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An Analysis Method for the Effects of Tunnelling on Loaded Non-Displacement Piles

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Abstract. In urban areas, the excavation of tunnels beneath pile foundations can be detrimental both for the superstructure and the piles. An analysis method is presented in this paper that models the axial response of non-displacement piles to tunnelling by using a two-stage continuum-based nonlinear elastoplastic approach. This method accounts for the effects of tunnelling-induced ground movements, non-linear soil behaviour, and unloading effects. The approach is used to analyse the relationship between pile head settlement and greenfield ground movements for purely-frictional and floating piles with varying pre-tunnelling safety factor in a uniform ground. Results show that the influence of initial safety factor on the interaction level depth is significant and depends on the distribution of pile shaft capacity. Design charts are given that allow estimation of the tunnelling-induced settlements of non-displacement piles for a linear distribution of greenfield movements along the pile. These charts can also be used for deep-excavations.

Keywords. Settlement; tunnelling; soil-pile interactions; nonlinear analysis.

1. Introduction

When tunnelling near to pile foundations, engineers estimate distortions and damage of the superstructure (e.g. buildings, infrastructure) based on an estimation of tunnelling-induced pile settlements. As a first approximation, by neglecting pile-pile interaction and the superstructure stiffness, a tunnel-single pile interaction is considered while assuming a constant pile head load, P, as shown in Figure 1a.

The pile settlements can be related to greenfield settlements $u_{g,gf}$ using the *interaction level* depth, z_i, at which the pile settlement equals the greenfield soil settlement profile. Note that the interaction level is close (but not identical) to the *neutral level*, which is the depth at which the shaft friction changes from negative to positive.

Previous research investigated qualitatively the relationship between the interaction level z_i, the initial pile safety factor $SF_0 = Q/P$ (where $Q = Q_b + Q_s$ is the ultimate pre-tunnelling load given by the shaft and base capacities) and the subsurface greenfield settlements. The interaction level is also affected by the distribution of the ultimate pile capacity that defines the type of deep foundations: *purely-frictional* (little

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base capacity), *floating* (both shaft and base capacities), and *end-bearing* (little shaft capacity) piles.

A reduced *SF0* as well as soil nonlinearity were found to reduce tunnelling-induced internal forces and to increase settlements, thus increasing z_i, (see, among others, [1-5]). In practice, engineers use design charts that relate pile head settlements to greenfield surface settlements and that depend on the location of the pile tip [6-7], as shown in Figure 1b. Alternatively, an empirical method can be used, in which the pile settlement matches the greenfield movements at (i) the surface, (ii) an intermediate pile length, and (iii) the pile tip (i.e. z_i/L_p=0, 2/3, 1) for (i) purely-frictional , (ii) floating, and (iii) end-bearing piles, respectively [8-9]. This method was also used during Crossrail preliminary design [10]. However, these approaches neglect the influence of SF_0.

Finally, the construction/installation method (displacement and non-displacement/bored piles) could impact the tunnel-pile interaction by influencing the distribution of pre-tunnelling mobilised soil reaction forces. While displacement piles can have, prior to tunnelling, a negative friction along the shaft and a tip reaction greater than the vertical service load ($Q_b>P$), non-displacement piles have a positive shaft friction mobilised and a tip force lower than the external load ($P>Q_b$). This paper is limited to the case of non-displacement piles.

For practitioners, there is a lack of design guidance that can account for all these aspects. Although Korff et al. [11]proposed a framework based on dimensionless groups for non-displacement single piles subjected to deep-excavation settlement profiles (that are decreasing with depth), the tunnelling-scenario is characterised by both increasing and decreasing greenfield profiles with depth, as displayed in Figure 1b.

This paper aims to (a) propose a nonlinear-elastoplastic continuum-based model for tunnel-pile interaction and, (b) investigate the tunnelling-induced settlements of non-displacement purely-frictional and floating piles in terms of dimensionless groups, similar to Korff et al. Design charts are given to quickly estimate the interaction level depth.

