnel–soil interface. Such expressions allow the rapid estimation of ground deformations by using a simple soil parameter (i.e., the Poisson's ratio), and their applicability has been successfully verified through comparison with a number of case histories.

While empirical and analytical methods provide a simple and practical means of estimating tunnelling-induced ground movements, numerical analyses (generally based on FEM or FDM) provide the most powerful tool for carrying out such predictions because of their ability to consider such factors as ground heterogeneity, soil nonlinearity, advanced soil models, 3D effects, complex tunnel geometries, the interaction with surrounding structures, and the tunnel construction method and sequence. In addition, numerical analyses allow for consideration of the near-field ground response around the tunnel (say in the region within one tunnel diameter) where the effect of factors, such as plastic strain, stress–path dependence, consolidation or the excavation method, becomes prominent. However, even though favourable comparisons with measured ground movements have been reported (e.g., Lee et al., 1994; Surjadinata et al., 2006), finite element models are often known to overpredict the width and to underpredict the gradient of the settlement trough (e.g., Chen et al., 1999; Pound, 2003). To obtain better predictions, it is often necessary to use advanced soil models and to carefully select the corresponding model parameters. Moreover, the designer should bear in mind the complexity and high computational costs involved, particularly if non-linear soil behaviour and 3D effects have to be taken into account.

1.2. Analysis of pile response

The second step of the procedure is usually carried out via a continuum-based or Winkler spring analysis of the piles subjected to the vertical and lateral tunnelling-induced soil movements evaluated using any of the methods described above. Current analysis methods are mainly restricted to purely elastic analyses or to single isolated piles (e.g., Chen et al., 1999; Xu and Poulos, 2001; Kitiyodom et al., 2005). It is indeed generally assumed that the effects of group interaction are beneficial to the pile response as compared to single isolated piles, i.e., group effects lead to a reduction in the deformations, forces, and moments induced in the piles.

2. PGROUPN analysis

The proposed analysis is based on the two-step approach described above and is carried out with PGROUPN (Basile, 2003, 2010), a computer program for pile-group analysis which is commonly adopted in pile design through the software Repute (Geocentrix Ltd., 2012). The main feature of the program lies in its capability to provide a 3D non-linear boundary element

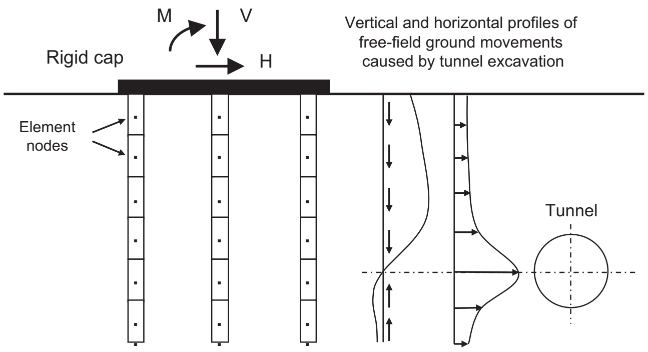

Fig. 1. BEM schematisation of the problem.

(BEM) solution of the soil continuum while incurring negligible computational costs. Use of a non-linear soil model is of basic importance in the design, as it enables the avoidance of the exaggeration of stress at the pile group corners (a common limitation of purely linear models), thereby reducing consequent high loads and moments. Further, compared to FEM or FDM analyses, BEM provides a complete problem solution in terms of boundary values only, specifically at the pile–soil interface. This leads to a drastic reduction in unknowns to be solved, thereby resulting in substantial savings in computing time and data preparation effort.

The analysis involves the discretisation 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 Fig. 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 analysis takes into account the simultaneous influence of all elements of all piles in the group, i.e., the "complete" solution of the soil continuum is adopted. Pile-group effects are therefore evaluated as a matter of course, thereby overcoming the approximations of the traditional interaction factor approach and the fundamental limitations of the Winkler models (based on empirical multipliers to account for group action). In addition, by retaining soil continuity, the input soil parameters required by PGROUPN have a clear physical meaning (e.g., the soil's Young's modulus and strength properties) and can be measured directly in a soil investigation. This aspect represents a significant advantage over Winkler approaches (such as the *t–z* and *p–y* curve methods) which disregard soil continuity, and therefore, have to rely on empirical parameters (e.g., the modulus of subgrade reaction).

The main capabilities of the PGROUPN program are summarised 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;
- non-homogeneous and layered soil profiles;
- linear or non-linear continuum-based soil model;
- general 3D loading conditions, including any combination of vertical, horizontal, moment, and torsional loading;
- output includes the distribution of displacements, stress, forces, and moments along the piles, plus the normal stress, displacements, and rotations of the pile cap.

In this paper, the PGROUPN analysis, originally developed for direct applied loading at the pile cap level, has been extended to deal with externally imposed ground movements which may be acting in the vertical and two orthogonal horizontal directions. The analysis is able to consider the case in which the vertical and horizontal soil movements act together, thereby simultaneously affecting both the axial and the lateral response of the pile group. This feature represents an advance over previous work in which the axial and the lateral pile responses are computed separately. It is noted that the extended PGROUPN analysis may be employed not only in the tunnelling case described herein, but in many circumstances in which pile foundations are subjected to "passive" loadings arising from vertical and horizontal movements of the surrounding ground. Examples include slope movement, excavation, the consolidation of clay, the swelling or shrinking of expansive clay, cavity development, the construction of adjacent piles or buildings, and kinematic effects induced by earthquakes (Basile, 2012).

A detailed description of the theoretical formulation of the PGROUPN analysis for the case of direct applied ("active") loading has been presented elsewhere (Basile, 1999, 2003). The modelling of the pile–soil interaction problem due to "active" and "passive" loading is quite similar, and hence, only a brief outline of the passive case is given below. 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.

2.1. Soil domain

Assuming purely linear elastic soil behaviour, the soil displacements, arising from both the stress caused by the pile–soil interaction and the external source of the ground movements, may be expressed as (e.g., Poulos, 1989)

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

where u_s are the soil displacements, t_s is the soil stress, 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 multi-layered soil profiles are often treated using the averaging procedure adopted by Poulos

(1979, 1990, 2009), i.e., in the evaluation of the influence of one loaded element on another, the value of the soil modulus is taken as the mean of the values at the two elements.

2.2. Pile domain

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

$$\{u_p\} = [G_p]\{t_p\} \tag{2}$$

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

2.3. 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 stress at the pile–soil interface, using the classical equations for the ultimate skin friction, the end bearing capacity, and the lateral pressure (e.g., Basile, 2003). In addition, non-linear soil behaviour is modelled, in an approximate manner, by assuming that the tangent soil's Young's modulus (E_{tan}) varies with the pile–soil interface stress (t) according to the common hyperbolic stress–strain law:

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

where E_i is the initial soil modulus, R_f is the hyperbolic curve-fitting constant, and t_{lim} is the limiting value of the pile–soil stress. Thus, the soil and the pile equations described above for the linear response are solved incrementally using the modified values of the soil's Young's modulus of Eq. (3) within the soil matrix $[G_s]$, while enforcing the conditions of yield, equilibrium, and compatibility at the pile–soil interface (Poulos, 1989; Basile, 1999).

3. Numerical results

The validity of the approach is verified through a comparison with published results from alternative numerical analyses. Attention will be focused on the effects of group interaction and soil nonlinearity.

3.1. Comparison with Kitiyodom et al. (2005)

The accuracy of the PGROUPN analysis is assessed for the case of an existing single pile adjacent to a tunnel under construction, as illustrated in Fig. 2. The free-field tunnelling-induced soil movement profiles to be input into the pile analysis have been calculated using the analytical expressions of Loganathan and Poulos (1998), as shown in Fig. 3. As expected, the soil movements increase with an increasing

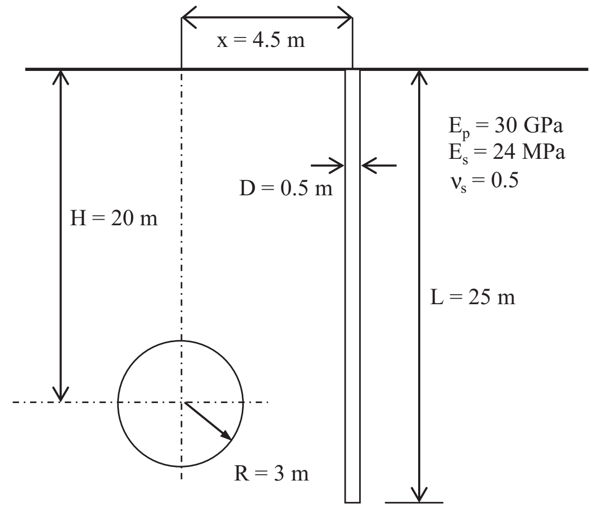

Fig. 2. Single-pile problem analysed.

ground loss ratio (ε) from 1% (a common design value) to 5% (an extreme value for which, in effect, the elastic assumption of the analytical solution is less valid). The vertical ground movements increase gradually with depth to a maximum located near the tunnel crown (at a depth of 17 m). Below this level, the vertical ground movements decrease rapidly while the lateral ground movements become dominant. Below the tunnel invert (at a depth of 23 m), both vertical and lateral ground movements quickly decrease with depth.

