Soil Dynamics and Earthquake Engineering 109 (2018) 282–285

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

Analysis of non-uniform piles driven into cohesive soils

Amin Sormeie, Mahmoud Ghazavi*

Faculty of Civil Engineering, K.N. Toosi University of Technology, Tehran, Iran

ARTICLE INFO

Keywords:
Pile driving
Tapered piles
Cavity expansion theory
Wave equation
Cohesive soils

ABSTRACT

In this paper, a theoretical method based on cavity expansion method (CEM) in conjunction with wave equation theory is used for analysis of driven tapered piles into cohesive soils. The results show that tapered piles are driven easier than cylindrical piles of the same length and volume. Also, with increasing the pile taper angle, the permanent pile displacement increases, which is significant in pile driving.

1. Introduction

It is desirable to drive piles into the ground with sufficient safety, least hammer blows, and minimum driving stresses in the pile. Issacs [1] and discovered that upon the hammer impact, a longitudinal wave propagates along the pile length. Smith [2] developed first one-dimensional numerical solution for pile driving based on wave equation theory.

Coyle and Gibson [3] proposed a non-linear expression for dynamic soil resistance during driving using triaxial impact test on soil specimens. Chow [4] and Smith and Chow [5] presented finite element solution for pile driving. Lee et al. [6] also presented a one-dimensional model based on the wave equation theory for pile driving analysis. Mabsout et al. [7] presented an axisymmetric finite element analysis for pile driving. Ghazavi et al. [8] performed a numerical 1-D finite element analysis for driven tapered piles using the wave equation theory in conjunction with elasto-dynamic theory of Novak et al. [9]. Sakr et al. [10] performed laboratory tests on tapered piles in a large pressurized soil chamber to study the behavior of FRP reinforced concrete driven piles with different taper angles. Ghazavi and Tavasoli [11] performed a three-dimensional numerical finite difference analysis on cylindrical, tapered, and partly tapered piles of the same volume material and lengths.

In this study, a wave equation based theory is presented for analysis of tapered piles during driving into cohesive soils, using the CEM.

2. Analytical method

The relationship between the pile displacement due to hammer impact and the mobilized soil resistance along the pile shaft is derived based on Smith [2] and Lee et al. [6]. For tapered piles subjected to static loading, this relationship was proposed in three phase, by Kodikara and Moore [12]. In the first phase, the pile and the ground are fully connected with sufficient bonding in the elastic range and thus they deform together. In the second phase, the ground still deforms elastically but slip occurs at the pile shaft-soil interface and the pile-soil interface fails in this phase. In the last phase, a plastic zone develops within the soil around the pile and slip occurs at the pile-soil interface. The pile is divided into some tapered segment (Fig. 1).

During pile driving, the soil resistance to the failure increases due to loading rate as introduced by Coyle and Gibson [13]:

$$R_{dyn} = R_{static}(1 + J^{*}\dot{w}^{N}) \tag{1}$$

where J^* is the rate effect coefficient, R_{static} is soil static resistance, N is exponential power less than one, R_{dyn} is enhanced static resistance called dynamic resistance, and $\dot{w}$ is the pile segment velocity during driving.

In phase one, the ground deformation may be predicted as Randolph and Wroth [14]:

$$w_g = \zeta \frac{\tau_x r_m}{G} \tag{2}$$

where τ_x is shear stress, r_m is the mean radius of the pile, G is the soil shear modulus, w_g is the ground deformation and ζ is defined by:

$$\zeta = \ln\left[\frac{2.5L(1-\nu)}{r_m}\right] \tag{3}$$

where ν is the Poisson ratio of the soil and L is the pile length.

In phase one, Eq. (4) is applied.

$$\tau_x = \frac{G}{\zeta r_m} w_p \tag{4}$$

* Corresponding author.
E-mail address: ghazavi_ma@kntu.ac.ir (M. Ghazavi).

https://doi.org/10.1016/j.soildyn.2018.03.014
Received 15 February 2018; Received in revised form 9 March 2018; Accepted 13 March 2018

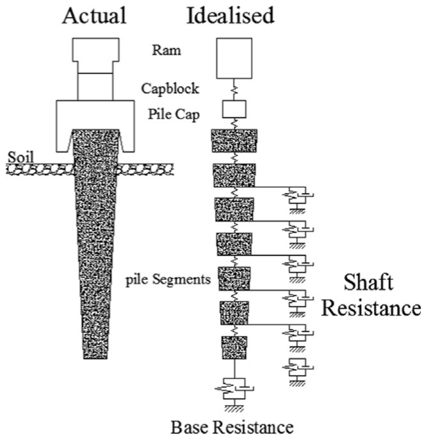

Fig. 1. Tapered pile idealization for pile driving analysis based on the CEM.

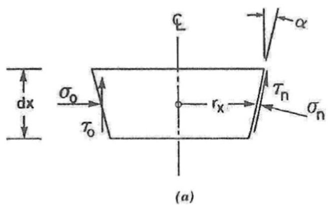

Fig. 2. Pile-ground interface (Kodikara and Moore [14]).

Table 1
Properties of soil and pile used by Mabsout and Tassoulas [19] and Ghazavi and Tavasoli [11].

