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Research Paper

Finite element model for piles in liquefiable ground

Rui Wang [a], Pengcheng Fu [b], Jian-Min Zhang [a,*]

[a] School of Civil Engineering/State Key Laboratory of Hydroscience and Engineering, Department of Hydraulic Engineering, Tsinghua University, Beijing 100084, China
[b] Atmospheric, Earth, and Energy Division, Lawrence Livermore National Laboratory, Livermore, CA 94550, USA

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ABSTRACT

This paper develops a three dimensional finite element modelling method for piles in liquefiable ground and applies it to the analysis of seismic pile responses. A unified plasticity model for large post-liquefaction shear deformation of sand provides the basis for the effective and efficient modelling of liquefiable ground. Special attention is dedicated towards the modelling of piles and soil–pile interface to accurately reflect the behaviour of piles. A staged modelling procedure is adopted to appropriately generate the initial conditions for the soil and piles and achieve hydrostatic pore pressure prior to seismic loading. Three centrifuge shaking table tests on single piles, both with and without pile cap and superstructure, in level and inclined liquefiable ground are conducted and simulated in validation and application of the proposed method. Further studies to investigate the effects of pile cap, lateral spreading, and non-liquefiable surface layer are undertaken numerically using the validated method. The results show these aforementioned factors to be influential in the dynamic and residual response of piles in liquefiable ground.

1. Introduction

Since the mass occurrences of pile foundation failures in the 1964 Alaska and the 1964 Niigata earthquakes [1,2], damage to pile foundations in liquefiable ground have been observed in numerous strong earthquakes (e.g. [3–7]). It is well recognised that the analysis of the seismic response of piles in liquefiable ground is an extremely important and, due to its intrinsic complexity, challenging subject in geotechnical earthquake engineering. Such analyses have evolved from simple static methods to more sophisticated high fidelity numerical simulations.

A variety of pseudo-static analysis methods have been proposed and adopted by design guidelines and codes for assessing the behaviour of piles in liquefiable ground. The Japanese Road Association (JRA) [8] and Dobry et al. [9] suggested force-based methods that treat liquefied soil layers as a limit lateral pressure acting on piles. Many other studies have adopted displacement-based approaches in the form of a static "beam on nonlinear Winkler foundation (BNWF)" or the "*p*–*y*" method, where soil resistance is reflected through a series of nonlinear springs attached to the pile. After introducing the nonlinear *p*–*y* method for laterally loaded piles [10,11], Matlock [12], Reese and O'Neill [13], API [14], and others established *p*–*y* curves for clays and sands that have been widely adopted. Subsequently, the *p*–*y* method was extended to liquefiable soils by applying a "*p* multiplier" [15,16], or by developing *p*–*y* curves for liquefied sand [17]. Combining the force- and displacement-based methods, Cubrinovski et al. [18] proposed to use limit pressures for non-liquefied crust layers and linear springs with a "stiffness degradation factor" for liquefied layers during liquefaction-induced lateral spreading. While the aforementioned pseudo-static methods are able to reflect the basic force–displacement relationship of soil–pile interaction and can be performed with ease, they are incapable of capturing the dynamically evolving soil properties and their effects on soil–pile interaction during earthquakes. Pseudo-static methods also suffer difficulties in appropriately combining inertial and kinematic loads (e.g. [19–21]).

Dynamic analysis is not limited by many of the empirical assumptions of pseudo-static methods, and can reflect the progressive changes in soil–pile interaction in liquefiable ground. Based on a dynamic *p*–*y* element developed by Boulanger et al. [22] that incorporated elastic, plastic, damping and gap components, Brandenberg et al. [23] associated the capacity of the *p*–*y* material linearly with the effective stress in the free-field for the degradation of *p*–*y* behaviour due to liquefaction. Liyanapathirana and Poulos [19] used a degraded soil stiffness in their *p*–*y* formulation to take liquefaction into consideration. These methods utilise the ground motion and effective stress obtained from free-field site response analysis, but cannot accurately consider near-field

* Corresponding author.
E-mail address: zhangjm@mail.tsinghua.edu.cn (J.-M. Zhang).

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properties. Varun [24] proposed a semi-empirical method of generating near-field pore pressure from free-field values and plastic work in the p–y element, which to some extent incorporates the effect of near-field soil. Although dynamic p–y methods provide a useful means to reflect the interaction between pile and free-field soil in liquefiable ground, they over-simplify the dynamic response of soil and the approximation of the material properties tend to be rather crude.

Three-dimensional (3D) dynamic continuum methods can model soil–pile interaction in liquefiable ground with a high fidelity by properly taking into account the effects of kinematic and inertial interactions, the effects of pore water pressures, and non-linear constitutive behaviour of soil [25]. Finn and Fujita [26] proposed a 3D finite element model that used an equivalent linear constitutive model for soil and beam elements which were connected directly to the soil elements for piles. However, Wotherspoon [27] and Sanchez and Roesset [28] reported that because beam elements do not reflect the geometrical cross section of the pile, such approach tends to underestimate the stiffness of the pile. As an improvement, Cheng and Jeremic [29] and Lu et al. [30] created a void in the finite element mesh to represent the pile geometry, and connected the pile beam–column elements with surrounding soil elements using rigid beam–column links, aiming at physically representing the pile cross section. Fully 3D models representing piles with solid elements can be used with appropriate element types and meshing accounting for bending of the pile [31], thereby providing a more realistic physical representation of the soil–pile system.