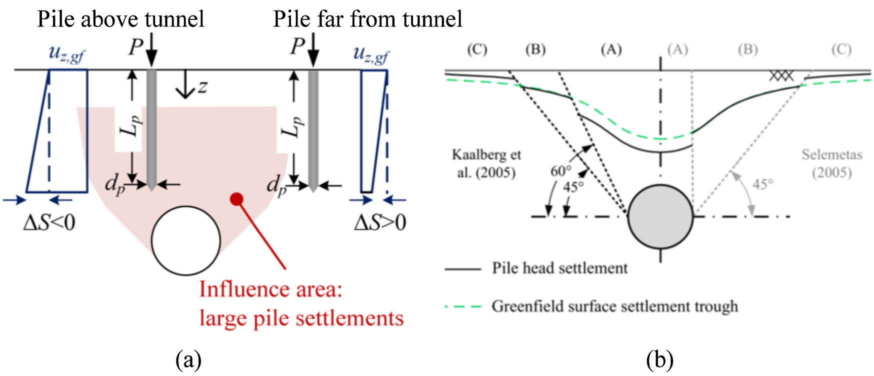

Figure 1. (a) Sketch of the tunnel-pile interaction problem and greenfield ground movements, $u_{g,gf}$. (b) Relationships between pile and greenfield surface settlements depending on the pile tip location [12].

2. Model

In this paper, a nonlinear elastoplastic continuum-based two-stage analysis method is adopted for tunnel-pile interaction. Tunnelling is considered only in terms of induced greenfield ground movements with no effective stress relief while the soil response to loading is not affected by the presence of the tunnel. This is consistent with previous works on tunnel-building and tunnel-pile interaction, which were carried out using boundary element (BEM), finite element (FEM), and finite difference (FDM) methods with either continuum or Winkler mechanical models of the soil (see, among others, [1, 3, 12, 13]).

The soil is modelled as a homogeneous and isotropic half-space (referred to as *continuum*) in which perfectly-plastic behaviour (due to slippage, gap, or soil failure) can occur at the pile-soil interface; soil nonlinearity is assumed to be confined to the area near the pile shaft and tip (referred to as near-pile), while the soil response far from the pile (far-pile; describing pile-soil-raft and pile-soil-pile interactions along the pile itself and between different piles) is assumed linear elastic [14]. In other words, while the response of individual piles is nonlinear, the interaction effects between piles remain largely elastic. This assumption has been shown to be appropriate from field tests [15] and numerical modelling [16].

The initial near-pile and far-pile responses of the soil to loading, which are described, respectively, by the diagonal and off-diagonal terms of the flexibility matrix obtained by integrating Mindlin's solutions along the pile boundary, depend on the soil's elastic parameters (i.e. the soil initial Young's modulus, $E_{s,0}$, and Poisson's ratio, v_s). In future works, the Authors will deal with a half-space that exhibits non-homogeneity with depth (e.g. layered soils).

The near-pile response around the pile is assumed either linear-elastic perfectly-plastic (EP solution) or nonlinear elastoplastic (NEP solution), as illustrated in Figure 2. To account for the perfectly-plastic local soil behaviour, sliders with limit forces are added at the pile-soil interface. The dependency of the soil stiffness on the loading path (i.e. different stiffness for loading and unloading) and the soil stiffness degradation with relative soil-pile displacements are considered. For loading and reverse loading the Young's modulus of the near-pile soil *Es* is decreased according to a hyperbolic law [1] depending on the ratio between local soil reaction forces and the ultimate forces. On the other hand, the local stiffness is assumed equal to the initial Young's modulus $E_{s,0}$ for unloading. Finally, EP and NEP behaviours are only implemented in the vertical direction, whereas a linear elastic response is considered in the horizontal; this assumption is reasonable for tunnelling problems [1].

The proposed FEM model was developed for vertical pile groups of length L_p, diameter d_p, and Young's modulus E modelled as Euler-Bernoulli beams embedded into the continuum. However, this paper is limited to tunnel-single pile interaction, while elevated caps, raft foundations, and the superstructure contributions are neglected by assuming constant pile head loads during tunnelling. All these aspects can be accommodated within the proposed formulation.

Considering the above assumptions, a FEM model was developed starting from the framework of [17-19] by solving the set of equations given in Eqs. (1)− (4). The fully linear elastic solution (EL) is obtained from Eqs. (1) and (2); the elastic perfectly-plastic solution (EP) results from Eqs. (1), (2), and (3); and the nonlinear elastoplastic solution (NEP) is given by Eqs. (1), (3), and (4).

