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Analysis of seismic pile response in liquefiable ground using a constitutive model for large post-liquefaction deformation

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ABSTRACT

This paper presents a three-dimensional finite element study on single piles in liquefiable soil using a unified plasticity model for the analysis of large post-liquefaction shear deformation of sand. The constitutive model is capable of providing a unified description of the monotonic, cyclic and post-liquefaction behaviour of sand at different states, by applying a unique decomposition of volumetric strains and formulations for dilatancy. Applying the model, three dimensional solid-fluid coupled finite element analysis of seismic single pile response in liquefied is conducted. The analysis method is first validated in simulation of a centrifuge shaking table test on single pile in liquefiable ground, and hence utilized to investigate the effects of kinematic and inertial interaction on pile bending moment.

Introduction

The seismic response of piles in liquefiable ground is a dynamic nonlinear three-dimensional problem. Ongoing research efforts in dynamic coupled formulations for soil skeleton–pore fluid problems (e.g. Zienkiewicz et al, 1999) and constitutive models for sand liquefaction (e.g. Yang and Elgamal, 2003; Boulanger and Ziotopoulou, 2013; Wang et al, 2014) have made three-dimensional dynamic continuum methods more effective and appealing for the analysis of piles in liquefiable ground (e.g. Finn, 2004; Cheng and Jeremic 2009). The dynamic continuum approach has the advantage of providing a more rational analysis for the soil-pile kinematic interaction and the structure-foundation inertial interaction, and especially the coupling of the kinematic and inertial interactions. The coupling of the kinematic and inertial forces have traditionally been treated as linear combinations of the peak kinematic and inertial loads with various coefficients (Liyanapathirana and Poulos, 2005; Caltrans, 2013), but has shown to be more complicated through experiments (Tokimatsu et al, 2005; Brandenberg et al, 2005).

This paper aims to simulate and investigate the seismic response piles in liquefiable ground through a three-dimensional solid-fluid coupled finite element analysis approach, with focus on the combined effects of kinematic and inertial interaction. A novel constitutive model for the analysis of large-post-liquefaction is presented and applied in the simulation of piles in liquefiable ground. The simulation method is validated against a centrifuge shaking table test on a single pile usystem, and then applied to study the roles of soil-pile kinematic interaction and the structure-foundation inertial interaction.

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Constitutive Formulation

One of the most important aspect of three-dimensional dynamic continuum analysis of piles in liquefiable ground is the choice of appropriate constitutive models for sand. A unified plasticity model for large post-liquefaction shear deformation was formulated to appropriately reflect the cyclic mobility (strongly related to dilatancy) and large post-liquefaction shear deformation.
The elastic shear and bulk moduli for the model follow the formulation of Richart et al (1970). A state parameter (Been and Jefferies, 1985) was introduced in the model formulations to provide unified description of sand under different densities and confining pressures through compliance with the critical state theory:

$$\Psi = e - e_c \tag{1}$$

with e being the current void ratio and e_c the critical void ratio.
The model operates within the framework of bounding surface plasticity by adopting modifying features from a model by Wang et al (1990) for plastic modulus and its respective mapping rule (Fig. 1). With the plastic modulus formulated as:

$$H = \frac{2}{3} hg\left(\bar{\theta}\right) G \exp(-n^p \Psi)\left(\frac{M \exp(-n^b \Psi)}{M_m}\left(\frac{\bar{\rho}}{\rho}\right) - 1\right) \tag{2}$$

where h and n^p is are model constants, $\bar{\theta}$ is the lode angle, Ψ is a state parameter, M is the critical state stress ratio, M_m is the maximum stress ratio history during loading, $\bar{\rho}$ is the distance between $\bar{\mathbf{r}}$ and $\boldsymbol{\alpha}_{\mathbf{in}}$ (Figure 1), and ρ the distance between $\mathbf{r}$ and $\boldsymbol{\alpha}_{\mathbf{in}}$ (Figure 1).
The model decomposes dilatancy in to reversible and irreversible components based on observations of drained cyclic torsional tests (Shamoto et al, 1997), with the rate of each respected component defined as:

