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Research Paper

An analytical approach for the prediction of single pile and pile group behaviour in clay

Brian B. Sheil [a], Bryan A. McCabe [b,*]

[a] Department of Engineering Science, University of Oxford, Parks Road, Oxford OX1 3PJ, UK
[b] College of Engineering and Informatics, National University of Ireland, Galway, University Road, Galway, Ireland

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ABSTRACT

In this paper, the '*t–z* method' is employed to describe the nonlinear behaviour of a single pile and is used to obtain simplified predictions of pile group behaviour by considering the interaction between two-piles in conjunction with the Interaction Factor Method (IFM). The principal inconvenience of the *t–z* method arises from the determination of the resisting curve's shape; an improvement upon this aspect is the main aim of this study. Partial slip is considered using a new analytical approach which is an adaptation of a model based on bond degradation. Pile installation effects and interface strength reduction are uncoupled and considered explicitly in this study. Lateral profiles of mean effective stress after pile installation and subsequent consolidation which were representative of predictions determined in a previous study using a modified version of the cavity expansion method (CEM) are adopted; these predictions are subsequently used to relate installation effects to changes in soil strength and stiffness. In addition, the 'reinforcing' effects of a second, 'receiver', pile on the free-field soil settlement is considered using a nonlinear iterative approach where the relative pile–soil settlement along the pile shaft is related to the soil spring stiffness. Through comparisons with previously published field test data and numerical simulations, the results indicate that the proposed approach provides a sufficiently accurate representation of pile behaviour while conserving considerable computing requirements.

1. Introduction

The topic of single pile and pile group behaviour under vertical load has been covered extensively in the literature over the past few decades. This has lead to the employment of numerous rigorous methods to consider the pile–soil system as one composite continuum including the boundary element method (e.g. [1]) and the finite element (FE) method (e.g. [2]). These numerical methods are regarded as some of the most robust approaches to single pile and pile group analysis and can take into account various complex factors such as soil anisotropy, constitutive modelling, pile–soil–pile interaction and complex 3-dimensional group geometries. A limitation to these continuum analyses, however, is that considerable numerical expertise and computational resources are required for the analysis of the entire pile–soil system.

Simplified analytical approaches have the advantage over rigorous continuum analyses that a designer can obtain an estimate of pile settlement quite quickly and without the need for the computing requirements associated with rigorous continuum analyses. Moreover, they may often prove the only feasible analysis for larger group sizes with non-standard geometries. The load transfer (*t–z*) method was first proposed by Seed and Reese [3] and has since been considered by numerous investigators to describe the linear elastic (LE) load–displacement relationship of a single pile [4–6]. These methods were later advanced by Mylonakis and Gazetas [7] to take account of the 'reinforcing' effects of a pile on the soil continuum for the prediction of two-pile interaction factors.

The concept of considering soil nonlinearity through the use of hyperbolic load-transfer functions, first documented by Kraft et al. [8], has since been adopted by numerous investigators, e.g. [9–13]. Wang et al. [14] improved upon these methods by using an iterative approach to incorporate the degradation in stiffness of the concrete pile under compressive loads using the well-documented nonlinear Hognestad model [15].

While the aforementioned approaches have advanced simplified single pile and pile group analysis, there are various limitations associated with each. One such limitation is the assumption of pre-failure perfect pile–soil bonding. Even at low load levels,

* Corresponding author. Tel.: +353 (0)91 492021.
E-mail address: bryan.mccabe@nuigalway.ie (B.A. McCabe).

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the development of slippage at the contact between pile and soil (defined herein as 'partial slip') is a likely phenomenon [16,17]. Trochanis et al. [18] also reported the importance of pile–soil slip with regard to the load–displacement response of a single vertically loaded pile and two-pile interaction factors. Similarly, Perri [19] noted that one of the main differences between a simple shear test and the soil surrounding an axially loaded pile is the possibility of slip occurring between the pile and the adjacent soil element. Furthermore, while Mylonakis and Gazetas [7] considered pile 'reinforcing' effects in a LE soil medium, a similar approach has not been implemented in recent nonlinear analytical models nor have the effects of soil disturbance arising from pile installation been considered explicitly.

In this paper, the '*t–z* method' is employed to describe the nonlinear behaviour of a single pile and is used to obtain predictions of pile group behaviour by considering the interaction between two-piles in conjunction with the Interaction Factor Method (IFM). The principal inconvenience of the *t–z* method arises from the determination of the resisting curve's shape; an improvement upon this aspect is the main aim of this study. Partial slip is considered using a new simplified approach which is an adaptation of the bond degradation model first proposed by Gens and Nova [20]. Using this method the degradation of bonds, due to particle re-orientation and the remoulding of the structure to eventually a residual state within the clay shear band surrounding the pile, is related to the amount of slip at the pile–soil interface. The 'initial stress technique' [21] is used to consider full pile–soil slip occurring at limiting shear stress.