Material	ρ (kg/m^3)	ν	c (MPa)	ϕ°	E (MPa)
Clayey soil	1600	0.35	2z	0	10
Pile	2400	0.20	–	–	24.86e3

For cohesive soils, this stress state is expressed by using vertical τ_0 component of stresses (Fig. 2) at the interface as:

$$\tau_0 = c_i \sec^2\alpha \tag{5}$$

Considering Coyle and Gibson [13] non-linear power law for the rate effect leads to:

$$\tau_{0d} = \tau_0(1 + J^*\dot{W}^N) \tag{6}$$

where α is the taper angle, τ_{0d} is the dynamic initial yield shear stresses at the interface, c_i is the cohesion at the pile-ground interface.

The pile dynamic deformation, $(w_p)_{Id}$, at this instant can be expressed as:

$$(W_p)_{Id} = \left(\frac{\zeta r_m}{G}\right)\tau_{0d} \tag{7}$$

In phase 2, slip occurs at soil-pile interface and the vertical pile displacement at any arbitrary point on the pile-soil interface is more than the soil vertical displacement. Thus, the radial expansion of the ground is given by:

$$\upsilon = (w_p - w_g)\tan\alpha \tag{8}$$

In phase 2, the increase in radial stress $\Delta\sigma$ due to radial expansion computed from Eq. (8) can be calculated from cylindrical cavity expansion as:

$$\Delta\sigma = K_e\upsilon \tag{9}$$

where $K_e = \frac{2G}{r_m}$

Therefore, the vertical shear stress τ_x acting on the pile-soil interface can be expressed as:

$$\tau_x = (\sigma_0 + \Delta\sigma)\tan(\delta + \alpha) + \frac{c_i\sec^2\alpha}{(1 - \tan\alpha\tan\delta)} \tag{10}$$

Combining Eqs. (2), (8), (9), and (10) gives:

$$\tau_x = \frac{K_e\tan\alpha\tan(\alpha + \delta)w_p + \sigma_0\tan(\alpha + \delta) + c''}{1 + \frac{K_e\zeta r_m}{G}\tan\alpha\tan(\alpha + \delta)} \tag{11}$$

where $c'' = \frac{c_i\sec^2\alpha}{(1 - \tan\alpha\tan\delta)}$

This phase will continue until the ground starts yielding at the pile-ground interface and considering Coyle and Gibson [13] non-linear power law and assuming the Mohr-Coulomb yield criterion may result in:

$$\sigma_{Yd} = \sigma_Y(1 + J^*\dot{w}^N) \tag{12}$$

where $\sigma_Y = \sigma_0(1 + \sin\varphi) + c\cos\varphi$ and characters φ and c are the soil internal friction angle and cohesion, respectively.

The pile dynamic deformation, $(w_p)_{Yd}$, at this instant can be

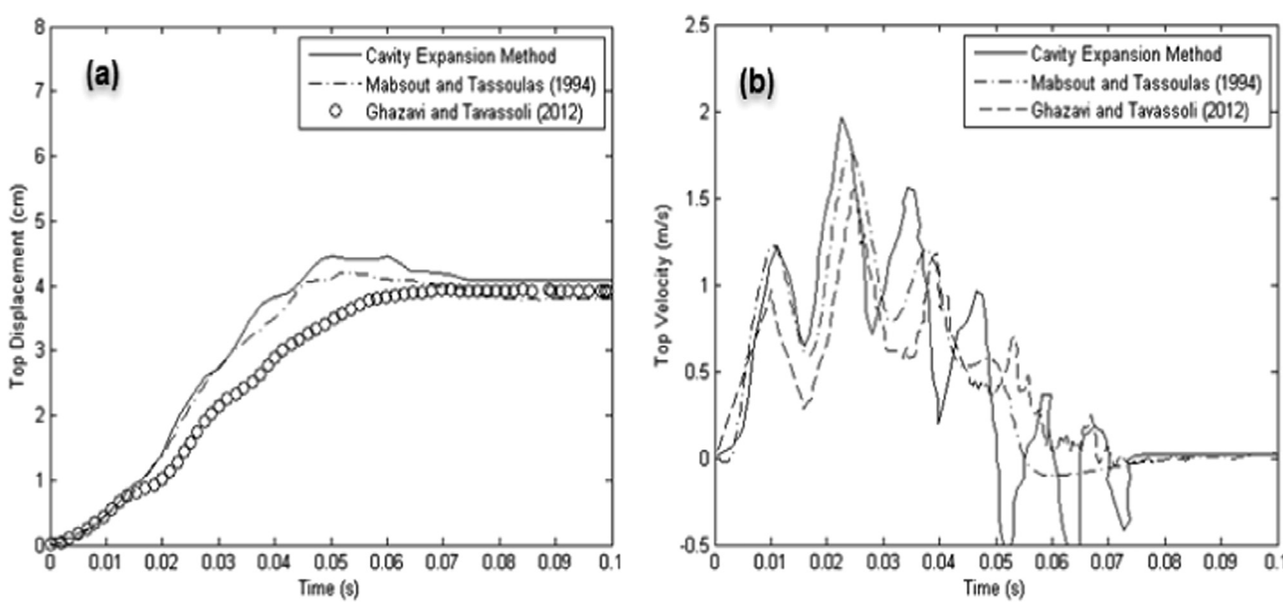

Fig. 3. (a) Variation of pile top displacement versus time for cylindrical pile. (b) Variation of pile top velocity versus time for cylindrical pile.