Assuming that no direct load (either vertical or horizontal) is applied at the pile-head and treating the soil as an elastic material, Fig. 3 compares the pile response calculated with PGROUPN with that predicted by alternative numerical procedures. The computer program PRAB (Kitiyodom et al., 2005) is based on the hybrid model which combines the Winkler approach for single-pile response with the Mindlin-based BEM analysis to evaluate the pile–soil–pile interaction. A more rigorous boundary element analysis is performed by the code GEPAN (Xu and Poulos, 2001) in which the cylindrical elements at the pile–soil interface are, in turn, divided into partly cylindrical and annular sub-elements. However, both programs are restricted to the linear elastic range.

An excellent agreement between analyses is observed in Fig. 3. The lateral pile deflections are very similar to the soil deflections (reflecting the relatively small lateral stiffness of the pile), with the maximum value occurring just above the tunnel axis level. The bending moment profile has a double curvature, with the maximum also occurring just above the tunnel axis level. The pile settlements are relatively uniform along the entire shaft (reflecting the relatively large axial stiffness of the pile), with the pile-head settlement which is less than the maximum vertical soil movement. Due to the downward vertical soil movements, negative skin friction is induced in the upper portion of the pile, resulting in a compressive axial force (i.e., a drag force) which increases from zero at the pile-head to a maximum just above the tunnel axis level (i.e., at the neutral plane). Below the neutral plane, the axial force

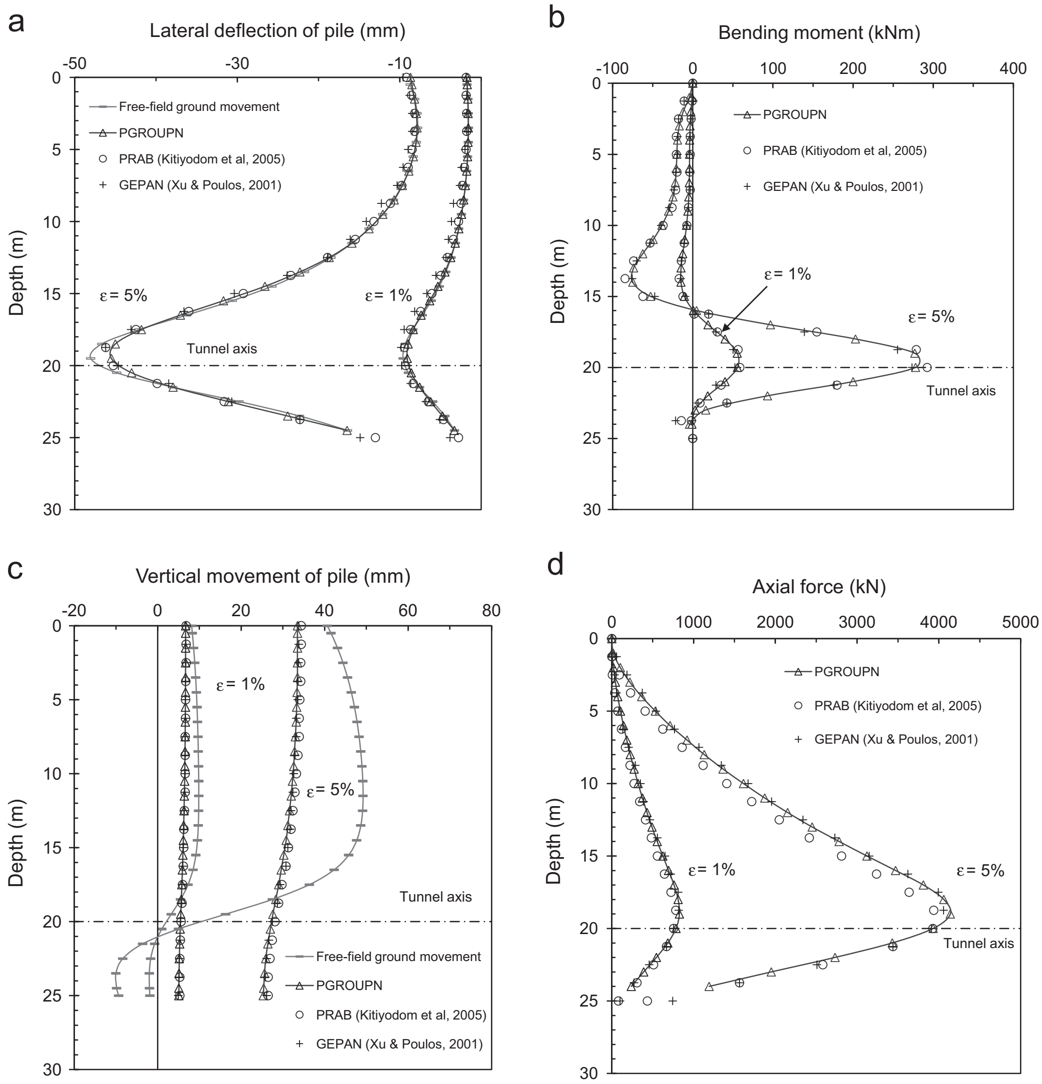

Fig. 3. Comparison of single-pile responses.

gradually decreases by being transferred to the soil by means of positive shaft resistance plus toe resistance. Finally, as expected, larger movements, forces, and moments are developed with an increasing ε, thus emphasising the importance of minimising ground loss during the tunnel excavation.

The effects of group interaction in the linear elastic range are examined for the problem illustrated in Fig. 4, where an existing fixed-head 2×2 pile group is located in proximity of a tunnel under construction. It is assumed that no external load is acting on the pile cap, while the tunnelling-induced ground movements have again been estimated using the solutions of Loganathan and Poulos (1998) for a ground loss ratio of 1%. The pile cap is considered to be fully rigid and free-standing (i.e., no interaction between the cap and the ground is considered), which is a reasonable assumption for this type of problem, as shown by Mroueh and Shahrour (2002).

Figs. 5 and 6 compare the pile-group response obtained from PGROUPN, PRAB, and GEPAN (as reported by Loganathan et al., 2001) for the front and back piles,

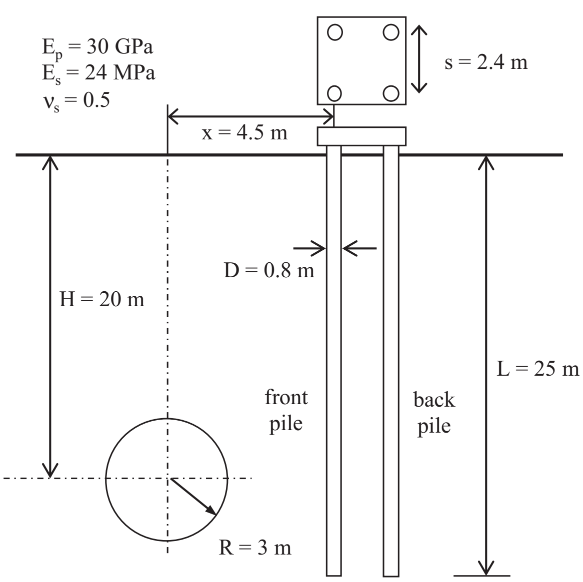

Fig. 4. Pile-group problem analysed.

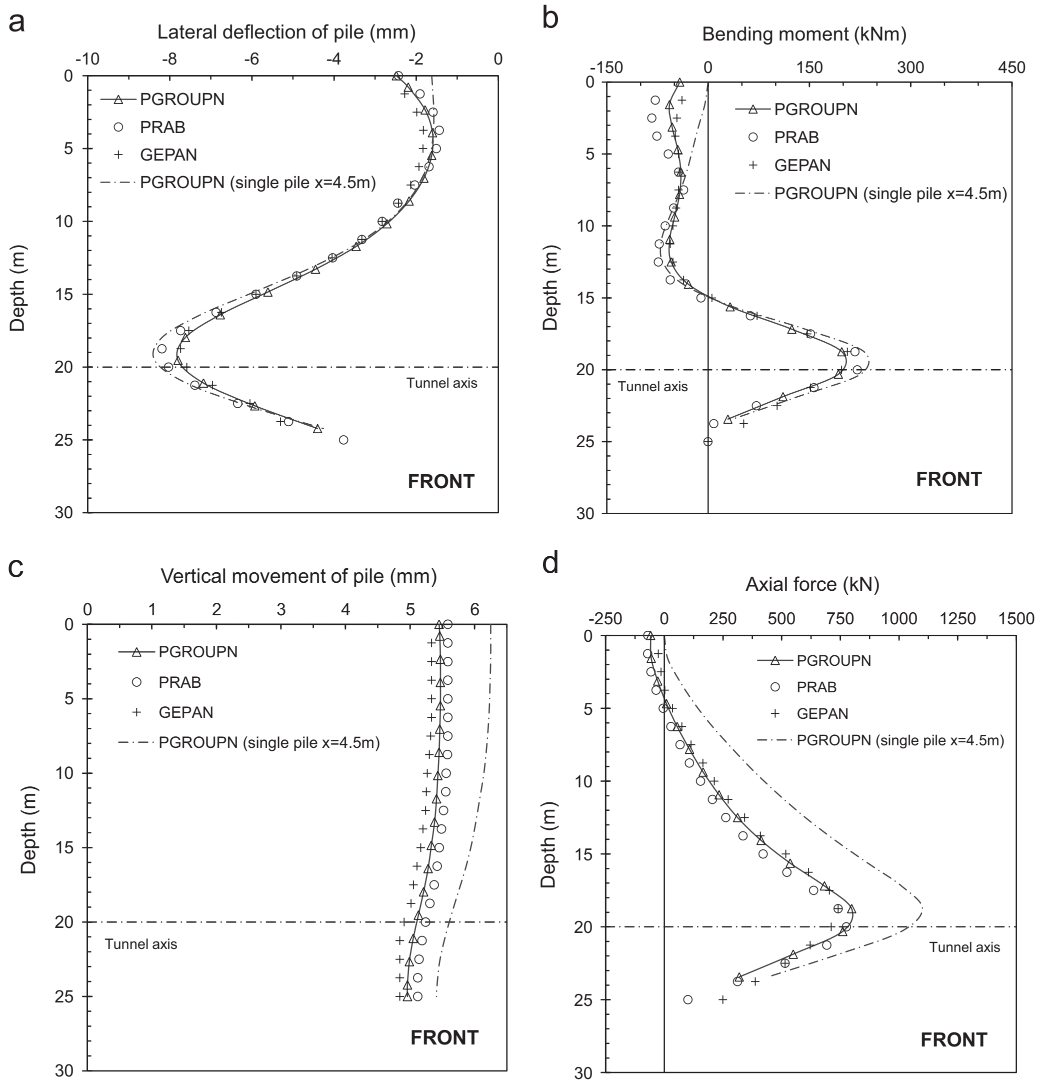

Fig. 5. Comparison of pile-group responses for front pile.

