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Lateral response of pile foundations in liquefiable soils

Asskar Janalizadeh, Ali Zahmatkesh*

Babol University of Technology, Babol, Iran

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ABSTRACT

Liquefaction has been a main cause of damage to civil engineering structures in seismically active areas. The effects of damage of liquefaction on deep foundations are very destructive. Seismic behavior of pile foundations is widely discussed by many researchers for safer and more economic design purposes. This paper presents a pseudo-static method for analysis of piles in liquefiable soil under seismic loads. A free-field site response analysis using three-dimensional (3D) numerical modeling was performed to determine kinematic loads from lateral ground displacements and inertial loads from vibration of the superstructure. The effects of various parameters, such as soil layering, kinematic and inertial forces, boundary condition of pile head and ground slope, on pile response were studied. By comparing the numerical results with the centrifuge test results, it can be concluded that the use of the *p-y* curves with various degradation factors in liquefiable sand gives reasonable results.

1. Introduction

The liquefaction is one of the challenging issues in geotechnical engineering and it damages structures and facilities during earthquakes. This phenomenon was reported as the main cause of damage to pile foundations during the major earthquakes (Kramer, 1996). In many earthquakes around the world, extensive damage to piles of bridges and other structures due to liquefaction and lateral spreading has been observed (Boulanger et al., 2003). Failures were observed in both sloping and level grounds and were often accompanied with settlement and tilting of the superstructure (Adhikari and Bhattacharya, 2008). The loss of soil strength and stiffness due to excess pore pressure in liquefiable soil may develop large bending moments and shear forces in the piles. If the residual strength of the liquefiable soil is less than the static shear stresses caused by a sloping site or a free surface such as a river bank, significant lateral spreading or downslope displacements may occur. The moving soil can exert damaging pressures against the piles, leading to failure (Finn and Fujita, 2002). The performance of structures above piles depends widely on the behavior of pile foundations under earthquake loading. During past earthquakes, because of inadequacy of the pile to sustain large shear forces and bending moments, the extensive damage in liquefiable soil has been caused due to both lateral ground movement and inertial loads transmitted to piles. Under earthquake loading, the performance of piles in liquefied ground is a complex problem due to the effects of progressive buildup of pore water pressures and decrease of stiffness in the saturated soil (Liyanapathirana and Poulos, 2005). These effects involve inertial interaction between structure and pile foundation, significant changes in stiffness and strength of soils due to increase of pore water pressures, large lateral loads on piles, kinematic interaction between piles and soils, nonlinear response of soils to strong earthquake motions, kinematic loads from lateral ground displacements, and inertial loads from vibration of the superstructure (Bradley et al., 2009; Gao et al., 2011).

Various approaches including shaking table and centrifuge tests and also various numerical methods have been developed for the dynamic response analysis of single pile and pile group. The soil–pile–structure interaction has been investigated using the centrifuge test (e.g. Finn and Gohl, 1987; Chang and Kutter, 1989; Liu and Dobry, 1995; Hushmand et al., 1998; Wilson, 1998; Abdoun and Dobry, 2002; Su and Li, 2006) and shaking table test (e.g. Mizuno and Liba, 1982; Yao et al., 2004; Tamura and Tokimatsu, 2005; Han et al., 2007; Gao et al., 2011; Haeri et al., 2012). The obvious advantage of shaking table and centrifuge tests is the ability to obtain detailed measurements of response in a series of tests designed to physically evaluate the importance of varying earthquake characteristics (e.g. level of shaking, frequency content), soil profile characteristics, and/or pile–superstructure characteristics (Wilson, 1998). However, some limitations exist in centrifuge tests, for example, sand grains in centrifuge tests correspond to bigger gravel particles in prototype (Towhata, 2008).

To simulate the piles in liquefiable soil layers, Finn and Fujita (2002), Klar et al. (2004), Oka et al. (2004), Uzuoka et al. (2007),

* Corresponding author. Tel.: +98 9158342367.
E-mail address: A.zahmatkesh@stu.nit.ac.ir (A. Zahmatkesh).
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Cheng and Jeremic (2009), Comodromos et al. (2009), and Rahmani and Pak (2012) used three-dimensional (3D) finite element method. The complexity and time-consuming nature of 3D nonlinear finite element method for dynamic analysis makes it useful only for very large practical projects or research and not feasible for engineering practice. However, it is possible to obtain reasonable solutions for nonlinear response of pile foundations with fewer computations by relaxing some of the boundary conditions in full 3D analysis (Finn and Fujita, 2002).