Soil constitutive model is a crucial component in these dynamic continuum simulation methods and should adequately reflect physical processes including plasticity, dilatancy, cyclic mobility, and especially post-liquefaction deformation. Various constitutive models been developed to capture the cyclic mobility and liquefaction features of sands. Widely used models include the UBCSAND models (e.g. [32,33]), generalised plasticity models (e.g. [34,35]), multi-surface models (e.g. [36,37]), and bounding surface plasticity models (e.g. [38–42]). In the 3D dynamic finite element simulations described above, Cheng and Jeremic [29] used the Dafalias–Manzari [40] model and Lu et al. [30] used a multi-yield surface plasticity model [37] for sand. Although each of these models has its own unique features and advantages, most fell short of providing adequate description of the post-liquefaction behaviour of sand [43].

This paper presents a three dimensional finite element analysis method and applies it on modelling the seismic response of piles in liquefiable ground. A novel plasticity model developed by Wang et al. [43] combined with u–p solid–fluid coupled element formulation is used to simulate the soil medium, which is concisely presented in Section 2. In Section 3, special attention is paid to modelling of piles to accurately reflect the bending of piles and soil–pile interaction. Section 4 presents the staged modelling procedure for generating realistic initial conditions prior to seismic loading. Three centrifuge shaking table tests containing single piles and their finite element simulation setup are described in Section 5. The proposed method is validated against test results in Section 6, and then further applied in a preliminary investigation of the effects of pile cap, lateral spreading, and non-liquefiable surface layer in Section 7.

2. Sand model

2.1. Constitutive formulation

Aside from being able to reflect the basic elastic–plastic behaviour, the constitutive model used for liquefiable sand surrounding piles is desired to: (1) reflect the cyclic mobility of sand; (2) capture the generation and accumulation of shear strains at liquefaction as observed in laboratory tests (e.g. [44–46]); and (3) account for the behaviour of sand at various densities and confining stress, as the density and confining stress is expected to change during shaking, especially near the piles. To this end, a unified plasticity model for large post-liquefaction shear deformation of sand [43] was developed within the bounding surface plasticity framework [47]. The constitutive model incorporates a physics-based formulation for post-liquefaction shear deformation and directly links cyclic mobility with dilatancy through a unique decomposition of volumetric strains, while in compliance with critical state soil mechanics concepts. These formulations enable a unified description for pre- to post-liquefaction stages under monotonic and cyclic loading.

Key features related to dilatancy and post-liquefaction deformation in the model are briefly discussed here, while a more detailed description of the model can be found in Wang et al. [43]. The model introduced a unique decomposition of volumetric strain ε_v:

$$\varepsilon_v = \varepsilon_{vc} + \varepsilon_{vd,ir} + \varepsilon_{vd,re} \tag{1}$$

where ε_{vc} is induced by mean effective stress change, $\varepsilon_{vd,re}$ is induced by reversible dilatancy, and $\varepsilon_{vd,ir}$ caused by irreversible dilatancy. This scheme was proposed by Shamoto and Zhang [48] and Zhang [49] based on observations from drained cyclic torsional tests on sand, and directly connects cyclic mobility with dilatancy.

Based on this decomposition, post-liquefaction shear deformation can then be generated at liquefaction. ε_{vc} has a unique dependency on the effective confining stress at any non-zero stress states, and a threshold value $\varepsilon_{vc,0}$ exists for which zero effective stress is reached. Once zero effective stress state (i.e. liquefaction) is reached, ε_{vc} would then be determined by volumetric compatibility in Eq. (1). For sand to leave the liquefaction state where ε_{vc} exceeds $\varepsilon_{vc,0}$, a corresponding amount of dilatancy induced volumetric strain is needed, which requires sufficient plastic shear strain ε_q^p according to dilatancy relations, causing post-liquefaction shear deformation as:

$$\varepsilon_{vd,ir} + \varepsilon_{vd,re} = \int D\left|\dot{\varepsilon}_q^p\right| \tag{2}$$

Similar to the decomposition of volumetric strain, dilatancy rate D is also decomposed into a reversible part D_{re} and an irreversible part D_{ir}:

$$D = \frac{\dot{\varepsilon}_v^p}{\left|\dot{\varepsilon}_q^p\right|} = D_{re} + D_{ir} \tag{3}$$

where the dilatancy rates are formulated based on experimental observations. Although formulated rather differently, the decomposition of dilatancy in the current model holds a similar notion to the fabric dilatancy tensor described by Dafalias and Manzari [40] in terms of correctly reflecting the development of dilatancy after load reversal, hence reproducing the cyclic mobility of sand. The reversible and irreversible dilatancy concept in the proposed model allows for the development of post-liquefaction shear strain with increasing load cycles.

The model complies with critical state soil mechanics to provide a unified description of sand at various state, and has been validated against a wide range of monotonic/cyclic drained/undrained laboratory tests and has shown great capabilities in providing unified description of sand behaviour under the considered conditions [43]. Fig. 1 shows a simulation of undrained cyclic torsional test on Nevada sand conducted by Kutter et al. [44], highlighting its capability in reproducing the cyclic mobility and post-liquefaction shear deformation of sand. The model's accurate description of cyclic mobility is important in reflecting the stiffness degradation for