respectively, showing a favourable agreement between elastic analyses. As expected, the deformations, axial force, and bending moment of the front pile (closer to the tunnel) are higher than those of the back pile. In order to compare the pile behaviour both as single isolated piles and within the group, the figures also show the responses of identical single piles located at an equal horizontal distance from the tunnel centreline (i.e., $x=4.5$, 6.9 m). It is observed that the lateral deformation and bending moment profiles for single piles are similar to those of the corresponding piles in the group (except for a difference in bending moment near the pile-head due to the fixity condition). The maximum bending moment occurs in the front pile around the tunnel axis level and is 14% lower than that of the corresponding single pile, whereas the maximum bending moment in the back pile is 23% higher than that of the corresponding single pile. The effects of pile–to–pile interaction appear to be more significant in the evaluation of the pile settlement and the axial force profiles. In particular, the effects of group interaction lead to a reduction in the maximum axial force of 27% in the front pile and 48% in the back pile, as compared to the corresponding single piles (i.e., a beneficial effect). The above results demonstrate the importance of considering the effects of the pile–to–pile interaction in order to obtain a more realistic prediction of pile-group behaviour.

A practical feature of the proposed approach is that the tunnelling-induced ground movements to be input into the pile-group analysis can be calculated using any suitable method. For example, the PGROUPN boundary element analysis may be used in combination with a separate 3D finite element or finite difference analysis, with the free-field ground movements predicted by FEM (or FDM), and with the pile-group response to these ground movements predicted by PGROUPN. This allows the adoption of a more powerful tool for the prediction of the free-field ground movements as compared to the available analytical solutions, thereby enabling consideration of such aspects as the ground heterogeneity, soil nonlinearity, advanced soil models, 3D effects, complex tunnel geometries, and the excavation sequence.

The combined approach therefore provides a practical compromise for many design situations, overcoming the need for a complete 3D FEM/FDM analysis of the tunnel–soil–pile

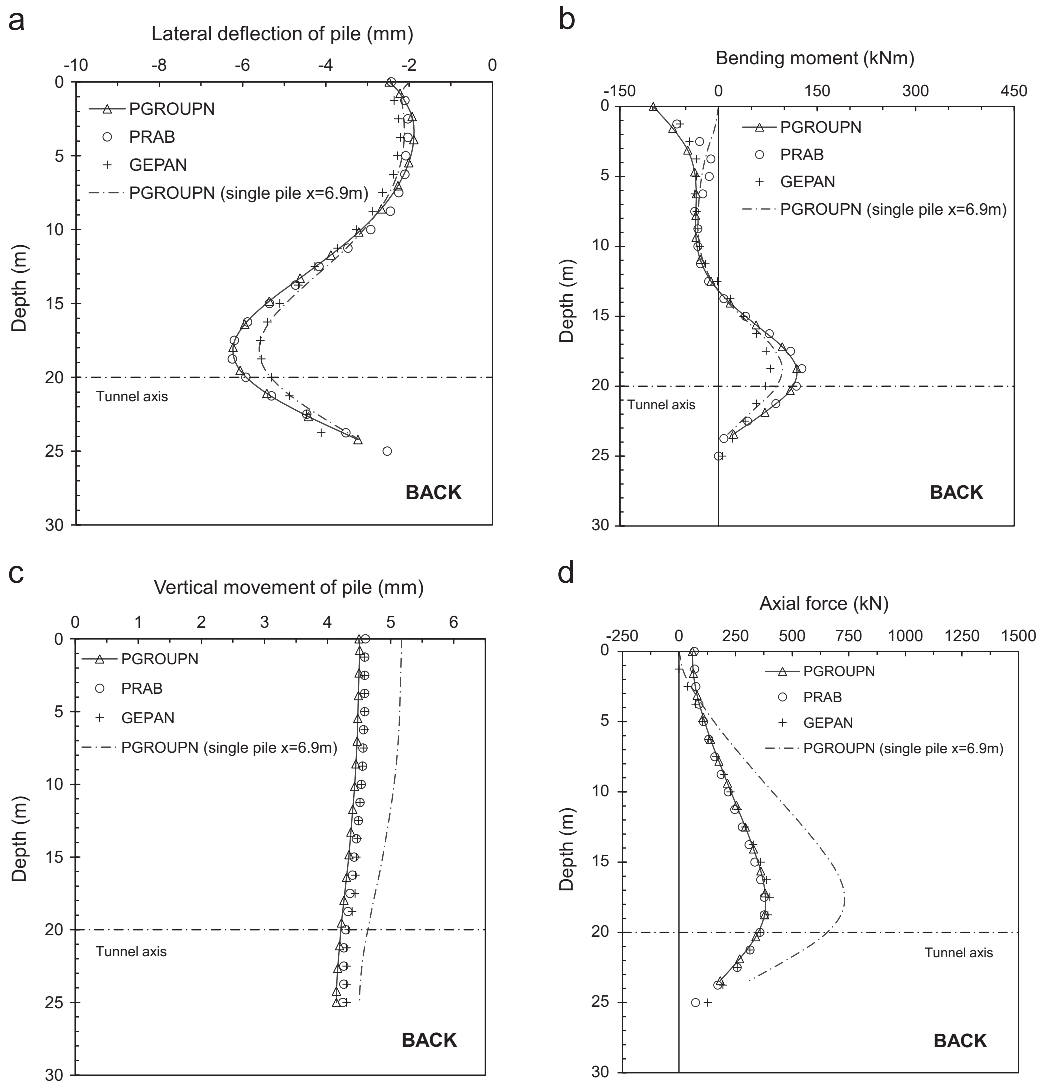

Fig. 6. Comparison of pile-group responses for back pile.

interaction which is limited by high computational costs and complexity, particularly if non-linear soil behaviour and complicated construction sequences are to be taken into account. As reported by Surjadinata et al., (2006), the execution time of a boundary element analysis is a mere fraction (approximately 1/1000th) of the time needed for a complete 3D finite element analysis. A key advantage of the combined approach is that only a single 3D FEM/FDM analysis for each tunnel configuration is required, independent of the multitude of configurations of pile foundations that may be of interest. The free-field tunnelling-induced ground movements, generated by the FEM/FDM analysis, are then input into a number of separate and inexpensive PGROUPN analyses of the existing pile foundations. The combined approach therefore has the potential to generate economical predictions for many practical cases, enabling engineers to investigate many more cases than are viable at present by a complete 3D FEM or FDM analysis and allowing parametric studies to be readily performed.

In order to illustrate an application of the combined approach, the 2×2 pile group in Fig. 4 has been analysed using the free-field tunnelling-induced ground movements computed by the finite difference program FLAC3D (Itasca, 2002) for a computed ground loss ratio of 4.69%, as reported in Fig. 7. The analysis is carried out under the assumption of linear elastic soil behaviour (even though, in reality, due to the high ground loss ratio, some soil nonlinearity is expected). The ground movements calculated by the analytical expressions of Loganathan and Poulos (1998) are also shown in the figure; these are used as input for the PGROUPN and PRAB analyses.

Fig. 8 compares the front-pile response obtained from PGROUPN, PRAB, and a complete FLAC3D analysis of the tunnel-soil-pile problem, as reported by Kitiyodom et al., (2005). It is observed that, although the shape of the deformation, axial force, and bending moment profiles are similar, the maximum values predicted by PGROUPN (and PRAB) are higher than those predicted by FLAC3D. Similar differences were reported by Loganathan et al., (2001) based on a comparison between FLAC3D and a boundary element analysis (similar to PGROUPN). As noted by Loganathan and colleagues, the differences in predictions likely arise from two sources, i.e., (1) the different modelling of the pile–soil

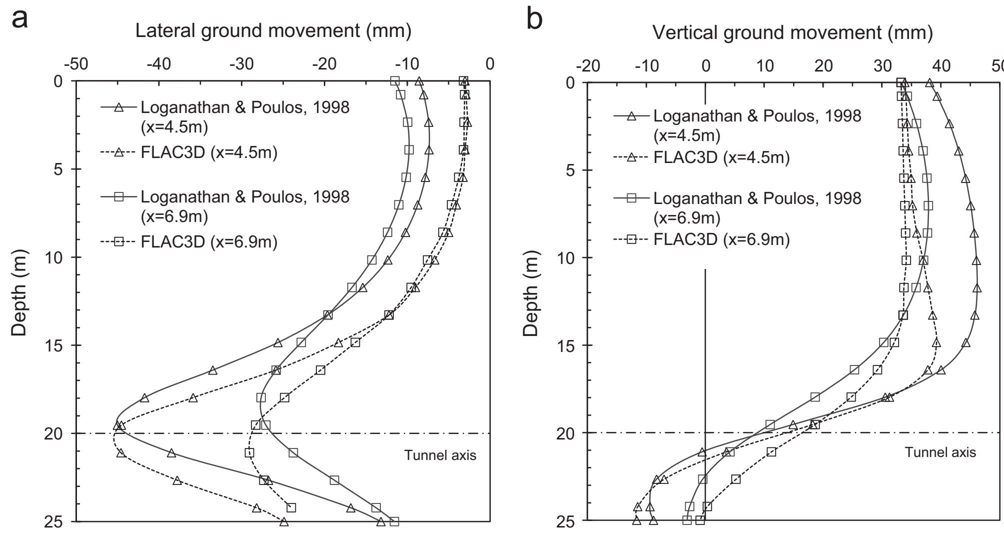

Fig. 7. Comparison of computed free-field ground movements.