The simple approach for modeling and simulation of the piles in liquefied grounds is based on scaling of *p*-*y* springs, where *p* and *y* are the soil resistance per unit length of the pile and pile lateral displacement, respectively. Because of complexity and time-consuming of two-dimensional (2D) and 3D numerical modeling, most of the designers and researchers prefer to use one-dimensional (1D) Winkler method based on finite element or finite difference method for the seismic analysis of pile foundations. In pseudo-static method, a static analysis is carried out to obtain the maximum response (deflection, shear force and bending moment) developed in the pile due to seismic loading. In Winkler models, *p*-*y* curves are used to define the behavior of the nonlinear spring at any depth. These *p*-*y* curves can be obtained from the results of model tests or field (Liyanapathirana and Poulos, 2005). The Winkler assumption is that the soil–pile interaction resistance at any depth is related to the pile shaft displacement at that depth only, independent of the interaction resistances above and below (Wilson, 1998).

This pseudo-static method has been suggested early by Miura et al. (1989), Miura and O'Rourke (1991), Liu and Dobry (1995), JRA (1996), AIJ (1998) and recently by Liyanapathirana and Poulos (2005) and Elahi et al. (2010). This method for pile seismic analysis sometimes underestimates, and sometimes overestimates shears, moments and deflection of the piles. However, in many practical conditions, the results of pseudo-static method are reasonable (Tabesh, 1997).

In this paper, a pseudo-static method has been applied for estimation of the response of pile during dynamic loading. First, definition of the geometry and the soil modeling parameters are presented. Next, the numerical model is vertified by means of the centrifuge test. And then the effects of various parameters, including soil layering, kinematic and inertial forces, boundary condition of pile head and ground slope, on the behaviors of piles are studied.

2. Numerical analysis

All simulations were conducted using the open-source computational platform OpenSees (McKenna and Fenves, 2007). This platform allows for developing applications to simulate the performance of structural and geotechnical systems subjected to static and seismic loadings. In this paper, the steps for calculation of pile response are summarized as follows:

(1) A free-field site response analysis was performed during the dynamic loading using 3D numerical modeling. From this analysis, time history of ground surface acceleration and the maximum ground displacement along the length of the pile can be calculated.
(2) The dynamic analysis was performed using the time history of ground surface acceleration calculated in Step 1 for pile length above ground and superstructure with a fixed base. From this analysis, the maximum acceleration of superstructure can be calculated.
(3) In 1D Winkler analysis, the maximum soil displacement profile calculated in Step 1 and the maximum acceleration of superstructure in Step 2 were applied to the pile as shown in Fig. 1.

First, the time history of the ground surface acceleration and the maximum ground displacement at each depth were obtained from the free-field site response analysis. Taboada and Dobry (1993) and Gonzalez et al. (2002) showed that the pore pressure time histories recorded at the same elevation are identical, indicating the 1D behavior of the model. In free-field analysis, the model consists of a single column of 3D brick elements. The soil layers were modeled using cubic 8-noded elements with *u*-*p* formulation in which each node has four degrees of freedom: three for soil skeleton displacements and one for pore water pressure. To consider the effect of the laminar box in the numerical simulation, nodes at the same depths were constrained to have equal displacements in the horizontal and vertical directions. The pore water pressures were allowed to freely develop for all nodes except those at the surface and above the water table. The bottom boundary was assumed fixed in all directions.

The material model plays a key role in the numerical simulation of the dynamic behavior of liquefiable soils. The model in Dafalias and Manzari (2004), a critical state two-surface plasticity model, was used in this paper. This model requires fifteen material parameters and two state parameters to describe the behavior of sands and has been amply tested for simulating the behavior of granular soils subjected to monotonic and cyclic loadings (Jeremic et al., 2008; Taiebat et al., 2010; Rahmani and Pak, 2012). The key advantages of the model are that (1) it is relatively simple and (2) it has a unique calibration of input parameters. Thus, a single set of parameters independent of void ratio and effective consolidation stress level was used for the Dafalias and Manzari's material model. Table 1 presents the material parameters for Nevada sand. The additional parameters used for free-field analysis are presented in Table 2. It can be noted that at the onset of liquefaction, change of soil particles creates additional pathways for water. This leads to a significant increase in permeability coefficient (Rahmani et al., 2012). In this study, the permeability coefficient value was increased 10 times the initial value (suggested by Rahmani et al. (2012)).

For free-field analysis, the simulations were carried out in two loading stages. At the first stage, the soil skeleton and pore water weight were applied to soil elements. The values of stress and strain in this stage were used as initial values for the next stage of loading. At the second stage, dynamic analysis was performed by application of an input motion to the model base.

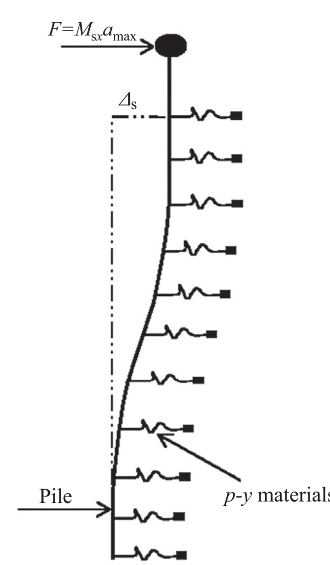

Fig. 1. A beam on the nonlinear Winkler foundation (BNWF) model for pseudo-static analysis.