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Technical Communication

Effect of geometric nonlinearity on the ultimate lateral resistance of piles in clay

Zihao Zhao[a,b,c], George Kouretzis[c,*], Scott William Sloan[c], Yufeng Gao[a,b]

[a] Key Laboratory of Ministry of Education for Geomechanics and Embankment Engineering, Hohai University, Nanjing 210098, China
[b] Jiangsu Research Center for Geotechnical Engineering Technology, Hohai University, Nanjing 210098, China
[c] ARC Centre of Excellence for Geotechnical Science and Engineering, Faculty of Engineering and Built Environment, The University of Newcastle, Callaghan, NSW 2308, Australia

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ABSTRACT

In this note we investigate how the assumption of small deformations affects the estimation of the ultimate resistance to lateral pile movements in clay. For that we perform coupled hydro-mechanical finite element analyses, using the Modified Cam Clay model to describe soil behaviour. Validation of the numerical methodology via small-strain analyses is followed by an investigation on how the ultimate resistance diverges from exact plasticity solutions when large displacements are considered. Results suggest that the failure mechanism depends on the pile displacement required to mobilise the ultimate resistance, and that small-strain plasticity solutions provide upper bounds of the ultimate resistance.

1. Introduction

Estimation of the ultimate soil resistance developing on piles subjected to lateral loads is a classical problem in foundation engineering. A number of studies deal with determining the distribution of soil resistance to individual pile movements with depth, based on full-scale or 1-g model tests [e.g. 1–3], analytical upper bound limit analysis [e.g. 4,5] and displacement finite element analysis [e.g. 6,7]. It is widely accepted that the limiting lateral resistance per unit pile length p_u increases with depth from an initial low value at the ground surface to a peak value at a certain, critical depth. This peak resistance is commonly expressed in terms of the normalised ultimate resistance factor N_{pu} (ultimate load per unit length normalised with respect to the undrained shear strength s_u and the pile diameter D). A number of researchers [e.g. 8–11] provide solutions for estimating N_{pu}, which are based on a plane strain model of the pile section and its surrounding soil, and assume a two-dimensional flow around failure mechanism. The analytical lower-bound plasticity solution of Randolph and Houlsby [8] provides $N_{pu} = p_u/s_uD$ for arbitrary values of adhesion at the soil-pile interface as:

$$N_{pu} = \pi + 2\arcsin\alpha + 2\cos(arcsin\alpha) + 4\left[\cos\left(\frac{\arcsin\alpha}{2}\right) + \sin\left(\frac{\arcsin\alpha}{2}\right)\right] \quad (1)$$

where α is the adhesion factor, equal to the ratio of the ultimate interface shear stress to the undrained shear strength.

The above plasticity solution is derived while assuming infinitely small deformations and rigid-plastic Tresca soil, and thus ignores the effect of geometric nonlinearity. The effect of this simplifying assumption on the ultimate lateral resistance is the focus of this technical note. In the following we will show that Eq. (1) provides an upper bound on the soil resistance, which decreases as the soil becomes softer and the deformations required to mobilise the ultimate resistance increase. Note that the scope of this study is limited to normally consolidated soils under isotropic initial stresses, therefore it is focused on highlighting geometric nonlinearity effects rather than investigating in detail the effect of soil undrained stiffness on the magnitude of the lateral capacity factor.

2. Material model and properties

The Modified Cam Clay (MCC) model [12] is used in this study to describe the mechanical response of clay while considering different stiffness in loading and unloading-reloading, using the typical parameters listed in Table 1. Although the MCC model is described in detail in many textbooks, certain aspects of the variant of the model implemented in ABAQUS/Standard [13] are repeated in the following, as it degenerates to the standard Cambridge formulation only under specific conditions. The yield surface of the model is given by the following expression [13]:

* Corresponding author.
E-mail addresses: georgios.kouretzis@newcastle.edu.au (G. Kouretzis), scott.sloan@newcastle.edu.au (S.W. Sloan).

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Table 1
MCC parameters.

Soil parameter	Value
Slope of compression line, λ	0.3/varying
Slope of critical state line, M	1.2
Initial void ratio, e_O	3
Poisson's ratio, ν	0.15
Initial isotropic mean stress, p_O' (kPa)	50
Overconsolidation ratio, OCR	1
Permeability, $k_v = k_h$ (m/s)	1×10^{-10}
Slope of recompression/swelling line, κ	0.03/varying

$$\frac{1}{\beta^2}\left(\frac{p}{a}-1\right)^2+\left(\frac{t}{Ma}\right)^2-1=0 \tag{2}$$

where:

$p=-\frac{1}{3}\mathrm{trace}\boldsymbol{\sigma}$ is the mean (effective) stress,

$t=\frac{1}{2}q\left[1+\frac{1}{K}-\left(1-\frac{1}{K}\right)\left(\frac{r}{q}\right)^3\right]$ is the deviatoric stress,

$q=\sqrt{\frac{3}{2}\mathbf{S}:\mathbf{S}}=\sqrt{3J_2}$ is the equivalent stress or von Mises stress and J_2 is the second deviatoric stress invariant,

$r=\left(\frac{9}{2}\mathbf{S}:\mathbf{S}\cdot\mathbf{S}\right)^{\frac{1}{3}}$ is the third stress invariant,

M is the gradient of the critical state line in the p-t plane,

β is a parameter that controls the shape of the yield surface on the wet side of the critical state line in the p-t plane,

α is the size of the yield surface,

K is the ratio of the yield stress in triaxial tension to the yield stress in triaxial compression, and controls the shape of the yield surface in the Π-plane,

In this study we consider $\beta = 1$ and $K = 1$, thus the shape of the yield surface does not depend on the third stress invariant, and its Π-plane section is the original MCC circle.