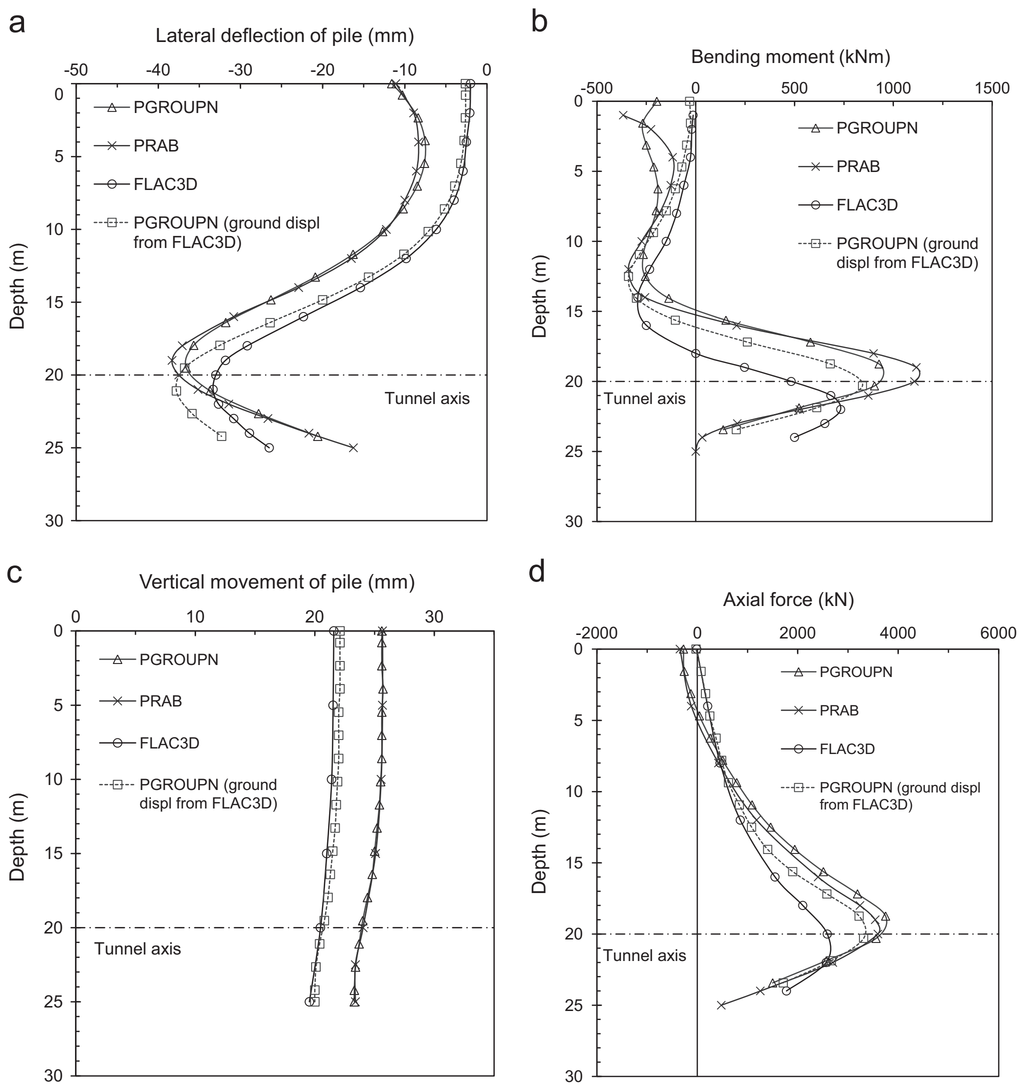

Fig. 8. Comparison of pile-group responses for front pile.

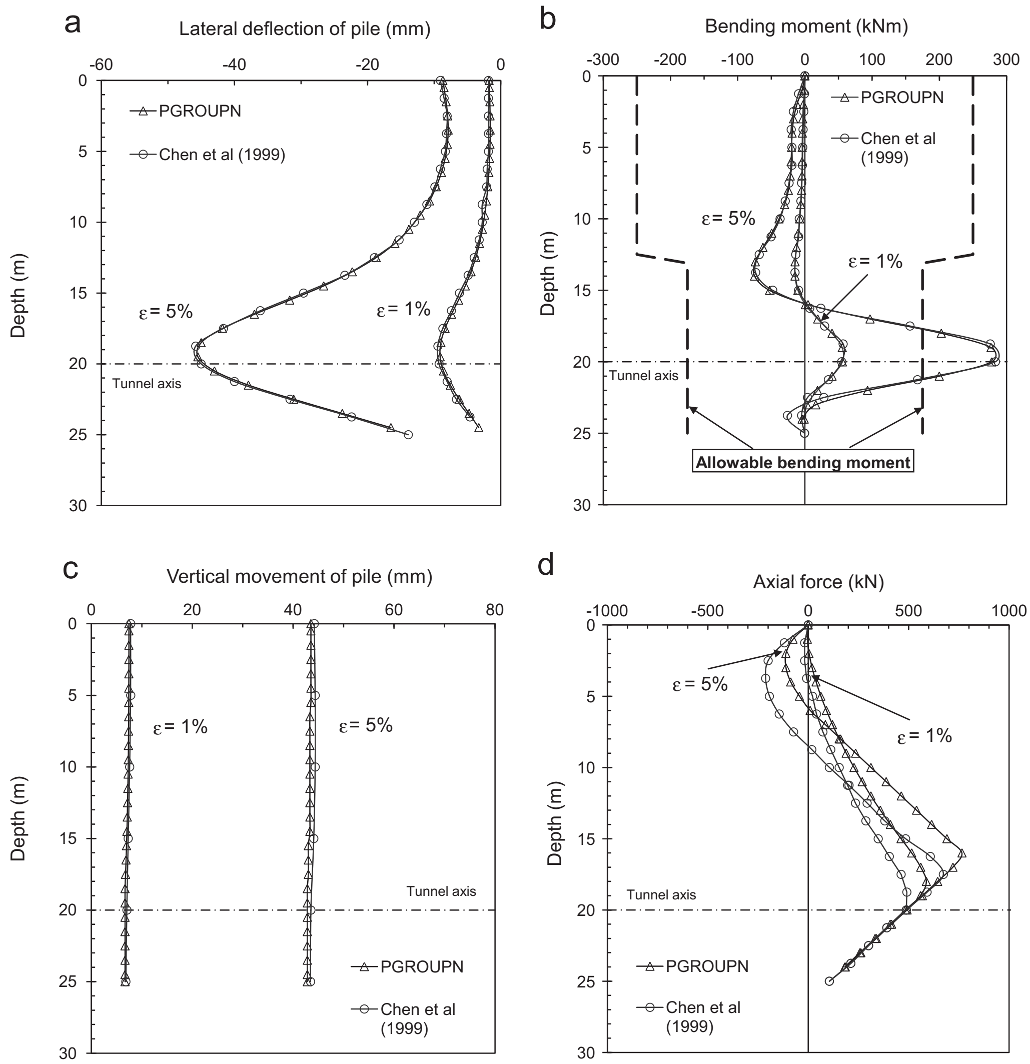

Fig. 9. Comparison of single-pile responses.

interface (this is modelled in FLAC3D by means of empirical spring constants), and (2) the larger free-field ground movements predicted by Loganathan and Poulos (1998) as compared to FLAC3D. Indeed, a closer agreement with FLAC3D is obtained using a combined approach in which the free-field ground movements from FLAC3D are input into the PGROUPN analysis.

3.2. Comparison with Chen et al. (1999)

The effects of non-linear soil behaviour at the pile–soil interface are examined for the single-pile problem analysed in the linear elastic range in Section 3.1. The input data and free-field ground movements due to tunnelling are those shown in Figs. 2 and 3, while the additional parameter required for the non-linear analysis is a soil undrained shear strength (Cu) of 60 kPa, resulting in a limiting skin friction of 48 kPa, and limiting end-bearing and lateral pile–soil pressure both equal to 540 kPa. In order to directly compare the results with the boundary element solution of Chen et al. (1999), which adopts an elastic-perfectly plastic interface model, the PGROUPN analyses have been carried out using zero values for R_f in Eq. (3). Such an assumption implies that the effects of soil nonlinearity are exclusively caused by plastic yielding at the pile-soil interface, while the dependence of soil stiffness on the stress level is disregarded.