Although undrained conditions are assumed, in contrast to the α-method of pile capacity design first proposed by Tomlinson [22], pile installation effects and interface strength reduction are uncoupled and considered explicitly in this study. In a previous study conducted by Sheil et al. [23], the FE software package PLAXIS 2-D was adopted (in conjunction with the MIT-S1 constitutive model) to model pile installation effects using a modified cavity expansion method (CEM) in two well-investigated soils, namely San Francisco Young Bay Mud (YBM) and Boston Blue Clay (BBC). The predictions of the permanent change in mean effective stress around an installed pile obtained from that study have been adopted herein to relate long-term stress changes to changes in both the strength and stiffness of the surrounding soil. In addition, the 'reinforcing' effects of a second, 'receiver', pile on the free-field soil settlement is considered using a nonlinear iterative approach where the relative pile–soil settlement along the pile shaft is related to the soil spring stiffness. The present approach is validated against three well-documented case histories.

2. Soil nonlinearity

The relationship proposed by Randolph and Wroth [24] has been employed to predict the reduction of shear stress with radial distance from the pile which can be defined as follows:

$$\tau_{soil} = \tau_i \left(\frac{R}{r}\right) \tag{1}$$

where τ_i is the shear stress in the soil at the pile–soil interface, τ_{soil} is the shear stress in the soil at a radial distance r from the pile's vertical axis of symmetry and R is the pile radius or equivalent pile radius (if non-circular).

The vertical displacements of the soil at a particular point surrounding the pile are then obtained by integrating the shear strains (γ) from that location outwards to a value of $r = r_m$:

$$w = \int_r^{r_m} \gamma dr = \sum_r^{r_m} \gamma \Delta r \tag{2}$$

where $\gamma = \tau/G_{sec}$, G_{sec} is the secant shear modulus and τ is the shear stress. The value of r_m, defined by Randolph and Wroth [24] as the radius at which the shear stress in the soil becomes negligible, was conservatively chosen as 200R. Soil displacements at the pile–soil interface are thus obtained by integrating the shear strains from a distance r_m to a distance R.

For the nonlinear predictions, the relationship proposed by Lee and Salgado [25] was adopted:

$$G_{sec} = G_0\left(1 - f\left(\frac{\tau}{\tau_f}\right)^g\right)\left(\frac{p'}{p'_0}\right)^n \tag{3}$$

where f and g are empirical curve fitting parameters, p' is the mean effective stress which has a far field value of p'_0, n is a constant between 0.5 and 1 and controls the stress dependency of soil stiffness, τ is the shear stress at a particular radial distance, r, from the pile and τ_f is the shear stress at failure. For pile loading in clays, undrained conditions are assumed; therefore the shear stress at failure can be defined as:

$$\tau_f = \frac{s_u}{2} \tag{4}$$

where s_u is the undrained shear strength of the soil.

At the pile–soil interface, however, the shear strength between pile and soil is often substantially less than the shear strength of the soil mass. In this study, reference is made to the databases reported by Potyondy [26] and Tiwari et al. [27] for the selection of an interface strength reduction factor, R_{inter}, such that:

$$s_{u,i} = R_{inter} s_{u,soil} \tag{5}$$

where $s_{u,i}$ is the undrained shear strength at the pile–soil interface and $s_{u,soil}$ is the undrained shear strength of the soil mass. Therefore, the limiting shear stress that can be maintained at the pile–soil interface corresponds to $0.5 * s_{u,i}$.

The base load–displacement relationship is calculated using the hyperbolic model proposed by Guo and Randolph [13] defined as:

$$w_B = \frac{P_B(1-\nu_s)\omega}{4RG_{iB}} \frac{1}{\left(1 - R_{fB}\frac{P_B}{P_{Bu}}\right)^2} \tag{6}$$

where w_B is the pile base settlement; P_B is the pile base load; ν_s is the Poisson's ratio of the soil; G_{iB} is the shear modulus at the pile base; ω is the pile base shape and depth factor which is often set equal to 1 [6,24]; P_{Bu} is the limiting base load; and R_{fB} is a parameter that determines the extent of soil nonlinearity. Assuming undrained conditions, the parameter P_{Bu} can be estimated as follows:

$$P_{Bu} = N_c \cdot s_u \cdot A_b \tag{7}$$

where N_c is the bearing capacity factor, taken as 9, s_u is the undrained shear strength at the pile base and A_b is the area of the pile base.

3. Pile–soil slip

3.1. Shear bands in clay

Numerous studies have reported that for clays, the soil within the shear band undergoes significant particle re-orientation and is eventually remoulded into a residual state [28]. While research on shear bands in sands has identified significant correlations between shear band thicknesses, t_s, and the mean grain size, d_{50}, shear band thicknesses in clays cannot be correlated to grain size so readily [29]. While values of t_s for clays reported in the literature vary from 1.4 to 20 mm, Vardoulakis [30] recommended a conservative value of $t_s = 200d_{50}$; this value gave the best fit to the data presented later in the paper.