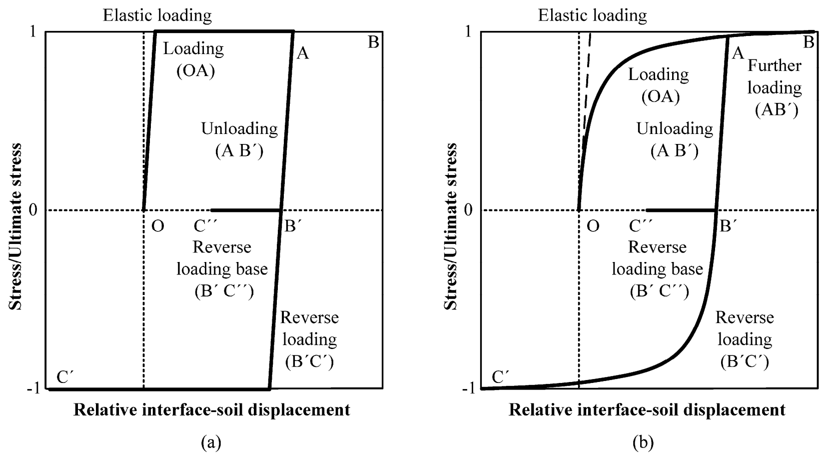

Figure 2. (a) Elastic perfectly-plastic and (b) nonlinear elastoplastic behaviour of the interface and the near-pile soil.

$$(\mathbf{S}+\mathbf{K}^*)\mathbf{u} = \mathbf{p} + \mathbf{K}^*\mathbf{u}^{\mathbf{cat}} + \mathbf{K}^*\boldsymbol{\Lambda}^*\langle\mathbf{f}\rangle + \mathbf{K}^*\mathbf{u}^{\mathbf{ip}}\ ;\ \mathbf{f} = (\mathbf{p} - \mathbf{S}\mathbf{u}) \tag{1}$$

$$\mathbf{K}^* = (\boldsymbol{\Lambda} - \boldsymbol{\Lambda}^*)^{-1} \tag{2}$$

$$\langle\mathbf{f}\rangle_i = f_{i,low} < (\mathbf{p} - \mathbf{S}\mathbf{u})_i < f_{i,up} \tag{3}$$

$$\mathbf{K}^* = \mathbf{R}\,(\boldsymbol{\Lambda} - \boldsymbol{\Lambda}^*)^{-1}$$

$$R_{ii} = \begin{cases} 1 & for\ unloading \\ \left(1 - R_f \dfrac{f_i}{f_{i,low/up}}\right)^2 & for\ loading\ and\ reverse\ loading \end{cases} \tag{4}$$

where **u** is the displacement vector of the pile (consisting of the three translational and three rotational degrees of freedom), **p** is the external loading vector at the pile head, **f** is the vector of forces applied by the foundation nodes to the soil, **S** is the stiffness matrix of the pile foundation, $\mathbf{u}^{ip}$ is the plastic slider displacement vector, $\mathbf{u}^{cat}$ is the greenfield ground displacement vector, $\boldsymbol{\Lambda}$ is the linear elastic soil flexibility matrix relating the soil displacement field to the point of application of a force, $\boldsymbol{\Lambda}^*$ is the nondiagonal term of $\boldsymbol{\Lambda}$ (i.e. soil flexibility matrix without the main diagonal), and $\mathbf{K}^*$ is the local (near-pile) stiffness matrix of the soil (i.e. the inverse matrix of the diagonal term of $\boldsymbol{\Lambda}$ for the linear elastic behaviour in the near-pile soil). $f_{i,up}$ and $f_{i,down}$ are the nodal limit loads in the vertical direction given by the integration of the ultimate base, $q_{b,f}$, and shaft, τ_f, pressures while considering no tensile capacity at the pile tip. **R** is the near-pile stiffness reduction matrix resulting in the initial linear elastic stiffness at unloading and hyperbolic stiffness degradation for loading and reverse loading, which was defined according to previous works [1, 4, 20]. $R_f = 1$ was used in this paper.

The EL equations can be directly solved, whereas EP and NEP require an incremental and iterative procedure. Firstly, the equilibrium equation is solved for incremental variations of the load vector **p** while $\mathbf{u}^{cat} = 0$. Secondly, for a constant **p** load, greenfield settlements $\mathbf{u}^{cat}$ are incrementally applied. Tunnelling-induced effects (movements and forces) are given by the difference between the variables measured at the end of the first and second stage.

3. Model validation

The predictions for a fully linear elastic (or EL) model (representative of perfect soil-structure bonding and linear near-pile soil behaviour) were validated against BEM results for tunnelling adjacent to pile groups connected by a rigid elevated cap [21].

In this paper, the EP model is compared, for tunnelling adjacent to a single pile, against BEM results from the PGROUN program [1]. A pile with L_p= 25m, d_p= 0.5m, and E = 30GPa was affected by the vertical and horizontal ground movements, estimated with semi-analytical formulas [22], induced by a 6m diameter tunnel with a depth to tunnel axis of 20m and a horizontal offset from the pile of 4.5m. For the clay soil, $E_{s,0}$ = 24MPa and ν_s = 0.5 while the ultimate $q_{b,f}$ =540kPa and τ_f = 48kPa. Tunnelling-induced displacements and forces at the pile axis for both axial and flexural effects from the EP analysis method are reported in Figure 3 at low and high tunnel volume losses $V_{l,t}$. The agreement is satisfactory; importantly, by limiting the shaft friction in the EP solution, tunnelling-induced settlements and axial forces increased and decreased significantly with respect to the EL solution, respectively.