$$D_{re} = \begin{cases} \sqrt{\frac{2}{3}} d_{re,1}\left(\mathbf{r_d} - \mathbf{r}\right) : \mathbf{n}, & (\mathbf{r_d} - \mathbf{r}) : \mathbf{n} < 0 \\ \left(d_{re,2}\chi\right)^2 / p, & (\mathbf{r_d} - \mathbf{r}) : \mathbf{n} > 0 \end{cases} \tag{3}$$

$$D_{ir} = d_{ir} \exp(n^d \Psi - \alpha \varepsilon_{vd,ir})(< \left(\mathbf{r_d} - \mathbf{r}\right) : \mathbf{n} > \exp(\chi) + \left(\frac{\gamma_{d,r} < 1 - \exp(n^d \Psi) >}{\gamma_{d,r} < 1 - \exp(n^d \Psi) > + \gamma_{mono}}\right)^2) \tag{4}$$

where D_{re} and D_{ir} represent the reversible and irreversible dilatancy rate. $d_{re,1}$, $d_{re,2}$, d_{ir}, n^d, α, $\gamma_{d,r}$ are material constants for dilatancy. χ is a function enhancing the dilatancy at load reversal. γ_{mono} is the cumulative shear strain since the last stress reversal.

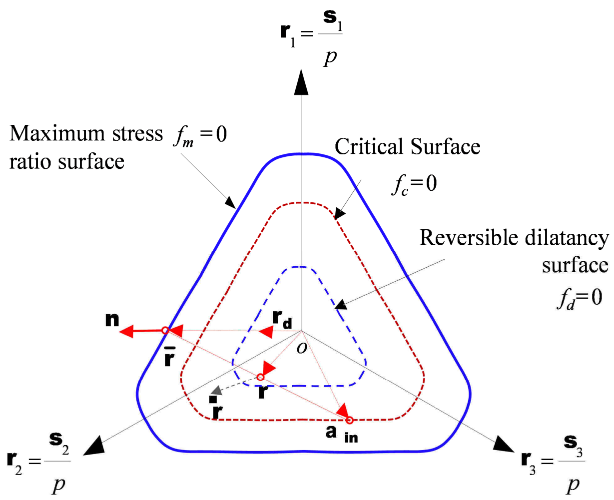

Figure 1. Surfaces and mapping rules

This unique formulation of dilatancy is vital in reflecting the cyclic mobility of sand and allow the undrained cyclic stress path to reach all the way to liquefaction, with reversible dilation generating and releasing during each load cycle and irreversible dilation accumulating asymptotically. At liquefaction, the elastic response of sand is considered unchanged while dilation is assumed to continue, thus generating an increasing shear strain each cycle at the state of liquefaction. Table 1 lists the model parameters and explains their physical meanings and calibration methods.

Table 1. Details of model parameters.

Parameter	Physical meaning	Calibration method*	Value for Fujian sand
G_o	Elastic shear modulus constant	Small strain T	200
κ	Rebound index	Rebound D T	0.006
h	Plastic modulus constant	D T	1.7
M	Stress ratio at critical state in compression	D/U T	1.3
$d_{re,1}$	Reversible dilatancy generation rate constant	D C T	0.45
$d_{re,2}$	Reversible dilatancy release rate constant	D C T	30
d_{ir}	Irreversible dilatancy rate constant	U C T	0.6
α	Decrease rate constant of irreversible dilatancy	U C T	40
$\gamma_{d,r}$	Reference shear strength length	U C T	0.05
n^p	State constant for bounding surface	D T	1.1
n^d	State constant for reversible dilatancy surface	D C T	8.0
λ_c			0.023
e_0	Critical state constants	D/U T	0.837
ξ			0.7

* Note: U is undrained, D is drained, C is cyclic, and T is triaxial (can also be torsional for cyclic tests).