The incremental stress–strain relation is provided as:

$$\dot{\varepsilon}_v=\begin{cases}\frac{\kappa}{1+e}\frac{\dot{p}'}{p'} & \text{in elastic recompression range}\\ \frac{\lambda}{1+e}\frac{\dot{p}'}{p'} & \text{in elastoplastic compression range}\end{cases} \tag{3}$$

where e is the void ratio. The latter is related to the volumetric strain ε_v as:

$$1+e=(1+e_0)(1-\varepsilon_v) \tag{4}$$

The tangent bulk modulus, K_t can be calculated from Eq. (3) as

$$K_t=\frac{1+e}{\kappa}p' \tag{5}$$

Furthermore, the tangent shear modulus, G_t is calculated by introducing the Poisson's ratio, ν as

$$G_t=\frac{3(1-2\nu)}{2(1+\nu)}K_t \tag{6}$$

In summary, the three parameters κ, λ and ν can be used to describe the undrained stiffness of normally consolidated (OCR = 1) MCC soil with initial void ratio e_O. The effect of soil stiffness on the developing soil resistance to lateral pile movements can be investigated by varying the model parameters κ and λ (Table 1) while keeping the initial state parameters constant, and setting the Poisson's ratio to be equal to $\nu = 0.15$.

The analysis can be further simplified by considering isotropic initial stress conditions. In this case, the undrained shear strength s_u of normally consolidated MCC soil used in the calculation of N_{pu} is given as [14]:

$$s_u=\sigma_v' g(\theta)\cos\theta\left(\frac{1+2K_0^{NC}}{3}\right)\left(\frac{1+B^2}{2}\right)^{1-\frac{\kappa}{\lambda}} \tag{7}$$

For plane strain conditions, as in the problem examined herein, the mobilised undrained strength depends on the shape of the plastic potential in the Π-plane [15]. As we assume associative flow, the Lode angle θ at failure is constant, and depends according to Potts and Gens [15] on the expression used to describe the yield surface in the Π-plane. In the case where the section of the yield surface in the Π-plane is a circle the Lode angle at failure is given as [15]:

$$\tan\theta=\frac{\sin\psi}{\sqrt{3}} \tag{8}$$

where ψ is the dilation angle, therefore failure occurs at $\theta = 0°$. Under these conditions $g(\theta)$ in Eq. (7) is equal to [14]:

$$g(\theta)=M_j=\sqrt{J_2}/p=M/\sqrt{3} \tag{9}$$

Thus for $K_O^{NC} = 1$ adopted in this study:

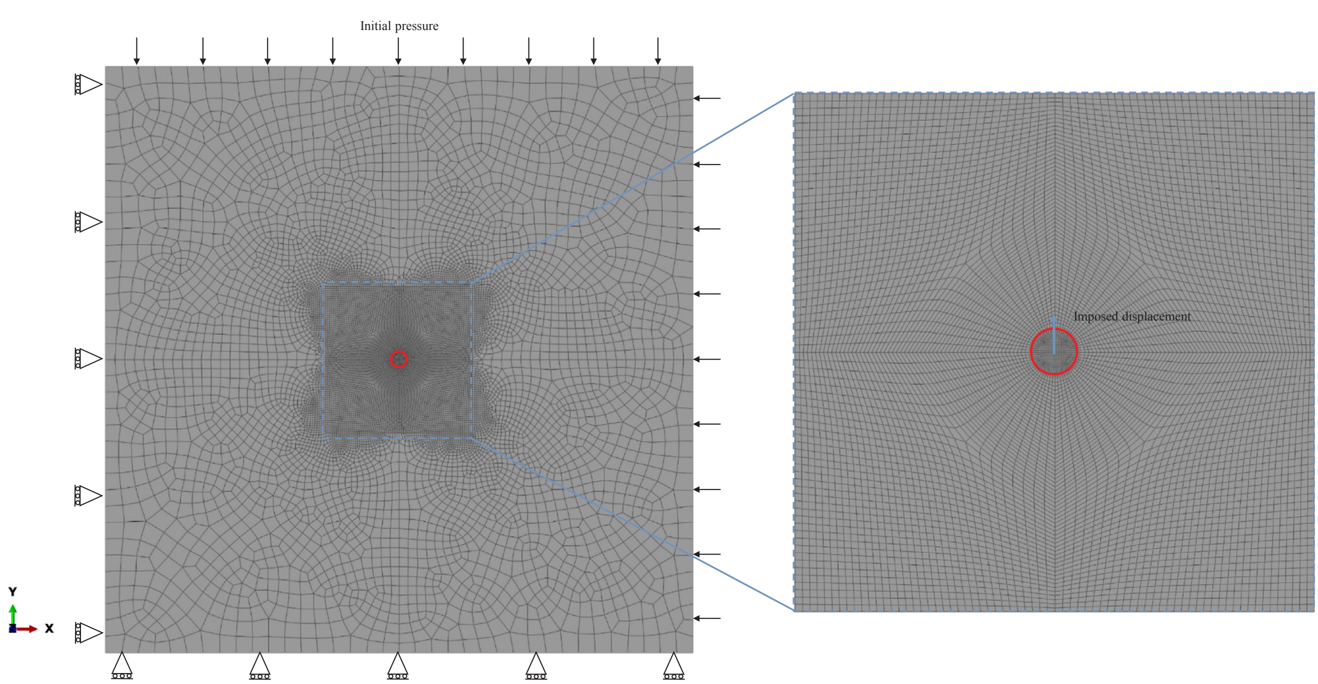

Fig. 1. Finite element mesh and geometry of the two dimensional plane strain model in ABAQUS.