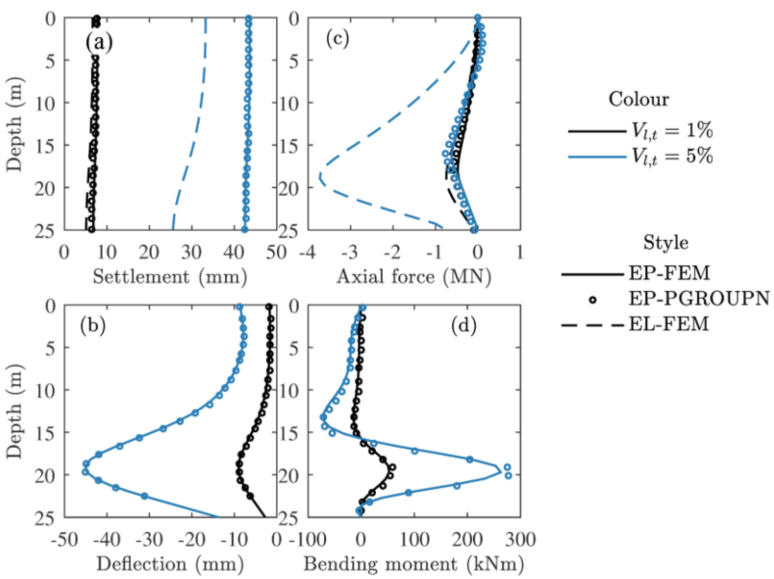

Figure 3. Axial and flexural effects due to tunnelling adjacent to a single pile.

4. Results

To investigate the interaction level, greenfield settlements that linearly increase or decrease with depth z were used (see Figure 1a), as a first approximation for tunnelling beneath the pile tip. The analysis considered a single pile of length L_p = 20m, diameter

$d_p = 0.5$m, and Young's modulus E sufficiently large to simulate a rigid pile in a homogeneous soil with a Young modulus $E_{s,0} = 24$MPa and a Poisson's ratio $v_s = 0.5$. The pile was discretised with finite elements of 1m. Because of the adoption of a rigid pile, the soil stiffness does not impact z_i.

Both purely-frictional and floating piles (labelled *FR* and *FL*, respectively) were considered with $q_{b,f}$ either null or proportional to τ_f at the pile tip. Two possible τ_f profiles along the pile axis were modelled: a constant (e.g. FR.con) and linearly increasing (e.g. FR.inc) profile of τ_f with z. A summary is given in Table 1.

Finally, low and high levels of greenfield movements were considered by analysing $u_{z,gf}$ characterised by a ratio $\Delta S/D_z = -10,-1,1, 10$ where ΔS is the greenfield differential displacement at the pile head and tip (which is defined in Figure 1a), and D_z is relative soil-pile displacement for an elastic-perfectly plastic pile failure, which is defined as the displacement obtained from the pile load-settlement curve $P-u_z$ by the intersection between the ultimate capacity Q and the tangent to the initial linear portion of the curve (see Figure 4). Note that $u_{z,gf}$ is modelled only in terms of ΔS because uniform greenfield settlement profiles result in no pile-soil interaction and a pile settlement equal to the soil value.

Table 1. Considered scenarios.

Label	Shaft limit stresses		Base limit stress	Q_b / Q_s	D_z
FR.con	$\tau_f(z=0) = 60$kPa	$\tau_f(z=L_p) = 60$kPa	0	0%	9.0mm
FR.inc	$\tau_f(z=0) = 30$kP	$\tau_f(z=L_p) = 90$kP	0	0%	9.4mm
FL.con	$\tau_f(z=0) = 60$kPa	$\tau_f(z=L_p) = 60$kPa	$q_{b,f} = 9\times60$kPa	5.6%	9.0mm
FL.inc	$\tau_f(z=0) = 30$kPa	$\tau_f(z=L_p) = 90$kP	$q_{b,f} = 9\times90$kPa	8.4%	9.7mm

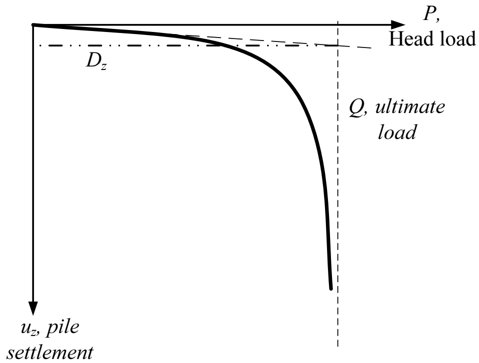

Figure 4. Pile pre-tunnelling load-settlement curve and the definition of D_z.