FEM Simulation

The constitutive model has been implemented into the FEM framework OpenSees (McKenna and Fenves, 2001) with the tag CycLiqCPSP. Combined with u-p form brick elements, soil liquefaction analysis can be achieve through soil–pore fluid coupled formulations, enabling the FEM simulation seismic pile response in liquefiable ground. Using this FEM approach, a centrifuge shaking table test on a single pile in saturated sand was simulated.

The shaking table test was conducted at the geotechnical centrifuge facility at Tsinghua University under 30 g centrifugal acceleration, with a single direction horizontal excitation input at the base of the model. The model was constructed within a laminar box to achieve periodic boundaries. A 6m long square pile was installed vertically into a ground consisting of two layers of Fujian sand, a 5m medium dense (*D*r=50%) layer sand overlying a 2.5 m dense (*D*r=80%) layer. The piles used were square aluminium piles with EI = 47.25 MNm^2. A HPMC (hydroxypropyl methylcellulose) solution with 30 times the viscosity of water was used as the pore fluid. A pile cap with a 10.8t superstructure on top was connected to the pile head. All parameters and measurements are given in prototype scale.

The finite element mesh for the numerical simulation of the centrifuge shaking table test is shown in Figure 2, which is only half of the actual physical model due to symmetry. The pile was simulated with second order brick elements and linear elastic isotropic constitutive model, the cross section of the pile in the simulations consisted of 6 elements to accurately calculate the bending moment and curvature of the pile. The two layers of sand were simulated using up elements and the unified plasticity model for large post-liquefaction shear deformation of sand. The model parameter values used in the simulations are shown in Table 1, the elastic shear modulus parameter (G_0), plastic modulus parameter (h) and critical state stress ratio (M) was obtained from drained triaxial test data, and elastic bulk modulus parameter (κ) was determined via the rebound curves of triaxial consolidation tests. The critical state parameters (λ_c, e_0, ξ) for Fujian sand reported by Yang and Sze (2011) were used.

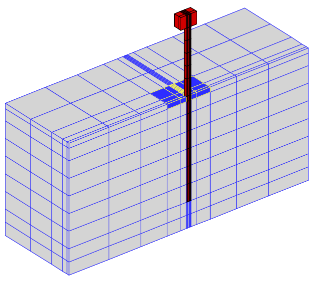

Figure 2. FEM mesh for the simulation of a single pile in liquefiable ground

Figure 3 shows typical results of horizontal acceleration and excess pore pressure in the ground from both test measurement and numerical simulation. It can be seen from comparing the two set of results that the numerical simulation well reproduced the seismic response of the liquefiable ground. The maximum input acceleration at the base of the model was -4.95 m/s^2, occurring at 6.76s, while the maximum acceleration at the ground surface was less than -3 m/s^2 and was greatly deamplified due to the buildup of excess pore pressures and subsequent decrease in effective stress in the ground. In both the centrifuge test and the numerical simulation, the top 4m of sand reached liquefaction after about 10s, with excess pore pressure ratio r_u = 1.0.