Fig. 9 shows a favourable agreement between the pile responses predicted by Chen et al., (1999) and PGROUPN for two ground loss ratios (ε) of 1% (a common design value) and 5% (an extreme value). It is crucial to observe the difference in pile response between the non-linear analyses of Fig. 9 and the preceding linear analyses of Fig. 3. The lateral deformation and bending moment profiles are nearly identical for both the linear and the non-linear analyses, thereby indicating that the limiting lateral pile-soil pressure levels were never reached along the pile. However, the effects of soil nonlinearity play a significant role in the evaluation of the axial response (in particular the axial force distribution); indeed, for the case of $\varepsilon = 5\%$, the limiting skin friction is nearly fully mobilised along the pile, thereby causing slippage at the pile–soil interface. The effects of soil nonlinearity lead to a reduction in the maximum axial force of 29% for $\varepsilon = 1\%$ and of 82% for $\varepsilon = 5\%$, as compared to the corresponding linear analysis. If a non-linear soil model with stress-dependent soil stiffness (i.e., with non-zero values of R_f in Eq. 3), such reductions in maximum axial force would become even more significant, specifically 43% for $\varepsilon = 1\%$ and 83% for $\varepsilon = 5\%$. It may also be observed that, in the non-linear analysis, both compressive and tensile axial forces are induced in the pile, with the compressive force in this case being larger. The above results demonstrate the importance of considering non-linear soil behaviour in order to obtain a more realistic and, in this case, economical prediction of the pile behaviour. It is worth noting that, for the 5% ground loss ratio, the maximum pile bending moment exceeds the allowable values estimated by Chen et al., (1999), i.e., 250 kNm in the top half of the pile and 175 kNm in the bottom half. This observation is of particular significance considering that the developed bending moment is solely induced by tunnelling without taking into account that induced by other types of loading (either vertical or horizontal) acting at the pile-head (e.g., from the superstructure).

In order to assess the effects of both pile–to–pile interaction and soil nonlinearity, the single-pile problem analysed in Fig. 9 is extended to the case of a fixed-head 3×3 pile group. The piles have a centre-to-centre spacing of three pile diameters and the tunnelling-induced ground movements have been derived from the analytical solutions of Loganathan and Poulos (1998) for a ground loss ratio of 1%. Fig. 10 reports the PGROUPN predictions of bending moment and axial force for pile 1, i.e., the most heavily loaded pile in the group. The predictions for an identical single pile located at an equal horizontal distance from the tunnel (i.e., $x = 4.5$ m) are also shown for comparison. There are no differences in the bending moment profiles between linear and non-linear analyses for either the single pile or the pile in the group. However, as already noted in the comparison of Fig. 5, group effects lead to a reduction in the maximum bending moment of 15% in pile 1 as compared to the single isolated pile. As previously observed, the effects of soil nonlinearity and group interaction

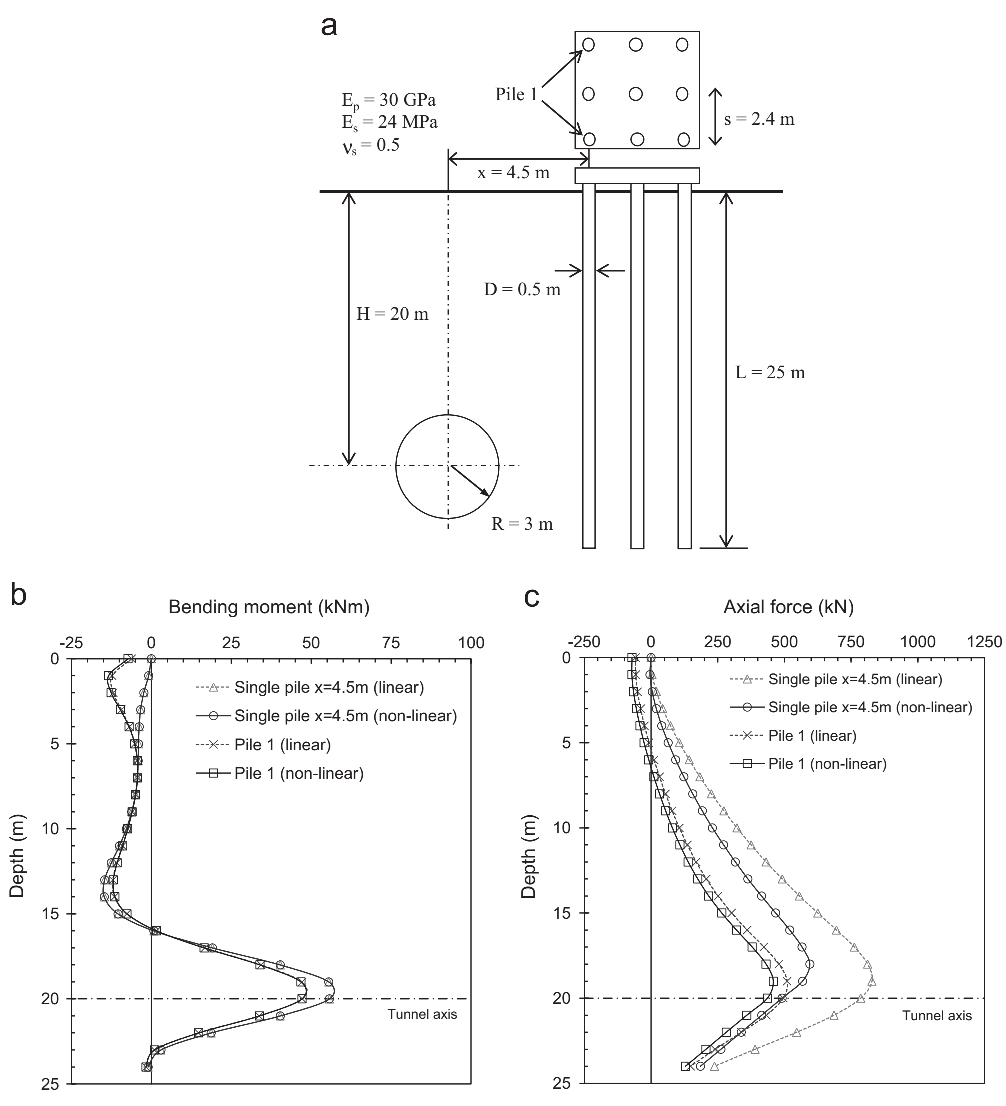

Fig. 10. Pile-group problem analysed and PGROUPN predictions.

play a more significant role in the evaluation of the axial force distribution, leading to a substantial reduction (by about 44%) in the maximum values obtained from a linear analysis of the single isolated pile.

4. Case history

Applicability of the approach in design is assessed through a comparison with a well-documented case history reported by Pang (2006) and relating to a piled-viaduct bridge in Singapore. The bridge, consisting of two abutments and 39 piers, is parallel to twin tunnels that were constructed as part of the Singapore Mass Rapid Transit (MRT) north-east line. Fig. 11 shows the relative position of the tunnels and the piles at one of the piers (Pier 20) supporting the bridge. The bored piles had a diameter of 1.2 m, a length of 62 m, a Young's modulus of 28 GPa, and were arranged in a 2×2 square configuration with centre-to-centre spacing of 3.6 m. Strain gauges were installed in the piles to monitor the axial force and the bending moment that developed in the piles during the tunnelling process. The tunnels were bored using two earth pressure balance machines, 6.5 m in diameter and with the axis of each tunnel being 21 m below the ground. The tunnels were located on either side of the pile-group with the extrados of the south-bound (SB) tunnel being only 1.6 m from the pile edge, while the north-bound tunnel (NB) was 3.3 m from the edge of the adjacent pile.

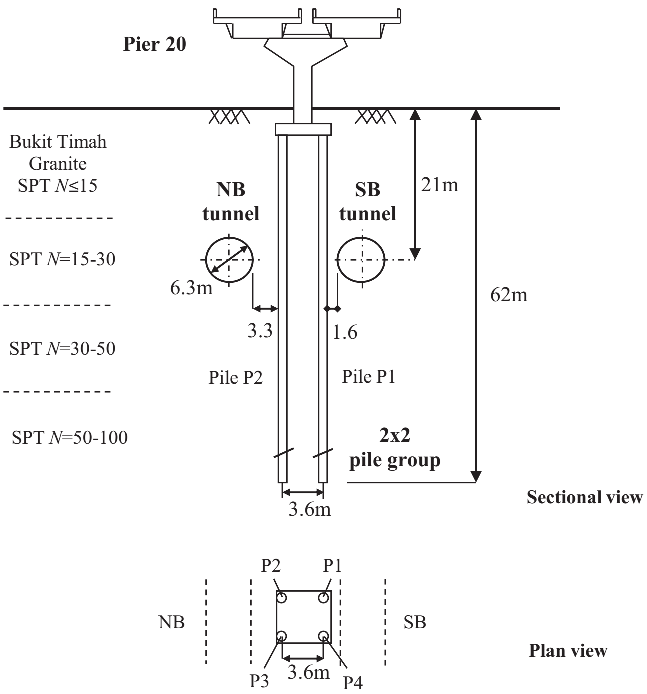

Fig. 11. Pile and tunnel layout at Pier 20 (after Pang, 2006).