The normalised interaction level z_i / L_p is plotted against safety factor in Figure 5 in terms of dimensionless groups by following the approach suggested by Korff et al. [11]. Solid and dashed lines are used for decreasing and increasing $u_{z,gf}$ with z, respectively (i.e. solid lines are representative of piles far from the tunnel while dashed lines for piles above the tunnel). Outcomes agree qualitatively with Korff et al. for decreasing $u_{z,gf}$.

Results illustrate that [a] $z_i / L_p = 0.5-0.65$ for both greenfield settlement profiles for high SF_0 (e.g. unloaded piles), [b] z_i / L_p tends to zero (head level) and unity (tip level) for piles far away from and directly above the tunnel, respectively, and [c] the sensitivity of z_i / L_p to the variation of the safety factor is remarkable within the design range of $SF_0 = 1.5-3$. Because the largest values of $u_{z,gf}$ are at the pile tip and head, respectively, for piles within and far from the tunnel influence zone, the statement [b] agrees with centrifuge testing in clays and sands [3, 5] which suggested that the lower the SF_0, the greater the pile settlements regardless of their position.

With respect to the ultimate shaft friction distribution, τ_f linearly increasing with z contributed to the increase in the interaction depth z_i/L_p towards the pile tip. Additionally, the impact of the settlement magnitude $\Delta S/D_z$ is notable only in the case of τ_f increasing with depth. On the other hand, for the considered floating piles FL having $Q_b/Q_s < 10\%$, the base contribution on z_i/L_p was negligible and, thus, results for purely-frictional FR and floating FL piles are similar in Figure 5.

To summarise, adopting the empirical methods that assume $z_i/L_p = 0$ and 2/3 for purely-frictional and floating piles could be misleading and the safety factor should be considered; alternatively, the proposed design chart may be used for a rational framework by tunnelling engineers.

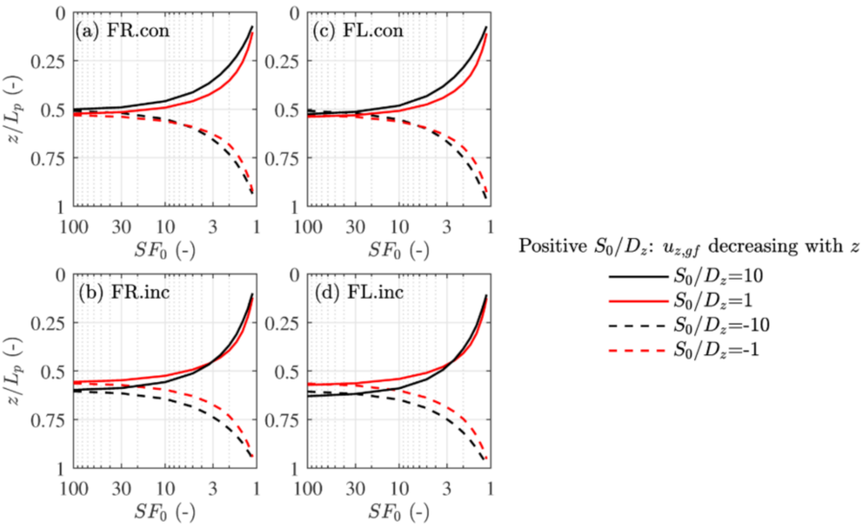

Figure 5. Normalised interaction level depth for varying pile safety factor and greenfield profiles.

5. Conclusions

In this paper, a nonlinear elastoplastic continuum-based model was proposed to study the problem of tunnel-pile interaction. The model was validated against BEM results accounting for elastic perfectly-plastic soil behaviour. Analyses were carried out to evaluate the settlements of purely-frictional and floating piles in a uniform ground at different levels of greenfield ground movements by implementing a nonlinear elastoplastic soil model. For the first time, design charts were given that allow estimation of tunnelling-induced settlements of non-displacement piles for different working conditions and pile capacity distributions (both along the shaft and at the pile base). These charts can also be used for deep-excavations producing similar greenfield displacements. Future works will deal with layered grounds, pile groups, and piled structures.

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