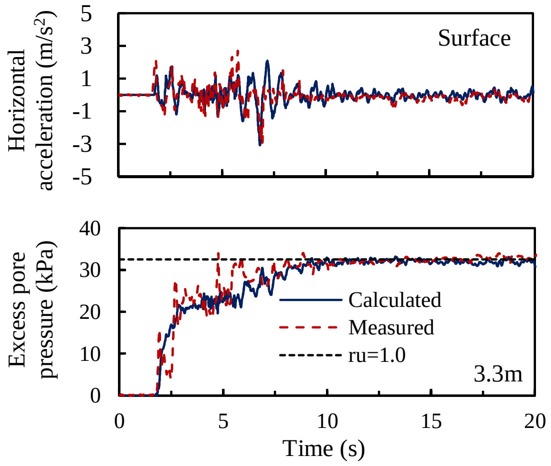

Figure 3. Calculated and measured acceleration and excess pore pressure

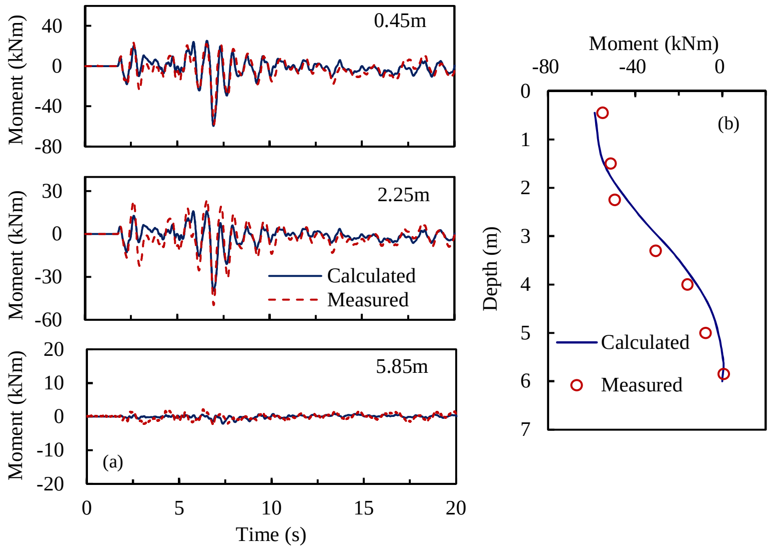

Figure 4. Calculated and measured pile moment: (a) pile moment histories at three depth, (b) peak pile moment along pile depth

The seismic response of the pile is illustrated in terms of pile moment in Figure 4. The calculated pile moment histories and the peak pile moment distribution along the pile are in good agreement with results from the centrifuge test. The maximum bending moment in the pile was -58kNm in the test and -60kNm in the simulation, which occurred later than the time of peak input acceleration, at 6.96s. The maximum bending moment was observed at the pile head, while the pile tip was free to rotate and had no significant moment.

Kinematic and Inertial Interaction

Figure 5 (a) plots the moment at pile head against soil surface displacement and structure acceleration in the simulation of the centrifuge test, where the super-structure was very rigid and had a period of 0.05s. The pile moment is negatively correlated to soil surface displacement while being positively correlated to structure acceleration, and the peak moment (negative) occurred simultaneously with the peak soil surface displacement (positive) and structure acceleration (negative). However, if the structure was more flexible these relationship could change. Figure 5 (b) shows the results from a case of 5s structure period, with everything else unchanged from the simulation of the centrifuge test. For this case, while the moment is still negatively correlated to displacement, there is no significant correlation between moment and acceleration with the structure acceleration been much smaller.

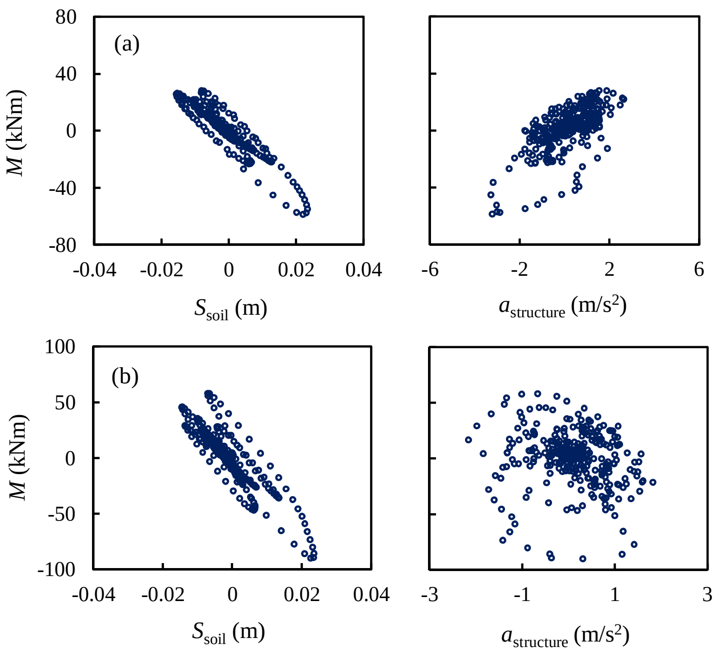

Figure 5. Pile head moment in relationship to soil surface displacement and structure acceleration: (a) structure period of 0.05s, (b) structure period of 5s.