The soil profile at Pier 20 consisted of residual soil derived from the Bukit Timah Granite and classified as G4 material. It consisted of reddish-brown sandy silty clay becoming stiffer with depth. The ground model and properties adopted in the PGROUPN analysis follow those employed by Poulos (2011) in a similar boundary element solution, as reported in Table 1. The engineering properties in Table 1 are derived from a correlation with the average SPT N value (whose records are reported by Pang, 2006), i.e., an ultimate shaft friction f_s of

$$f_s = 0.6(2.8N + 10) \text{ kPa} \tag{4}$$

an ultimate end-bearing capacity f_b equal to

$$f_b = 0.1N \text{ MPa} \tag{5}$$

an ultimate lateral pile-soil pressure p_y taken as

$$p_y = 0.1N \text{ MPa} \tag{6}$$

a soil's Young's modulus for axial pile response E_{sv} of

$$E_{sv} = 3N \text{ MPa} \tag{7}$$

a soil's Young's modulus for lateral pile response E_{sh} equal to

$$E_{sh} = 2N \text{ MPa} \tag{8}$$

The pile response due to the first excavated tunnel (i.e., the SB tunnel) is initially assessed and then the study is extended

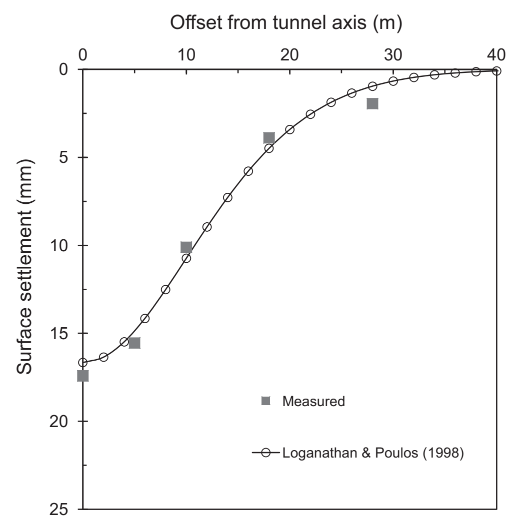

Fig. 12. Measured and computed free-field surface settlements.

Table 1
Summary of ground conditions (after Poulos 2011).

Layer	Thickness (m)	Average SPT, N	f_s (kPa)	f_b (MPa)	p_y (MPa)	E_{sv} (MPa)	E_{sh} (MPa)
1	16	15	31	1.5	1.5	45	30
2	14	25	48	2.5	2.5	75	50
3	14	40	73	4.0	4.0	120	80
4	Large	80	140	8.0	8.0	240	160

to the effects of the twin-tunnel advancement (i.e., the SB and NB tunnels). The surface settlement trough recorded at Pier 20, due to the SB tunnel excavation, is reported in Fig. 12 and shows a fairly good agreement with that calculated using the analytical expressions of Loganathan and Poulos (1998). The measured ground loss was found to be 1.38%. Figs. 13 and 14 show PGROUPN predictions for the profiles of vertical and horizontal movement along piles P1 and P2 of the group, respectively. Also shown are the free-field subsurface ground movement profiles of Loganathan and Poulos (1998) which are used as input in the PGROUPN analysis. The figures also include the results from a complete 3D FEM analysis of tunnel–soil–pile interaction by ABAQUS adopting the rather sophisticated Strain-Dependent Modified Cam Clay model (which requires a large number of parameters) and involving a mesh with 5080 elements and 19,107 nodes, as reported by Pang (2006). For comparison, the PGROUPN analysis only requires a few soil parameters (in this case derived from the SPT data, as shown in Table 1) and adopts a mesh with 316 boundary elements and 316 nodes, thereby resulting in a negligible amount of computing time. A favourable agreement between analyses is observed with a general tendency of the FEM solution to predict larger pile movements than PGROUPN.

Fig. 15 compares the axial force distribution in piles P1 and P2, including the results obtained by Poulos (2011) using a two-stage approach based on a boundary element solution restricted to the single-pile response (i.e., any group effect is ignored) and based on a separate computation of the axial and lateral pile response. A favourable agreement between the boundary element solutions of PGROUPN and Poulos is observed (also reflecting the identical soil properties adopted in the analyses) with the maximum axial forces being quite close to the measured values, while the FEM analysis produces

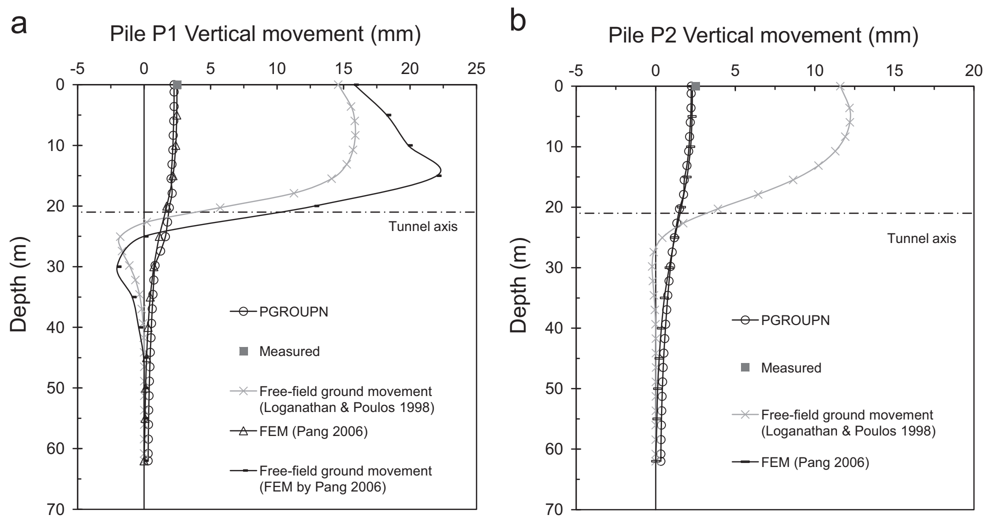

Fig. 13. Vertical movement profiles for (a) pile P1 and (b) pile P2.

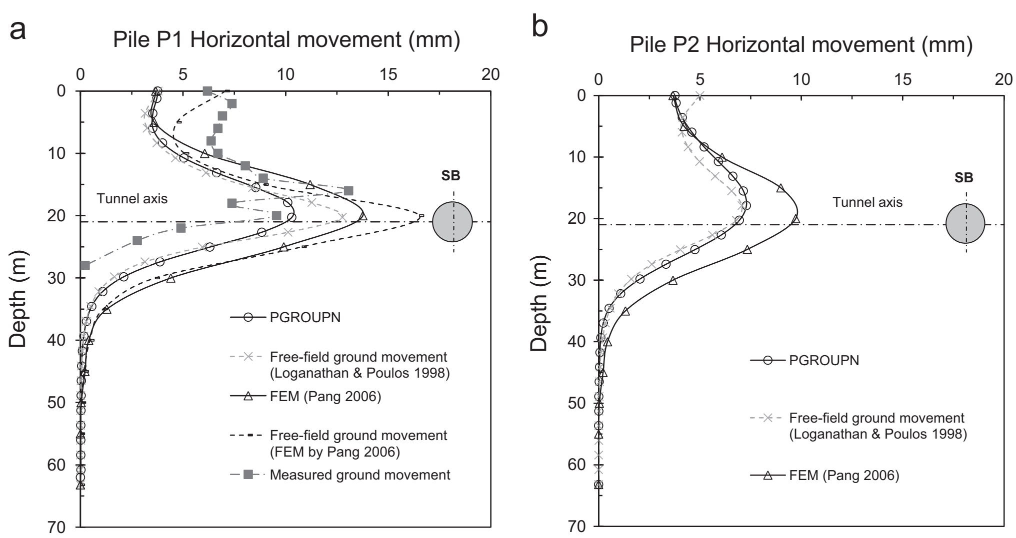

Fig. 14. Horizontal movement profiles for (a) pile P1 and (b) pile P2.

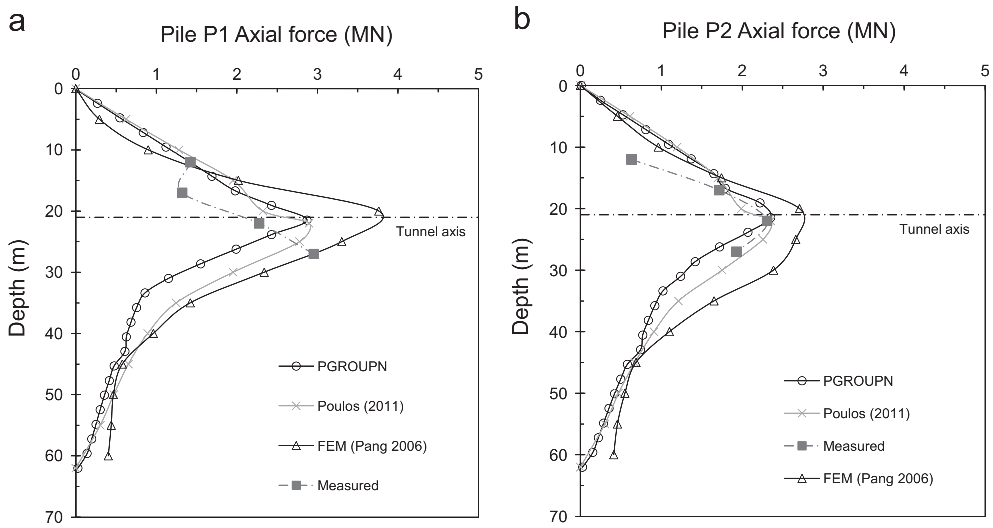

Fig. 15. Measured and computed axial force distributions for (a) pile P1 and (b) pile P2.