To further investigate the influence and coupling of the kinematic and inertial interactions, calculations were conducted using the existing simulation setup but with varying ranges for structure flexibility and soil modulus (Figures 6 and 7). Figure 6 shows that the peak pile moment does not necessarily increase with larger structure acceleration and hence larger inertial force, and their correlation depends on the structure period. Within the 300% change in structure acceleration, only a 61% change in pile moment was observed. However, Figure 7 shows that the peak pile moment constantly increases with increasing soil displacement, yielding a 115% change in pile moment for 91% change in soil surface displacement.

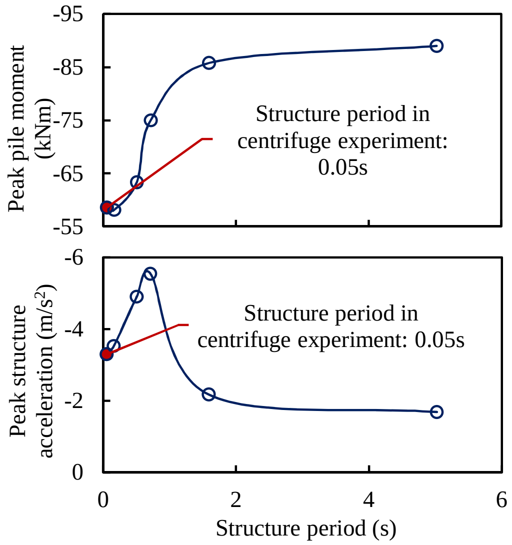

Figure 6. Peak pile moment and structure acceleration for various structure periods.

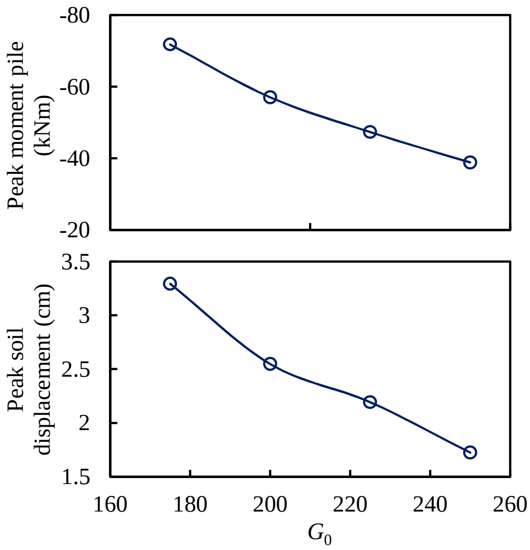

Figure 7. Peak pile moment and structure acceleration for various soil stiffness.

Conclusions

In this paper, the basic formulations for a unified plasticity model for the analysis of large post-liquefaction shear deformation of sand was presented and applied to three-dimensional finite element simulation and analysis of single piles in liquefiable soil. The simulation of a centrifuge shaking table test showed good agreement between numerical and test results.

The method was then utilized to investigate the effects of kinematic and inertial interaction on pile response. For the case of single pile with pile cap in liquefiable ground studied in this paper, calculation results showed that kinematic interaction played a more prominent role in pile moment. Pile moment was negatively correlated to displacement and thus kinematic interaction, while its correlation with structure acceleration depended on the period of the structure, the coupling of these two interactions shouldn't simply be viewed as a linear combination.

Acknowledgments

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