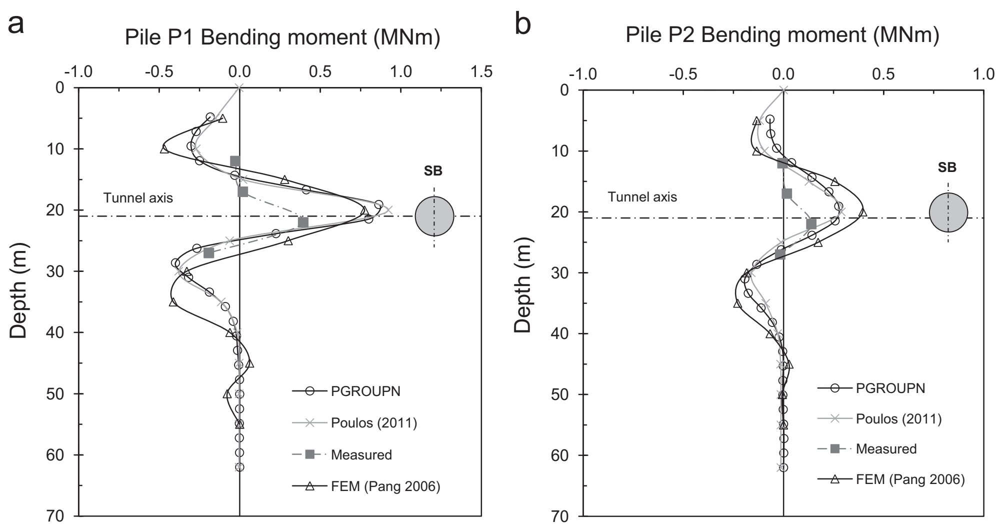

Fig. 16. Measured and computed bending moment distributions for (a) pile P1 and (b) pile P2.

somewhat higher maximum values. These differences likely arise from two sources, i.e. (1) the actual values for ultimate pile shaft friction assumed in the numerical analyses, as pointed out by Poulos, and (2) the larger free-field vertical ground movements predicted by FEM as compared to Loganathan and Poulos (1998), as reported in Fig. 13. As expected, the peak axial force in the front pile, P1, is higher than that in the back pile, P2, due to both the pile–to–pile interaction and the effects of distance from the tunnel. It is also observed that no axial force due to the bridge load is applied to the piles given that construction had only reached the pile-cap level prior to the tunnel advancement.

The bending moment profiles along piles P1 and P2 are compared in Fig. 16. A general consistency between the profiles of the computed and the measured bending moments is observed with a tendency for the numerical analyses to overpredict the maximum values occurring near the tunnel axis. A possible explanation for this overprediction is that the strain gauges were only installed at four levels, and therefore, their location may not have allowed the largest bending moment to be measured, since an extremely rapid rate of change in moment with depth occurs in the vicinity of the tunnel centreline.

The above study is then extended to an analysis of the pile response due to both the SB and the NB tunnel excavations, with the NB tunnel which was constructed on the other side of the pile group (Fig. 11) and passed Pier 20 about 50 days later than the SB tunnel. Measured ground loss for the NB tunnel was found to be 1.67% (i.e., higher than that for the SB tunnel). Figs. 17 and 18 show PGROUPN predictions for the profiles of vertical and horizontal pile movements, respectively, together with the free-field ground movement profiles of Loganathan and Poulos (1998), which are used as input in PGROUPN. The free-field ground movements due to both the SB and the NB tunnels are obtained by superposition. As for the vertical response, Fig. 17 shows that the subsequent NB

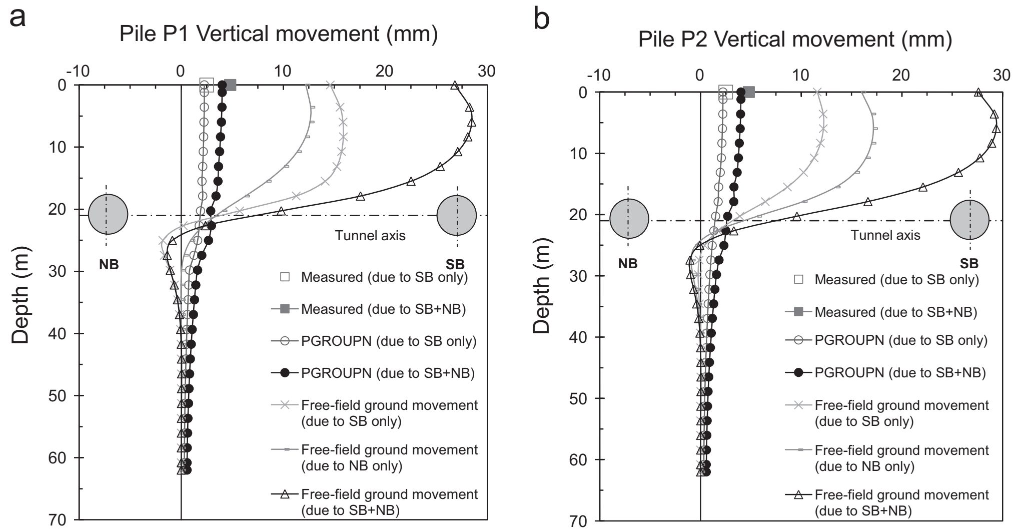

Fig. 17. Vertical movement profiles for (a) pile P1 and (b) pile P2.

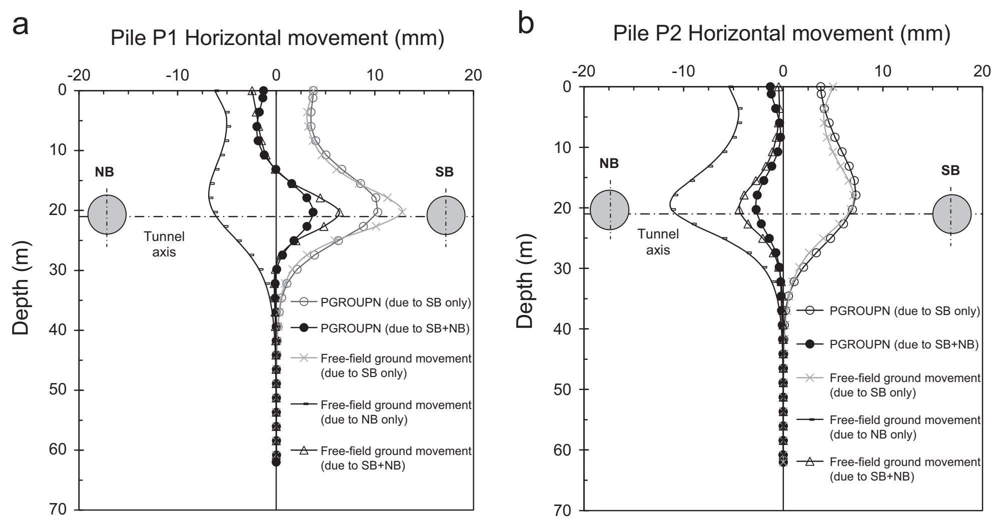

Fig. 18. Horizontal movement profiles for (a) pile P1 and (b) pile P2.

tunnel excavation leads to an increase in ground and pile movements, with a favourable agreement between PGROUPN and measured values at the pile head. As for the horizontal response, Fig. 18 shows that the NB tunnel excavation produces a profile of ground movements in the opposite direction to that produced by the SB tunnel, as expected. This results in a “rebound effect” towards the NB tunnel with a reduction in horizontal movements for pile P1 (the pile nearer to the SB tunnel) and a change in direction of the horizontal movement profile for pile P2 (the pile nearer to the NB tunnel).

Profiles of the pile axial force are reported in Fig. 19 and show a reasonable agreement between the analyses and the measurements. It is observed that the subsequent NB tunnel excavation produces an additional axial force (dragload) on the piles with an increase in the maximum measured values from 3.0 MN to 5.2 MN for pile P1 and from 2.3 MN to 5.7 MN for pile P2. It is worth noting that the maximum values of axial forces recorded for piles P1 and P2, due to both tunnels, correspond to 43% and 47%, respectively, of the allowable pile structural load of 12 MN reported by Pang.

The computed and measured profiles of the bending moments along the piles are reported in Fig. 20. As for the recorded values, pile P1 shows no significant changes in bending moment due to the subsequent NB tunnel excavation, whereas pile P2 (which is nearer to the NB tunnel) shows some rebound effect of bending towards the NB tunnel, especially at the tunnel crown. However, the rebound effect of the bending moments is more significant in the numerical predictions of the PGROUPN and, in particular, the FEM analysis. Nevertheless, there is some consistency between measured and computed values in relation to the moment distribution with depth and the order of magnitude of the developed moments.

Overall, it may be concluded that the simplified two-stage approach, based on the PGROUPN boundary-element solution, is capable of capturing the key features of the measured

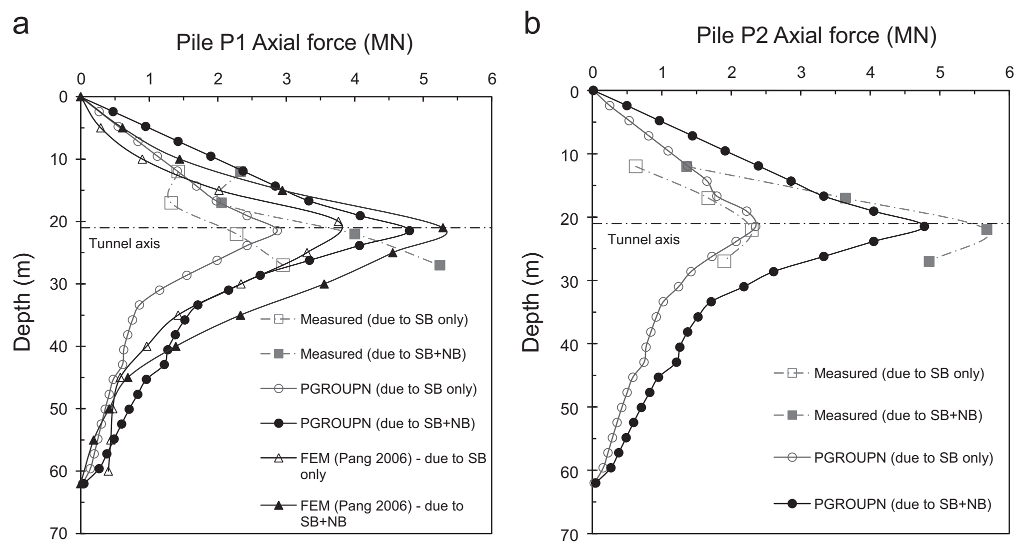

Fig. 19. Measured and computed axial force distributions for (a) pile P1 and (b) pile P2.

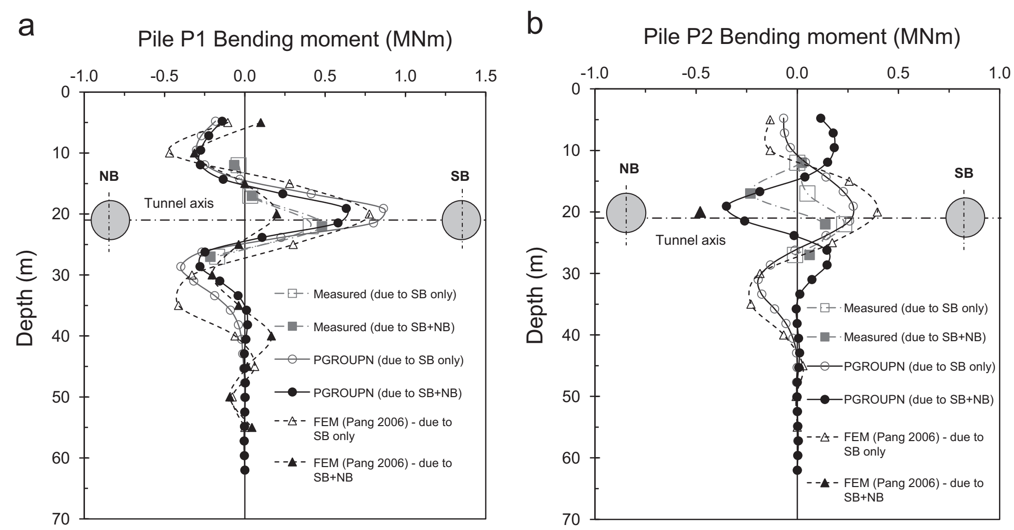

Fig. 20. Measured and computed bending moment distributions for (a) pile P1 and (b) pile P2.

pile behaviour in the field. Predictions of pile response by PGROUPN are also comparable in accuracy to that provided by the complete 3D FEM analysis of the tunnel–soil–pile interaction reported by Pang (2006). As noted by Poulos (2011), it is expected that the discrepancies between the boundary element and the finite element analyses are mainly due to the different ground parameters employed rather than any major differences arising from the two types of analysis methods. The above results lend confidence to the application of simplified two-stage approaches in design, thereby offering a useful means of checking more complete and complex numerical methods of analysis.

5. Conclusions

This paper has presented an efficient and practical two-stage procedure, based on the boundary element method, for estimating the deformations, loads, and moments induced on existing pile foundations during the tunnelling process. A description of the method, its verification, and its applicability have been discussed. Based on the results presented in the paper, a number of considerations may be made:

- The proposed approach is capable of generating reasonable predictions of pile-group responses for many design cases, thus offering a practical compromise between the limitations of simplified analysis methods (mainly restricted to the linear elastic range or to single-pile responses) and the complexity and time-consuming nature of complete 3D analyses of the tunnel–soil–pile interaction. Due to the negligible computational costs (both in terms of data preparation and computer execution time), a large number of cases can be analysed efficiently, enabling parametric studies to be readily performed.

- The effects of group interaction have an influence on the pile-group deformations and the internal force distribution. In particular, the effects of pile–to–pile interaction lead to a reduction (i.e., a beneficial effect) in the maximum values of bending moments and, especially, the axial force in the most heavily loaded pile of the group, as compared to a single isolated pile located at an equal horizontal distance from the tunnel centreline.
- The effects of soil nonlinearity generate a significant reduction in the maximum axial force in the pile, as compared to a linear elastic analysis, thereby allowing more realistic and economical predictions of pile-group responses.
- Two-stage combined approaches, in which the free-field ground movements are predicted by a rigorous FEM or FDM analysis and the pile-group response is predicted by a relatively simple BEM analysis (such as PGROUPN), have the potential to generate economical predictions for many practical cases, thus representing a major savings over the cost of conducting a complete 3D FEM or FDM analysis.

References

Basile, F., 1999. Non-linear analysis of pile groups. Proceedings of Institution of Civil Engineers, Geotechnical Engineering vol. 137 (No. 2), 105–115.

Basile, F., 2003. Analysis and design of pile groups. In: Bull, J.W. (Ed.), Numerical Analysis and Modelling in Geomechanics. Spon Press, pp. 278–315 (Chapter 10).

Basile, F., 2010. Torsional response of pile groups. Proceedings of the 11th DFI & EFFC International Conference on Geotechnical Challenges in Urban Regeneration, London, 13p.

Basile, F., 2012. Pile-group response under seismic loading. In: Proceedings of the 2nd International Conference on Performance-Based Design in Earthquake Geotechnical Engineering, Taormina, Italy, 28–30 May 2012, 16p.

Brinkgreve, R.B.J., Broere, W., 2003. The influence of tunnel boring on foundations and buildings in urban areas – a numerical study. In: Vermeer, PA, Schweiger, HF (Eds.), Geotechnics and Soft Soils, pp. 257–263.

Chen, L.T., Poulos, H.G., Loganathan, N., 1999. Pile responses caused by tunneling. J. Geotech. Geoenviron. Eng., ASCE, 125; 207–215.

Geocentrix Ltd., 2012. Repute 2.0, Software for Pile Design Reference Manual. Geocentrix Ltd., Banstead, UK, 49p.

Itasca, 2002. FLAC3D User's Guide. Itasca Consulting Group.

Kitiyodom, P., Matsumoto, T., Kawaguchi, K., 2005. A simplified analysis method for piled raft foundations subjected to ground movements. Int. J. Numer. Anal. Methods Geomech. 29, 1485–1507.

Lee, R.G., Turner, A.J., Whitworth, L.J., 1994. Deformations caused by tunnelling beneath a piled structure. In: Proceedings of the XIII International Conference ICSMFE. New Delhi, pp. 873–878.

Loganathan, N., Poulos, H.G., 1998. Analytical prediction for tunnelling-induced ground movements in clays. J. Geotech. Geoenviron. Eng., ASCE, 124; 846–856.

Loganathan, N., Poulos, H.G., Xu, K.J., 2001. Ground and pile-group responses due to tunneling. Soils Found. 41 (1), 57–67.

Mair, R.J., Taylor R.N.Burland, J.B., 1996. Prediction of ground movements and assessment of risk of building damage due to bored tunnelling. In: Proceedings of International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground. Mair & Taylor, London, pp. 713–718.

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

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

Pang, C.H., 2006. The effects of tunnel construction on nearby pile foundations. Ph.D. thesis. National University of Singapore, 362p.

Peck, R., 1969. Deep Excavations and Tunneling in Soft Ground, State of the Art Report. In: Proceedings of the 7th International Conference ICSMFE. vol. III, Mexico, pp. 225–281.

Poulos, H.G., 1979. Settlement of single piles in nonhomogeneous soil. J. Geotech. Eng., Am. Soc. Civ. Eng. 105 (GT5), 627–641.

Poulos, H.G., 1989. Pile behaviour – theory and application. In: Proceedings of 29th Rankine Lecture, Géotechnique 39 (3), pp. 365–415.

Poulos, H.G., 1990. User's guide to program DEFPIG – Deformation Analysis of Pile Groups, Revision 6. School of Civil Engineering, University of Sidney, 55p.

Poulos, H.G., 2001. Methods of analysis of piled raft foundations. ISSMGE TC18 Report (46p.).

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

Poulos, H.G., 2011. Comparisons between measured and computed responses of piles adjacent to tunnelling operations. Geotech. Lett. 1 (April–June), 11–15.

Pound, C., 2003. Prediction of damage to buildings and other structures from tunnelling. In: Proceedings of the International Conference Response of buildings to excavation-induced ground movements. CIRIA SP201, UK, pp. 27–34.

Sagaseta, C., 1987. Analysis of undrained soil deformation due to ground loss. Geotechnique 37 (3), 301–320.

Surjadinata, J., Carter, J.P., Hull, T.S., Poulos, H.G., 2006. Analysis of effects of tunnelling on single piles. In: Bakker, K.J. (Ed.), Proceedings of the 5th International Symposium Geotechnical Aspects of Underground Construction in Soft Ground. Taylor and Francis, London, pp. 665–671.

Verruijt, A., Booker, J.R., 1996. Surface settlement due to deformation of a tunnel in an elastic half plane. Geotechnique 46 (4), 753–756.

Xu, K.J., Poulos, H.G., 2001. 3-D Elastic analysis of vertical piles subjected to passive loadings. Comput. Geotech. 28, 349–375.

Zhang, Z., Zhang, M., 2013. Mechanical effects of tunneling on adjacent pipelines based on Galerkin solution and layered transfer matrix solution. Soils Found. 53 (4), 557–568.